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BUILDING CONSTRUCTION UNDER SEISMIC CONDITIONS IN THE BALKAN REGION

VOLUME 1

DESIGN AND CONSTRUCTION OF SEISMIC RESISTANT REINFORCED CONCRETE FRAME AND SHEAR-WALL BUILDINGS

BUILDING CONSTRUCTION
UNDER SEISMIC CONDITIONS
IN THE BALKAN REGION

UNDP/UNIDO PROJECT RER/79/015

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DESIGN AND CONSTRUCTION OF SEISMIC RESISTANT REINFORCED CONCRETE FRAME AND SHEAR-WALL BUILDINGS

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION
executing agency for the
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VOLUME 1

DESIGN AND CONSTRUCTION OF SEISMIC RESISTANT REINFORCED CONCRETE FRAME AND SHEAR-WALL BUILDINGS

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PREFACE

The Regional Project "Building Construction under Seismic Conditions in the Balkan Region", UNDP/UNIDO RER/79/015, has been carried out with the participation of the Governments of Bulgaria, Greece, Hungary, Rumania, Turkey and Yugoslavia, and with the United Nations Industrial Development Organization acting as Executing Agency for the United Nations Development Programme. Mr. J.G. Bouwkamp served as Chief Technical Advisor.

Within the framework of the Project, a set of seven Manuals has been produced, reflecting to a considerable extent the experience of the participating nations in earthquake resistant design and construction. These Manuals were developed by the National Delegates of the Project Working Groups, the Chief Technical Advisor and the Consultants.

The following Manuals have been prepared:

Volume 1 : Design and Construction of Seismic Resistant Reinforced Concrete Frame and Shear-Wall Buildings

Volume 2 : Design and Construction of Prefabricated Reinforced Concrete Building Systems

Volume 3 : Design and Construction of Stone and Brick-Masonry Buildings

Volume 4 : Post-Earthquake Damage Evaluation and Strength Assessment of Buildings under Seismic Conditions

Volume 5 : Repair and Strengthening of Reinforced Concrete, Stone and Brick-Masonry Buildings

Volume 6 : Repair and Strengthening of Historical Monuments and Buildings in Urban Nuclei

Volume 7 : Seismic Design Codes of the Balkan Region

ACKNOWLEDGEMENTS

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The Project has been directed by the Project coordinating Committee. The membership of the Committee was as follows:

Bulgaria	G. Brankov
Greece	Th. Tassios and J. Sbokos
Hungary	F. Hunyadi
Rumania	V. Cristescu
Turkey	M. Erdik
Yugoslavia	N. Vukotic

From the United Nations the following individuals participated in the deliberations of the Coordinating Committee:

O.A. Nordstrand - UNDP, Athens, Greece, Resident Representative and UNDP Principal Project Representative,
E. Csorba - UNIDO, Vienna, Austria, Senior Industrial Development Officer, and
J.G. Bouwkamp - UNIDO, Thessaloniki, Greece, Chief Technical Advisor.

DISCLAIMER

The material contained in these Volumes includes detailed findings in earthquake engineering - particularly, objective evaluations of causes and effects in earthquake damage - and in the seismic and geologic characteristics of the physical environment.

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NOTE

Earthquakes have caused considerable loss of life and property damage in the Balkan Region and are considered as the most important natural hazard in this region. Improved building practices leading to better structures to resist the seismic forces are effective means for reducing these losses.

The main objective of this Manual is to present basic principles and procedures for the design and construction of earthquake-resistant reinforced concrete framed and shear-wall structures in the Balkan region. The Manual is intended to serve as an aid for engineers and to provide guidance in design and construction of seismic-resistant reinforced concrete buildings.

In developing this document, available analytical and experimental research data, major seismic design codes and lessons learned from past earthquake damages in Balkan countries have been taken into account. In addition, aspects associated with the economic, social and technical level of the Balkan countries have been considered.

The recommendations presented in this Manual are principally intended for new structures. Although the basic principles of this Manual may also apply to strengthening of existing buildings and design of "special structures", additional rules and requirements are needed for such cases. Special structures and strengthening are outside the scope of this Manual. The Manual is not intended to cover the case of buildings of very small socio-economic importance (small rural houses, etc.).

The Manual consists of three Parts:

Part 1 - Summary of Detailing and Proportioning Requirements
Part 2 - Design of Seismic Resistant Reinforced Concrete Structures
Part 3 - Design Examples

Part 1 presents a summary of minimum detailing requirements for seismic resistant structures and Part 2 contains the main text. In the latter Part, detailed information is given on the seismic behavior of reinforced concrete members and structures. Design objectives and criteria, and detailing principles are discussed. A guideline is also presented on quality control. Part 3 contains three design examples, namely, a shear-wall building, a frame building and a building with a mixed, frame-shear wall, or dual system. These design examples have been prepared by the national delegates of Rumania, Bulgaria and Yugoslavia, respectively, and are based on the pertinent National Building and Seismic Codes. Simple methods suitable for hand calculations have been used for lateral load analysis.

In the last section of Part 3, dynamic time-history analyses have been carried out for all three examples. The design examples have been re-evaluated in the light of results obtained from these dynamic analyses.

The Working Group consisted of National delegates of the participant countries with Dr. Ugur Ersoy, Professor, Middle East Technical University, Ankara, Turkey, serving as Convenor. Other members of the Working Group were: Dr. Mincho Dimitrov, Building Research Institute, Sofia, Bulgaria; Dr. Dan Dumitrescu, Professor, Institute of Construction, Bucarest, Rumania; Dr. Vladimir Kalevras, Professor, University of Thrace, Xanthi, Greece; Dr. Boris Simeonov, Professor, IZIIS, Skopje, Yugoslavia. Consultants of the Working Group were Dr. Mete Sozen, Professor, University of Illinois, Urbana, Ill., USA and Dr. Miodrag Velkov, IZIIS, Skopje, Yugoslavia.

Professor Jack G. Bouwkamp, Professor, University of California, Berkeley, California, USA, served as Project Chief Technical Advisor and participated in all Working Group discussions.

PART 1

SUMMARY OF DETAILING AND PROPORTIONING REQUIREMENTS

1. INTRODUCTION

This summary has been prepared to be used as a design aid for proportioning and detailing reinforced concrete structures to accomodate displacements and forces associated with strong earthquake motions.

Observed behavior of reinforced concrete structures in earthquakes and in the laboratory, as well as response analyses indicate that critical considerations in producing earthquake-resistant construction are:

(1) Selection of structural configuration
(2) Choice of details and proportions
(3) Quality control

Selection of overall geometry and types of lateral-force resisting elements (walls, frames and trusses) is a very important decision with respect to performance and economy of the structure. Unwise decisions made at this stage may be very expensive and sometimes impossible to be compensated for by measures prescribed in later stages of design. Examples of structural configurations to be avoided and those to be preferred are illustrated in Part 2 of this Manual (Part 2, Chapter 5).

Experience and experiments have shown time and again the vital role of proper detailing for earthquake resistance of reinforced concrete construction. The main purpose of this summary is to provide for the structural engineer a set of tables and figures for convenient reference in making decisions about types of details and member proportions.

That the structure must be constructed as required by the engineering drawings and specifications (quality control) demands no proof. For earthquake-resistant construction, quality control is more important because the structure is not likely to be loaded laterally until the earthquake occurs, while for gravity loading the dead load and construction loads do serve toward exposing some of the design errors. Matters related to quality control are discussed in Part 2 of this Manual (Chapter 7).

2. PROPORTIONING AND MINIMUM DETAILING REQUIREMENTS FOR BEAMS, COLUMNS AND WALLS

2.1 Introductory Remarks

Member proportions and minimum reinforcement details are given for two levels of seismic performance. Seismic performance refers to the capability of the entire structural system to accomodate the lateral displacements anticipated in the event of the design earthquake. For seismic performance category B, the structure is expected not to develop a brittle mode of failure when displaced cyclically into the nonlinear range of response. For seismic performance category C, the structure is expected to be tough and to possess the energy-dissipation capacity demanded by severe and long-duration ground motions. Any structure in category C is expected to satisfy all requirements specified for category B as well as those for category C. It can be noted that category A reflects a design developed without seismic load considerations.

It should be pointed out that the details stipulated here are the bare minima considered desirable. Local specifications should govern wherever these requirements exceed those described here. Obviously, it is the prerogative and the responsibility of the structural engineer to make certain that the structure has the required characteristic strengths to resist the load combinations specified by the governing building codes.

Most of the requirements for the two seismic performance categories are summarized in Tables 2.1 through 2.3. Sections below contain the requirements which could not be included conveniently in the Tables.

2.2 Requirements for Beams and Slabs

Variation of Moment Capacity Along Span: For both categories B and C, moment capacity at any section should not be less than one fourth of the capacity at support.

Transverse Reinforcement: For both categories B and C, transverse reinforcement should be provided to resist the shear corresponding to the development of ultimate moment capacitites M_{r1} and M_{r2} (based on the characteristic strengths) and the permanent vertical load g as shown in Fig. 1.1.

Lap Splices: For both categories B and C, lap splicing of tensile reinforcement is permitted only if hoop or spiral reinforcement over length of lap should not exceed h/4 or 150 mm 10 times the diameter of the lapped bar. Lap splices should not be used (a) within joints, (b) within a distance twice the member depth from the joint face, and (c) at locations where flexural yielding may occur.

Eccentricity of Beam and Column Axes: For category C, eccentricity of any beam in relation to the supporting column, measured by the distance between the geometric centers of their sections, should not exceed one fourth of the dimension of the column side to which the beam is attached.

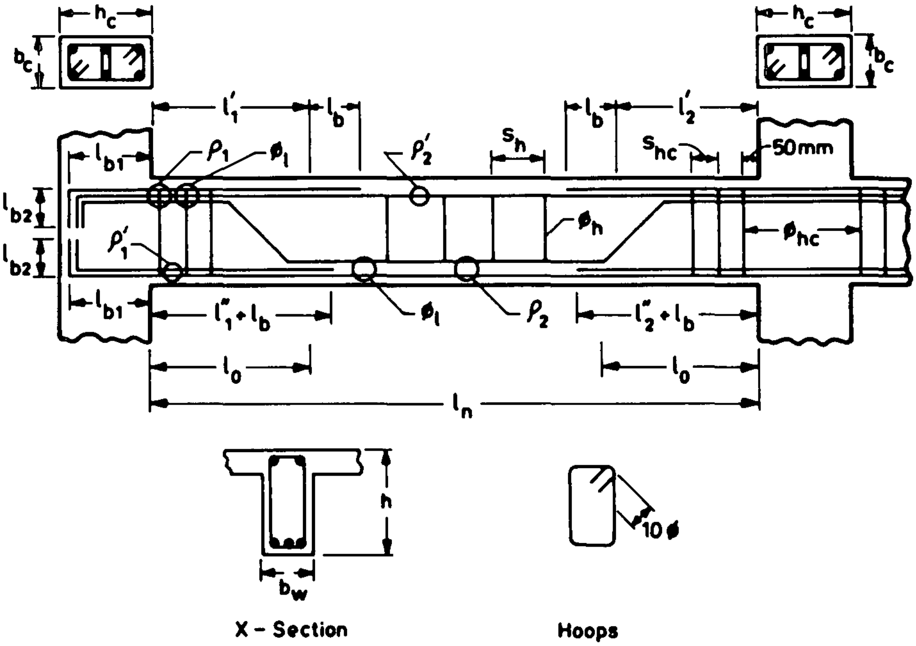
Amount of Flexural Reinforcement to be Considered Effective: For categories B and C, the amount of flexural reinforcement in slabs, L-beams and T-beams to be considered effective in resisting earthquake effects should be placed within widths described below.

(1) At interior columns, within a distance of 2.5 times the slab thickness from each side of the column.
(2) At exterior columns, within the width of the column.

Slabs Used as Diaphragms: In slabs used as diaphragms to transmit forces caused by seismic effects, minimum reinforcement ratio each way at any section should not be less than 0.0025. Spacing of reinforcement should not exceed 200 mm in each direction. Slab thickness should not be less than 150 mm.

2.3 Requirements for Columns and Joints

Transverse Reinforcement: For category B, transverse (hoop) reinforcement should be provided to resist the shear corresponding to the development of ultimate moment capacities M_{r1} and M_{r2} based on the characteristic strengths as shown in Fig. 1.1.



Detail	Seismic Performance Category	
	B	C
min b_w	200 mm	Same as B
max b_w	Column width + 3/4 h	" " "
min h	300 mm	" " "
max h/b_w	4.0	" " "
min ℓ/h	4.0	" " "
min ρ_2 or ρ'_2	$\rho_1/4$ but not less than 2-Ø12	" " "
min ρ max ρ	$1.0/f_{yk}$ 0.025	" " "
ℓ_o (confined region)	2 h	" " "
max s_h	h/2 but not more than 250 mm	" " "
max s_{hc} (confined region)	h/4 but not more than $10Ø_\ell$ and 200 mm	but not more than 8Ø and 150 mm
max s_{hs} (splices)	h/4 but not more than $10Ø_\ell$ and 150 mm	Same as B
min ρ'_1/ρ_1	1/2	" " "
min $Ø_h$	6 mm	8 mm
min $Ø_{hc}$	8 mm	10 mm
min $(\ell'_i+\ell_b)$	$\ell_n/4$	Same as B
min ℓ_{b1}	$250\ Ø_\ell$	" " "
min ℓ_{b2}	$12\ Ø_\ell$	" " "
max V	$0.15\ f_{ck}$	" " "

Table 2.1 - Requirements for Beam Detailing

For category C, the shear to be resisted should be 1.15 times that defined for category B.

Lap Splices : For categories B and C, lap splices should be only within the center half of the member length. Lap length should be determined on the basis of requirements for a tension splice.

Reinforcement for Confinement : For category B, hoops should be provided over length ℓ_o (as defined is Table 2.2) measured from each end of column. Over these lengths ℓ_o, diameter of the hoop reinforcement should not be less than 8 mm. Spacing of the hoops should not exceed (a) one half of the shorter cross-sectional dimension of the column, (b) ten times the diameter of the longitudinal reinforcement, and (c) 200 mm.

For category C, requirements in addition to those for category B are the following.

Hoop reinforcement over length ℓ_o should not exceed 100 mm.

Confinement reinforcement over length ℓ_o should not be less than that indicated by the expressions given below.

For spiral reinforcement,

$$\rho_s = \frac{4A_{sh}}{Ds_{hc}} = 0.12 \frac{f_{ck}}{f_{yk}}$$

For rectangular hoop reinforcement,

$$A_{so} = 0.3 s_{hc} h_c \frac{f_{ck}}{f_{yk}} (\frac{A_c}{A_{ck}} - 1)$$

$$A_{so} = 0.12 s_{hc} h_c \frac{f_{ck}}{f_{yk}}$$

where A_{so} is the total cross-sectional area of transverse reinforcement (including supplementary cross-ties) at a section of length s_h along column axis. Other terms are defined in Section 3 of this summary.

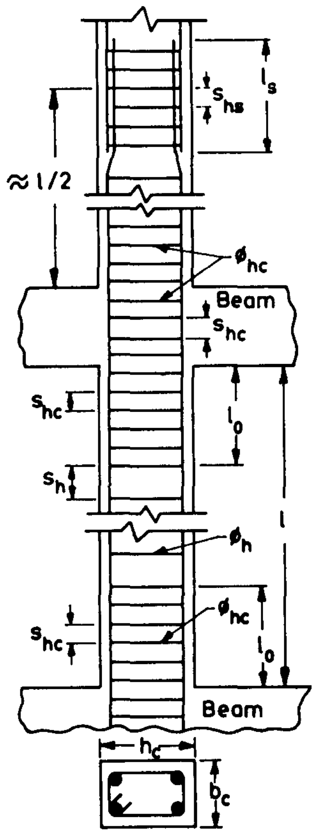
Columns Restrained by Walls : For both categories B and C, columns in contact with nonstructural walls which are stiff enough to shorten the effective height of the column, should have hoop reinforcement as defined above for length ℓ_o over the total height of the column.

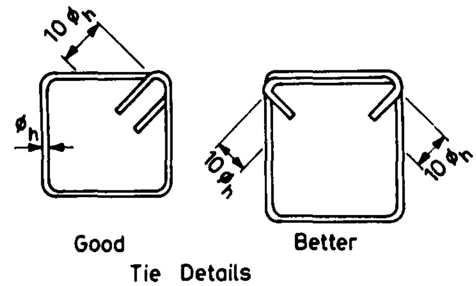
Joints : For category B, confinement reinforcement specified for length ℓ_o should be continued through the joint.

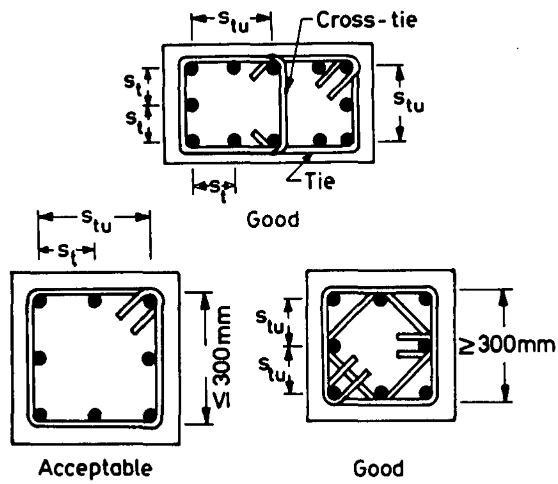
For category C, the joint shear stress should be limited as described below in addition to the requirement for category B.

Unit shear stress in the joint should not exceed $0.25f_{ck}$ for joints laterally confined on all four sides by beams and $0.20f_{ck}$ for joints not laterally confined.

The shear force should be the algebraic sum of forces in the beam longitudinal reinforcement and the column shear. The force in the beam







Some examples of confinement provided by different tie and cross-tie arrangements

Detail	Seismic Performance Category	
	B	C
min b_c	250 mm	
max ℓ/b_c	25	16 (10 for cantilevers)
ℓ_o	h but not less than $\ell/6$ and 450 mm	Same as B
ρ_{min}	0.01 but not less than 4-Ø14 (tied) or 6-Ø14 (spiral)	Same as B
ρ_{max}*	0.04	" " "
ℓ_s	$\alpha\ell_b$**	" " "
max s_{tu}	300 mm	200 mm
max s_t	b/2	Same as B
max s_h	$12\phi_\ell$ but not more than 200 mm	" " "
max s_{hc}	b/2 but not more than $10\phi_\ell$ and 150 mm	But not more than 100 mm
max s_{hs}	b/2 but not more than $10\phi_\ell$ and 150 mm	Same as B
min ϕ_h	8 mm	" " "
min ϕ_{hc}	8 mm	10 mm
max h_c/b_c	4.0	2.5
max $N/A_c f_{ck}$	0.6	0.4
max V	0.15 f_{ck}	Same as B

* Including splices, ** α depends on percentage of lapped bars at that section

Table 2.2 - Requirements for Column Detailing

longitudinal reinforcement may be taken as $A_{si} \cdot f_{yk}$ at each face.

To compute the joint shear stress, the effective width of the joint should be taken as being equal to :

The smaller of (a) column width (cross-sectional dimension perpendicular to beam axis) or (b) beam width plus half of the column cross-sectional dimension parallel to the beam axis, if the column width exceeds that of the beam.

The smaller of (a) the beam width or (b) column width plus half of column cross-sectional dimension parallel to the beam axis, if the column width does not exceed that of the beam.

2.4 Structural Walls

Reinforcement : For categories B and C, reinforcement ratios for vertical and horizontal reinforcement should be equal. Reinforcement required for shear should be distributed uniformly and should be determined from the expression given below :

$$\rho_h = \frac{A_{sh}}{bs_v} = \frac{\tau_d - \tau_c}{f_{yk}}$$

Maximum unit shear stress should not exceed $0.12f_{ck}$.

At least four-12 mm or six-10 mm reinforcing bars should be placed at each edge of the wall cross section and these bars should be confined by Ø8 hoops at a spacing not exceeding 150 mm.

Boundary Members : For categories B and C, boundary members should be provided at edges of all sections where the design compressive stress exceeds $0.2f_{ck}$ for a loading combination including earthquake effects.

Compressive stress should be calculated using a linearly elastic model of the wall using uncracked plain sections.

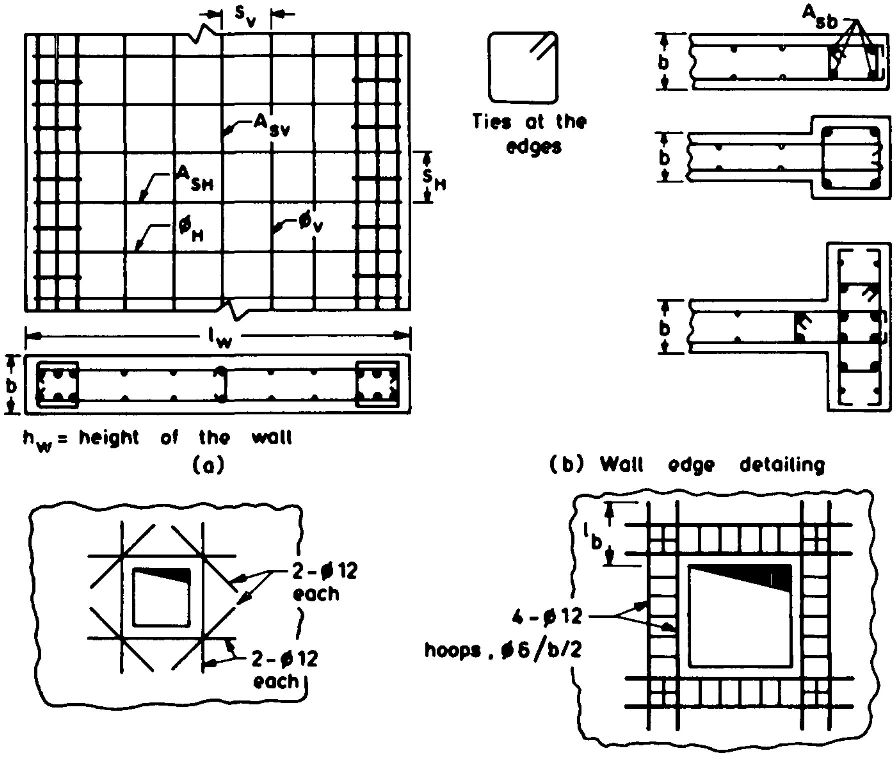
Boundary members should be proportioned using design requirements applicable to short columns. The boundary element on each edge of the section should have the capacity to resist the axial load defined as the sum of all gravity loads on the wall plus the vertical force associated with the overturning moment caused by earthquake effects.

Transverse hoop reinforcement for boundary elements should be as defined for confinement of columns over the length ℓ_o.

Effective Flange Width : The following definitions apply for categories B and C. Two different sets of definitions are given : (a) the first to be used in the case of more than one wall monolithic with walls acting as flanges and (b) the second for a single wall with flanges.

For case (a), the effective flange width on each side of the wall acting as web should be bounded by

(1) half the clear distance to the next wall,
(2) closest boundary of openings in the wall acting as flange,
(3) 10% of the wall height, but the total effective flange width should not exceed the length of the wall acting as web (ℓ_w).



(a)

(b) Wall edge detailing

(c) Detailing for small openings

(d) Detailing for large openings

Detail	Seismic Performance Category	
	B	C
min ℓ_w/b	4.0	Same as B
min b	150 mm but not less than 1/20 floor height	" " "
min h_w/ℓ_w	-	2.0
min ρ_v or ρ_H*	0.0025	Same as B
max ρ_v	0.035	" " "
max ϕ_v or ϕ_H	b/10	" " "
max s_v or s_H	250	" " "
min A_{sb}	4-ø12 or 6-ø10	" " "
Transverse steel at the edges	$\geq$ ø8/150 mm	" " "
max $N/A_c f_{ck}$	1/3	" " "
max V	0.12 f_{ck}	" " "

*Does not include edge or boundary reinforcement

Table 2.3 - Requirements for Detailing of Structural Walls

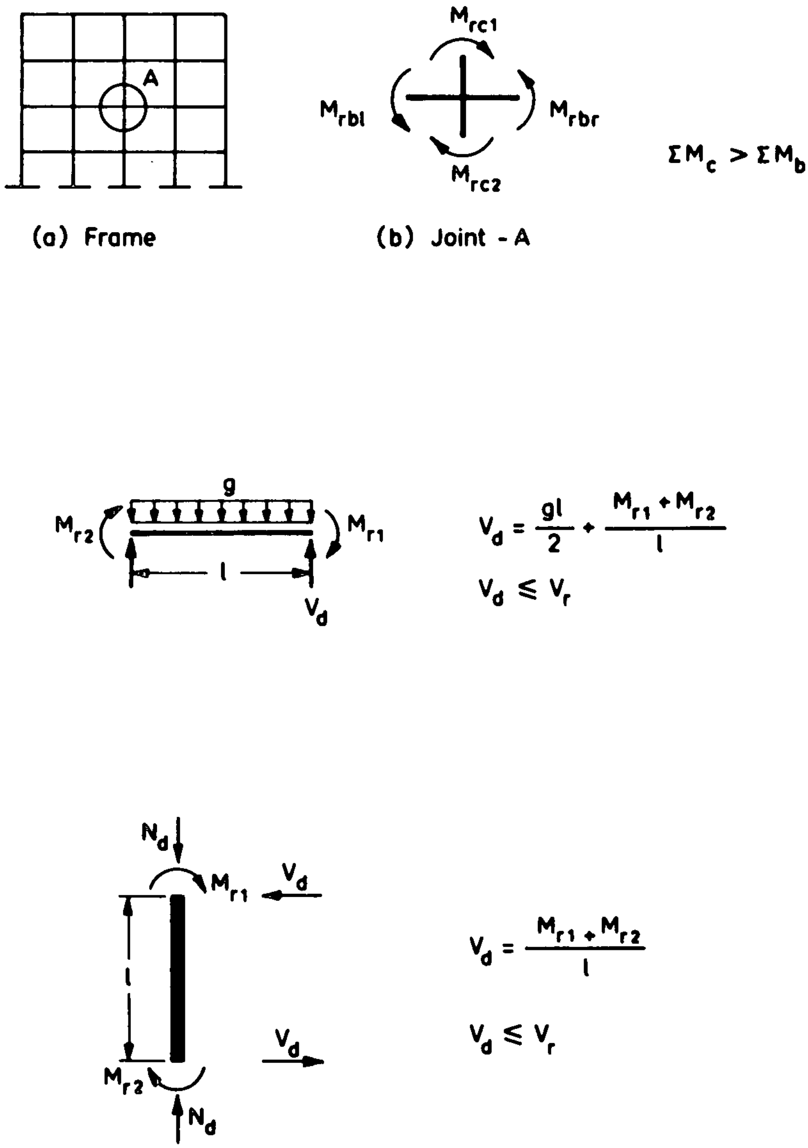
For case (b), the effective flange width on each side of the wall acting as web should be bounded by

(1) 10% of the wall height,
(2) five times the thickness

of the wall acting as the flange, but total effective flange width should not exceed the length of the wall acting as web (ℓ_w).

<u>Splices :</u> For categories B and C, splices of vertical reinforcement in regions of potential yielding should be avoided. In no case should more than 30% of reinforcement be spliced in those regions. Splices should be staggered in the vertical direction by at least twice the splice length.

For category C, not more than 30% of the horizontal reinforcement should be spliced at the same section and splices should be staggered in the horizontal direction by at least twice the splice length. Splices of horizontal and vertical reinforcement in the two curtains should not occur at the same section in both curtains of reinforcement.



M_{rc1}
A
M_{rbl}
M_{rbr}
M_{rc2}
$\Sigma M_c > \Sigma M_b$
(a) Frame
(b) Joint - A
g
M_{r2}
M_{r1}
l
V_d
$V_d = \frac{gl}{2} + \frac{M_{r1} + M_{r2}}{l}$
$V_d \leq V_r$
N_d
M_{r1}
V_d
l
$V_d = \frac{M_{r1} + M_{r2}}{l}$
V_d
$V_d \leq V_r$
M_{r2}
N_d

Fig. 1.1

3. NOTATIONS

A	Area, acceleration
A_c	Gross of concrete section area
A_{ck}	Core area of concrete section
A_s or A_{si}	Cross-sectional area of tensile reinforcement
A_{sb}	Total cross-sectional area of longitudinal reinforcement at the edges (or boundary members) of structural walls
A_{sh}	Cross-sectional area of transverse reinforcement
A_{so}	Total cross-sectional area of transverse reinforcement (including supplementary cross-ties) at a section of length s_h or s_{ch} along column axis
b	Smaller cross-sectional dimension of structural wall
b_c	Smaller cross-sectional dimension of column
b_w	Web width of flexural member
b_f	Flange width of structural wall
D	Core diameter
d	Effective depth
E	Earthquake loading
F	Action, force
f_{bd}	Design bond strength
f_{ck}	Characteristic concrete strength
f_{cd}	Design concrete strength = f_{ck}/γ_{mc}
f_{yk}	Characteristic yield strength of reinforcement
f_{yd}	Design yield strength of reinforcement = f_{yk}/γ_{ms}
G	Permanent load
H	Overall height of the building
h	Total beam depth
h_c	Larger cross-sectional dimension of column
h_i	Height of i'th floor from the foundation level
h_{sx}	Story height
h_w	Height of structural wall
I	Importance factor
K	Behavior factor
ℓ	Span
ℓ_n	Clear span
ℓ_b	Anchorage length
ℓ_o	Length of confined region in beams and columns

ℓ_i' or ℓ_i''	Distance where reinforcement is no longer needed
ℓ_s	Splice length
M_d	Design moment
M_r	Ultimate moment capacity of the section
N_d	Design axial force
N_o	Axial load capacity of the section
n	Number of stories
P	Prestressing force
Q	Variable load
R_d	Resistance or design capacity
S	Site coefficient
S_d	Design load effect
s_t	Distance between two adjacent longitudinal bars
s_{tu}	Maximum distance between two longitudinal bars restrained by a 90 degree bend or a cross-tie
s_h	Spacing of transverse reinforcement
s_{hc}	Spacing of transverse reinforcement at confined regions
s_{hs}	Spacing of transverse reinforcement over the splice length
s_v	Spacing of vertical bars in structural walls
s_H	Spacing of horizontal bars in structural walls
T	Fundamental period
W_i	Vertical load on i'th floor
V	Shear force
V_d	Design shear force
V_r	Ultimate shear capacity
Δ	Deflection or displacement
$\Delta_{e\ell}$	Elastic interstory drift
Δ_u	Maximum displacement
Δ_y	Yield displacement
ϕ	Bar diameter or curvature
ϕ_ℓ	Bar diameter of longitudinal reinforcement
ϕ_h	Bar diameter of transverse reinforcement
ϕ_{hc}	Bar diameter of transverse reinforcement in confined regions
ϕ_u	Ultimate curvature
ϕ_y	Yield curvature
ρ_i	Reinforcement ratio for tensile reinforcement ($A_{si}/b_w d$)
ρ_i'	Reinforcement ratio for compressive reinforcement ($A_s'/b_w d$)
ρ_s	Volumetric ratio of spiral reinforcement
ρ_v	Ratio of vertical reinforcement in structural walls

ρ_H	Ratio of horizontal reinforcement in structural walls
σ_c	Concrete stress
σ_s	Steel stress
τ	Shear stress
τ_d	Design shear stress
τ_c	Shear stress carried by concrete
μ	Ductility factor
γ_{mc}	Material factor for concrete. For cast-in-place concrete, can be taken as 1.5
γ_{ms}	Material factor for steel. Can be taken as 1.15

PART 2

DESIGN OF SEISMIC RESISTANT R/C STRUCTURES

1. INTRODUCTION

The main object in structural design is to produce a serviceable structure which (a) can be constructed conveniently, (b) has a high probability of survival under the anticipated loads, and (c) can be constructed at an acceptable cost. To fullfil these objectives, adequate statistical data should be available on loading and resistance of the structure and structural components. Although quite a number of data on seismic action and on the behavior of members under loads simulating earthquakes are available, these by no means can be considered as adequate. It is important to note that the behavior of reinforced concrete members under repeated reversed loading depends highly on the loading history. Presently, researchers have not yet agreed on the loading history to be used in testing structural components. Therefore, different conclusions have been drawn depending on the severity of the loading history used.

In drafting design rules for the Balkan region, in addition to research results, the International and National Codes of countries having extensive experience in earthquake engineering should also be taken into consideration. However, one should not forget that seismic events observed and building technology in California, New Zealand and Japan are quite different from those of the Balkan region. Therefore it will not be proper to adopt directly the design philosophy and the principles developed for these countries to the Balkan region.

A survey of past earthquakes in Balkan countries reveals that a great majority of the damages or failures have occured due to:

- mistakes made in choosing the structural system or configuration
- inadequate detailing
- inadequate quality control.

In developing the design philosophy and requirements for the design and construction of seismic resistant buildings for the Balkan region, emphasis should be placed on these three points.

A summary of minimum requirements for proportioning and detailing have been presented in Part 1 of this Manual. In Part 2 information on seismic behavior of reinforced concrete buildings will be reviewed and design objectives and design criteria will be summarized. Also principles of good detailing will be discussed and general recommendations will be made for a proper quality control.

2. INFORMATION ON SEISMIC BEHAVIOR OF R/C BUILDINGS

2.1 Introduction

Existing knowledge concerning earthquake behavior of cast-in-place reinforced concrete medium-rise buildings (up to 15 stories) is presented here, so that existing problems are identified and appropriate solutions proposed.

Main sources from which this knowledge has been accumulated are:

a. Analytical research
b. Experimental research
c. In situ measurements
d. Pre-earthquake surveys of existing buildings
e. Post-earthquake surveys.

None of these sources is sufficient by itself. In most cases, reliable knowledge is based on information from more than one source. A considerable amount of data concerning the subject under examination has become available recently, especially in the Balkan region.

The objective of this chapter is the systematic presentation of the highlights of international and regional experience for possible improvement of existing design principles and procedures, detailing techniques and quality control in the region.

What follows is not a state-of-the-art report on the subject, but a concise presentation of basic available information directly related to earthquake behavior of cast-in-place, medium-rise RC buildings in the Balkan region. It is believed that without basic knowledge on behavior, discussion of appropriate requirements and procedures of design is meaningless.

2.2 Analytical Research

Based on the widespread use of computers and the availability of advanced computer program packages, analytical research has advanced considerably during the last decade and has offered a number of valuable results.

Analysis possibilities and problems :

Today, complex RC structures can be analyzed both for the elastic and the inelastic ranges of behavior, for static and/or dynamic excitation. One, two, pseudo-three and three dimensional models can be used.

There are many problems which have not yet been solved for a reliable analysis of structures, some of which as listed below :

a. Structure to structure interaction
b. Frame-infill interaction
c. Bond-slip in joint areas

Although more sophisticated methods of analysis are available, it seems that, in general for the near future, equivalent static analysis will be the predominant practice for the design of cast-in-place R/C buildings. This type of analysis yields reasonable results for buildings with good structural configurations, but may fail to identify overstress problems in case of bad structural configuration.

The engineer should be aware of the main parameters which can lead to overstress. Such parameters are identified in Table 2.1 and are illustrated in Figure 2.1. Overstress problems arising from unavoidable bad structural configuration can be solved either through an appropriate analysis capable of predicting the effects of these problems or through simplified analysis and approximate corrective coefficients |21|*

* Numbers in brackets refer to the reference list given at the end of Part 2 of this manual.

Table 2.1 Overstress Parameters

	Location			
	Structural Elements	Story	Structural System: Plan	Structural System: Elevation
Small section dimensions	(a)			
Slenderness	(b)	(e)	(k)	(n)
Abrupt stiffness changes	(c)	(f)	(l)	(o)
Asymmetry	(d)	(g)	(m)	(p)
Interaction between structural members of different stiffness and between structures		(h)		(q)
Strong beams-weak columns		(i)		
Flat slabs		(j)		

Note: letters in ( ), see Fig. 2.1

Ductility Demands :

Ductility in reinforced concrete structures in general is defined as the ratio of a specified distortion at a particular stage of loading to that at the onset of yielding. Although ductility is not a unique measure of energy dissipation, it may be considered as a useful index that can measure the suitability of a structure in seismic areas.

Nonlinear dynamic analyses of code-designed multistor structures responding to typical severe earthquake ground motions have given an indication of the order of post-elastic deformations, and hence the "ductility factor" required. However, the number of variables involved in such analyses is so great that no more than qualitative statements concerning ductility demand can be made. For example, the type of ground motion has a considerable influence. Nevertheless some general conclusions can be drawn.

A measure of the ductility of a structure is the displacement ductility factor μ designated as

$$\mu = \Delta_u/\Delta_y \qquad (2.1)$$

where

Δ_u = maximum horizontal deflection of the structure, generally measured either at the top of the structure or at the point of action of the resultant horizontal seismic load.

Δ_y = horizontal deflection at that point of the structure when yield is first reached.

A number of dynamic analyses have indicated that the maximum horizontal deflections reached by a structure, which is not strong enough to resist the full elastic response inertia load and yields with elasto-plastic load-deflection characteristics, may be approximately the same as that of a structure which is strong enough to respond in the elastic range. This "equal maximum deflection" response is illustrated in Fig. 2.2(a). Dynamic analyses have also indicated that for structures with a short fundamental period of vibration, or for degrading stiffness systems, a better approximation is given by the "equal maximum potential energy" response illustrated in Fig. 2.2(b) which requires the area OCD to be equal the area OEFG.

As discussed earlier, the design horizontal seismic load in the equivalent static analysis procedure is significantly less than the elastic response inertia load. The ratio of elastic response inertia load to the design seismic load (OA/OB in Fig. 2.2(a) and (b) is referred to as "the behavior factor, K".

It is evident that the equal maximum deflection assumption of Fig. 2.2(a) means,

$$K = \mu \tag{2.2}$$

The equal maximum potential energy assumption of Fig. 2.2(b) may be shown |23| to mean that

$$K = \sqrt{2\mu - 1} \tag{2.3}$$

A comparison between the values of μ obtained from Eqs. (2.2) and (2.3) for the various values of K recommended in Section 5.3 (Table 5.3) are shown in Table 2.2.

Table 2.2 Relationship Between K and μ

$K = \dfrac{\text{Elastic Response Load}}{\text{Design Load}}$	2	3	3.5	4	5
μ from Eq. (2.2)	2	3	3.5	4	5
μ from Eq. (2.3)	2.5	5	6.6	8.5	13

The higher value for μ for each K value given by Eq. (2.3) is only expected to apply to short period or degrading stiffness systems. As would be expected, the required μ value for the structure is greatest for high K values, but of course the ductility demand can be reduced by using a lower K value.

The local ductility demand at a plastic hinge in a yielding structure may be expressed by the curvature ductility factor ϕ_u/ϕ_y, where ϕ_u is the maximum curvature (rotation per unit length) at the section and ϕ_y is the curvature at the section at first yield. It should be emphasized that the required curvature ductility factor ϕ_u/ϕ_y at plastic hinge sections will generally be much greater than the Δ_u/Δ_y value for the structure, since once yielding commences further displacement occurs mainly by rotation at the plastic hinges. This aspect of behavior in the yield range is discussed further below.

The exact characteristics of the earthquake ground motions that may occur at a given site cannot be predicted with certainty and the modelling of some aspects of the behavior of complete structures is still open to question. Hence it is impossible to evaluate all aspects of the complete behavior of a reinforced concrete building when subjected to very large seismic disturbances. Nevertheless it is possible to impart to the structure features that will ensure the most desirable behavior. In terms of damage, strength and ductility (including energy dissipation) this means ensuring a desirable sequence in reaching the strengths of the various modes of resistance of the structure. It implies a desired hierarchy in the failure modes of the structure. The rational procedure for achieving this aim in earthquake resistant design is first to choose the energy dissipating mechanism for the structure and to detail the chosen yielding regions for adequate strength and ductility. Then the remaining possible types of failure in the yielding regions and other parts of the structure are avoided by deliberately providing sufficient strength for them to withstand amplified design actions. The design for amplified actions is to ensure that the strength for that action is not reached before the chosen energy dissipating mechanism develops. This procedure will ensure (as far as possible) that yielding will occur only in the chosen manner during a severe earthquake.

In general, typical reinforced concrete members in which flexure is dominant ($N < 0.1 f_{ck} A_c$) exhibit a ductile behavior, because such a behavior is governed by steel rather than concrete (underreinforced beams). However, ductility is reduced by the presence of high shear or loss of anchorage. Therefore to ensure ductile behaviour, shear and anchorage failures should be prevented by taking the necessary precautions. Premature, brittle type of shear failure can be prevented by properly designed lateral reinforcement. Anchorage failures can be prevented by proper detailing, which will be discussed in Chapter 6.

Member behavior becomes less ductile as the level of the axial compression increases. Therefore structural members, such as columns, which are subjected to significant axial compression ($N > 0.10 f_{ck} A_c$) exhibit less ductile behavior as compared to beams. In many cases the desired ductility cannot be obtained unless special precautions are taken. The ductility of members can be greatly improved if concrete is confined by spirals or rectangular hoops.

Since in general, members with small or no axial compression, such as beams, are more ductile as compared to members with significant axial loads, such as columns, it is desirable to have plastic hinging in the beams rather than in columns when the structure has to go into the inelastic range during a severe earthquake. In most of the seismic codes this type of behaviour is ensured by having "strong columns - weak beams". Therefore at any beam-column joint the sum of the absolute values of flexural strengths of columns should not be less than the sum of the absolute values of flexural strengths of beams framing into that joint. This is illustrated in Figure 2.1(i). The beam and column moments shown in the figure are not design moments but are the capacities (ultimate) of members computed considering the cross-sectional dimensions, longitudinal steel and the axial load.

Since shear failure is much more brittle as compared to flexural failure in beams and columns,it is desirable to have the shear capacity to fully reflect the available flexural capacity. This becomes an essential requirement for structures built in highly seismic regions.

Inelastic dynamic analyses of single-degree-of-freedom systems have indicated high ductility demands for certain types of ground motion at low periods. Fig. 2.3 shows that, for the Bucharest type of ground motion, ductility demands are much higher than those for the well know El Centro type of excitation. Ductility demands for the Volvi '78 earthquake are rather small, but for the Montenegro (Ulcinj) '79 earthquake rather high |20|.

A close examination of ductility demands for single-degree-of-freedom systems in relation to the corresponding ground motion records reveals four basic ground motion input parameters that influence structural response:

a. Maximum acceleration,
b. Frequency content,
c. Strong ground-shaking duration
d. Number and duration of long-duration acceleration pulses

The destructiveness of the El Centro earthquake is mainly due to the long duration of strong ground shaking, while the destructiveness of the Bucharest earthquake is mainly due to a single pulse, which lasted for 1.7 sec.

On the other hand, the basic structural parameters that influence single-degree-of-freedom response are:

a. Normalized base shear strength ($\eta = C_y/\alpha_{max} = V_y/(G + \psi Q).\alpha_{max}$, where G = dead load and Q = live load)
b. Fundamental period
c. Type of hysteretic behavior.

If the overall displacement ductility demand for the single-degree-of-freedom system is known, curvature ductility demands can be determined with the help of the diagrams of Fig. 2.4. It can be seen that the curvature ductility demands (μ_b or μ_c) are usually equal to 3 to 7 times the overall displacement ductility demand (μ) for the case of beam sidesway mechanism and 20 to 40 times μ for the case of column sidesway mechanism.

<u>Example</u>:

Consider a structure with:

Normalized base shear strength	$\eta = C_y/\alpha_{max} = 0.5$
Fundamental period	$T = 0.5$ sec.
Number of stories	$n = 8$
Column/beam stiffness ratio	$\alpha = k_\upsilon/k_\delta = 1.0$
Column/column stiffness ratio	$\alpha = k_{\upsilon i}/k_\upsilon = 1.0$
Effective/plastic hinge length ratio	$\ell_\alpha = \ell_\upsilon/\ell_p = 10.0$

Earthquake	μ	Beam mechanism		Column mechanism
		μ_b	μ_c	μ_c
El Centro	5	17	12	100
Volvi	4	14	10	95
Bucharest	8	35	24	200
Ulcinj	8	35	24	200

The possibility of column mechanism formation should be eliminated through proper design.

2.3. Experimental Research

Structures as a whole, structural elements and subassemblages should exhibit satisfactory performance under anticipated loading. The performance criteria for reinforced concrete members and subassemblages can be summarized as,

i - Good serviceability under service loads (minimum crack width, minimum deformation, fire resistance and protection of reinforcement against corrosion).

ii - Under seismic excitation, ability to dissipate significant amount of energy through inelastic behavior under large amplitude cyclic deformations, without substantial reduction of strength (adequate ductility).

A great amount of experimental data has been presented during the last 10 years, throwing light into fundamental behavior of RC structural elements (SE) and subassemblages. Full scale testing to destruction of complete buildings is of course not feasible. Yet, parametric experimental testing of structural elements (SE) and subassemblages has furnished valuable information on; (a) confinement, (b) bond and anchorage, (c) splices, (d) structural elements, joints and subassemblages (shear and flexural), (e) construction joints and (f) infills. These will be discussed in the following paragraphs. It should be noted that only the information which is thought to be directly related to the objectives of this Manual is presented here. More detailed discussion can be found in references |9|, |19|.

(a) Confinement

Confinement can be achieved by transverse hoops (closed stirrups or ties) and spiral reinforcement. Confinement improves the ductility significantly (Fig. 2.5a). Adequate spirals and closely spaced rectangular hoops can also increase the strength by a small amount (Fig. 2.5a). Of course transverse reinforcement is also very effective in providing lateral support for longitudinal reinforcement in compression (Fig. 2.5b).

Due to geometry and continuity along the length, spiral reinforcement is very effective in providing the desired confinement. Since the behavior of rectangular hoops in providing the confinement is dominated mainly by flexure, effective confinement can only be provided at the corners. Also for rectangular hoops confinement is most effective at the level of transverse reinforcement, while in between there is reduced confinement effect (Fig. 2.5a).

Confinement increases the strain capacity significantly. The strain capacity depends on the configuration of transverse and longitudinal steel, yield strength and the volumetric percentage of the transverse steel.

(b) Bond and Anchorage

Bond is an essential requirement for good structural use of reinforced concrete. Typical bond stress-slip relationship obtained under monotonic loading is shown in Fig. 2.5(c). The main parameters which influence the bond are (under monotonic loading) :

- Geometry of bar surface (deformed or tension)
- Position and direction of bars
- Confinement
- Transverse compression of concrete
- Tensile strength of concrete
- Concrete cover and cover cracks

It should be noted that concrete cover also serves for two other purposes, protection of reinforcement from corrosion and increase of fire resistance.

Alternating cyclic loading of high intensity creates severe bond deterioration problems. Slip branch is evident in hysteretic bond-slip relation for alternating loading. Bond deterioration influences the strength of members and joints in an unfavourable way, but more important than this, stiffness decreases significantly due to bond deterioration.

Bond strength can be accomplished by providing adequate anchorage or development length. Therefore adequate anchorage is indispensable for good behavior of reinforced concrete structures under seismic action. The anchorage length is a function of steel stress, bar diameter and bond strength ($\ell_b = \frac{\sigma_s \phi}{4\tau_b}$). Anchorage can be provided by :

- Anchorage by a straight length (for deformed bars only)
- Anchorage by bars with 135^0 or 180^0 hooks (for plain bars)
- Anchorage by bars with 90^0 bents (for deformed bars)

The required straight anchorage length for reinforcing bars ranges between :

. 40 - 50Ø for S220 smooth, C20 ÷ C12 and monotonic loading
. 40 - 50Ø for S400 deformed, C20 ÷ C12 and monotonic loading
. 70 - 90Ø for S220 smooth, C20 ÷ C12 and alternating loading
. 60 - 90Ø for S400 deformed, C20 ÷ C12 and alternating loading

(c) Splices

Splices of reinforcement are inevitable in the construction of cast-in-place RC buildings. The problem of splices is essentially a problem of anchorage, so that the main parameters affecting the behavior of splices are the same as the main parameters affecting anchorage (that is, the parameters affecting bond).

The most common type of splices is lapped splices, which can be made with straight, hooked, or 90^0 bent bars.

Transverse tension is developed along lap spliced bars, which can result in cover cracking. Transverse tension from adjoining splices should not be additive, but it should be distributed along the axis of the element, by proper staggering of splices at suitable distances. Transverse tension effects can also be minimized by providing suitable transverse reinforcement.

The required lap length of bars in tension ranges between 1.0 and 2.0 times the required anchorage length depending on percentage of bars lapped at that section, concrete cover and clear distance between lapped bars.

(d) Structural elements, joints and subassemblages

Structural behavior of a RC section in pure flexure under monotonic loading is greatly influenced by the relative amount of reinforcement (Fig. 2.5(d)). Very small and very large reinforcement percentages result in brittle behavior, while relatively small percentages give satisfactory combinations of stiffness, strength and ductility.

Confinement, together with adequate compression reinforcement greatly increase the available ductility of the section, Fig. 2.5(e).

Shear can reduce the flexural capacity of a member when the shear span (a) is small ($a/d = M/Vd = 2\text{-}4$).

Axial load also significantly influences stiffness, strength and ductility of a member. Stiffness increases for an increase of the compressive axial load up to a certain value (0.3 to 0.5 N/N_o) and decreases for further increase of the axial load.

Flexural strength increases with an increase of the axial compressive load up to a certain value (0.2-0.4 N/N_o) and decreases for further increase of the axial load, Fig. 2.5(f). There are many design aids for estimating the strength of a RC section under M+N.

Ductility decreases drastically for an increase of the axial compressive load up to a certain value (0.2-0.4N/N_o) and remains practically constant for further increase of the axial load, Fig. 2.5(f).

Alternating loading causes a reduction of stiffness, strength and ductility of members and subassemblages. The reduction of stiffness and strength increases with the number of cycles, Fig. 2.5(g).

In members with low shear ($v \leq 0.25\sqrt{f_{ck}}$(MPa), $a/h = M/Vh > 7\text{-}8$) the reduction of stiffness and strength is rather small, while in members with high shear ($v > 0.50\sqrt{f_{ck}}$ (MPa), $a/h = M/Vh < 0\text{-}5$) the reduction is very significant, Fig. 2.5(h). Special detailing used in some test specimens have improved the behavior |9| as shown in Fig. 2.5(i).

The reduction of stiffness and strength due to alternating loading is greater in the case of biaxial eccentricity of the axial load.

In joints, alternating loading creates a problem of yield penetration and bond slip, which increases with the number of cycles.

Structural walls with low ℓ_w/h_w values exhibit a significant reduction of stiffness and strength under alternating loading, but their strength and ductility can be improved by the provision of adequate edge confined reinforcement or special boundary members.

Coupling beams usually have short shear spans ($a/h \simeq 2\text{-}4$) and during an earthquake experience a large number of intense loading cycles. If adequate shear reinforcement has been provided, good behavior can be expected, but if the shear reinforcement is insufficient, their behavior can be very poor. In certain cases use of diagonal reinforcement will improve the behavior of coupling beams |9|.

(e) Construction joints

Construction joints are inevitable in the construction of cast-in-place RC buildings and constitute a region of structural weakness, mainly in the case of structural walls.

Joint strength may be estimated from Eq. (2.5) |23| as:

$$V_{uR} = 1.0\rho_v f_{yk} \tag{2.5}$$

(f) Infills

Unreinforced brick masonry infill walls without connectors or seismic joints are used frequently in the Balkan region. Their contribution to overall structural behavior is usually ignored and the bare RC structural system is assumed to react to the seismic forces. Yet, the contribution of infills to overall structural behavior is very significant and should be taken into account during design.

During alternating loading, two behavior stages can be distinguished, Fig. 2.6(a):

i. Stage I : Uncracked frame-infill system
ii. Stage II : Cracked frame-infill system (infill separation)

Basic limit states of the system are, Fig. 2.6(b) :

i. Serviceability Limit State (SLS) :
  . Infill separation
  . RC admissible crack width

ii. Ultimate Limit State (ULS) :
  . Infill tension, diagonal cracking
  . Infill compression, corner crushing
  . RC member failure
  . Combined failure mode

Experimental results show a substantial increase of stiffness and strength of the structural system, due to the presence of the infill, even for the case of moderate quality of construction of the brick masonry, Fig. 2.6(c). Due to the existence of the infill, strength and energy dissipation capacity increase. The increase in stiffness is more significant. Tests have shown that increase in stiffness due to the infill can be 400% or more |21|.

The approximate equivalent strut method of analysis yields reliable results in predicting the composite stiffness of frame + infill systems.

2.4. In Situ Measurements

In situ measurements can be made before, during and after an earthquake. They aim at quantifying actions affecting the building and structural properties of members and of the structure as a whole.

A limited amount of existing data, for apartment houses and office buildings, for the Balkan region indicate that : (a) live loads are of the order of $0.5 \div 1.0$ kN/m^2 only, (b) environmental thermal effects are more severe than it is anticipated in design, causing SLS damage in buildings and (c) for the great majority of residental cast-in-situ buildings concrete quality ranges between C8-C16 (f_{ck} = 8-16 MPa, $f_{ck,mean}$ = 16-24 MPa) in some countries.

Ambient vibration measurements made have provided information on predominant vibration periods, damping and modes of vibration of buildings |21| .

Vibration measurements made on reinforced concrete buildings of 4-10 stories have indicated that for such buildings the initial fundamental period of vibration ranged between 0.3 and 1.2 seconds, depending on the slenderness of building, on the type of structural system (wall, frame, dual) and on the amount and quality of infill walls |21|. A limited number of tests made in Greece on residential buildings (4-8 stories, medium slenderness with $H/B < 4$, dual system with limited structural walls, with considerable amount of brick infills) have indicated that the initial period of bare system ranged between 0.6-1.0 seconds, the period of the infilled system ranged between 0.3-0.5 seconds and the final period of the infilled system (after a number of years) ranged between 0.4-0.8 seconds |21|.

Unfortunately, there are no strong motion response recordings for RC buildings in the Balkan region. A limited amount of data from the US indicates that the fundamental period of vibration of RC buildings during strong ground motions ranges between 1 to 3 times the initial ambient vibration period.

2.5. Pre-earthquake Surveys

Pre-earthquake surveys can yield valuable information concerning :

a. Prevailing building systems
b. Prevailing structural systems
c. Design state of practice
d. Construction state of practice
e. Undercapacity and overstress problems of existing buildings so that existing needs for the improvement of the situation will be identified.

Building and structural systems

The continuous building system (buildings in row, without seismic joints) used in town centers for cast-in-place RC residential buildings (Fig. 2.7), creates the problem of structure to structure interaction, with detrimental effects, especially for corner buildings. The problem becomes more severe when adjacent buildings have different dynamic characteristics. The importance of this major problem was dramatically demonstramed during the earthquakes of Vrancea (1977), Volvi (1978) and Alkyonides (1981).

Structural systems with severe overstress problems are presented in Fig. 2.8. Interior balconies, closed balconies, successive penthouses, indirect column and beam supports, infills and extremely bad overall configuration create severe overstress problems, which require special analysis and design methods.

Design practice

Design of private medium rise-medium cost cast-in-place RC residential buildings is usually based on oversimplified design, without due consideration of structure to structure interaction, of frame-infill interaction and of bad configuration problems.

Construction practice

Inadequate supervision (due to insufficient quality control specifications) and low workmanship can lead to severe undercapacity problems such as those illustrated in Fig. 2.9.

Undercapacity and overstress problems in existing buildings

Prevailing medium-to-severe undercapacity and overstress problems in medium rise-medium cost cast-in-place RC residential buildings are :

a. Infill masonry understrength	
b. Insufficient ties	(undercapacity)
c. Inadequate structural continuity	(problems)
d. Shear behavior	
e. Bad structural configuration (mainly joints)	
f. Bad story layout	(overstress)
g. Bad layout of building in plan	(problems)
h. Bad layout of building in elevation	
i. Structure to structure interaction	

Moreover, a number of buildings show SLS damage due to environmental thermal effects, as well as due to differential settlement. In both cases, cracking has generated a reinforcement corrosion problem.

2.6. Post-earthquake Surveys

Post-earthquake surveys can yield valuable information on types and frequencies of structural damage. Careful examination of structural characteristics of damaged buildings can reveal main causes of understrength and overstress, which led to damage. Great care should be exercised at this stage, because usually there are more than one reasons that interact to create the observed damage. Causes of undercapacity (poor concrete quality, insufficient ties, etc) are usually more easily detected than causes of overstress.

The most prevailing type of earthquake-induced damages in reinforced concrete buildings have occured in the following locations and members : (a) Columns (short columns where high shear stress are produced in addition to flexure), (b) stair slabs, (c) joints of linear elements, (d) slab-column joints in flat plate structures, (e) infill walls (separation between the infill and the frame and/or cracking of infill masonry) |22|. The last two types of damages, although of Serviceability Limit State level, proved to be of great economical importance.

In general, the causes of failure or damages have been :

a. Structural configuration
   - soft stories
   - torsion created by unsymmetrical arrangement of the core or other vertical lateral-load resisting elements

- flexible floors
- short columns formed either by connecting structural members or infill walls (for window openings).

b. Inadequate detailing
- Inadequate transverse reinforcement
- Lap splices at critical regions (at the face of joints)
- Lap splices with insufficient lap length and insufficient transverse reinforcement along the lap length.

c. Inadequate or unqualified quality control
- Steel not placed in accordance with the design drawings
- Poor quality of concrete

Detailed correlation of undercapacity and overstress effects versus damage is missing.

In most of the surveyed damage cases, a combination of more than one causes of undercapacity and of overstress seems responsible for the observed damage. Causes of understrength and overstress which led, by combination, to most of the damage cases seem to coincide with the causes of under-capacity and overstress which were identified by pre-earthquake surveys.

Causes of undercapacity and causes of overstress are estimated to have been of approximately the same importance in triggering earthquake damage.

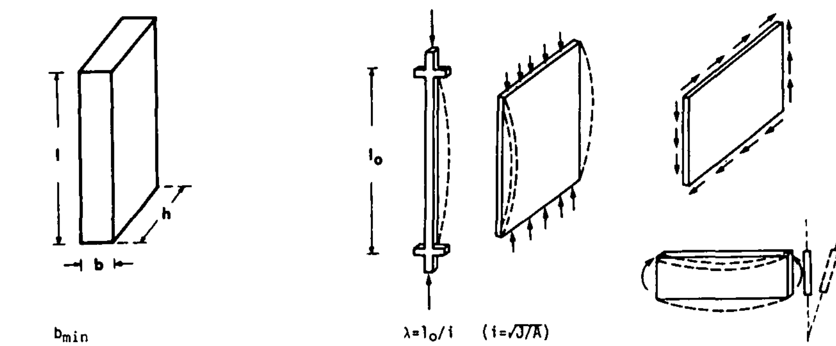
A number of typical structural damage cases is presented in Figs. 2.10 through 2.15.

2.7. Site Effects

Seismic wave motions attenuate as they emanate outward from the epicenter of the earthquake. The high frequency components attenuate more rapidly than the lower frequencies. Sites near active faults can thus be expected to have high amplitude motions with considerable high frequency content. Conversely sites at some distance from an active fault can be expected to have less intense motion made up primarily of lower frequency components.

Seismic waves are transmitted through the base rock and then up through the overlying soil. Certain soil characteristics, such as the seismic shear wave velocity, appear to modify the waves as they pass upward to the surface. Softer soils tend to amplify the lower frequency components of motion while firmer soils amplify the higher frequency motions.

Local soil conditions should be considered in the seismic design of reinforced concrete structures. The effect of soil conditions on the structural response can be established based on soil profile types. In the codes, soil profile types and related site coefficients S are specified which are used to modify the standard elastic response spectrum to acount for the site conditions.

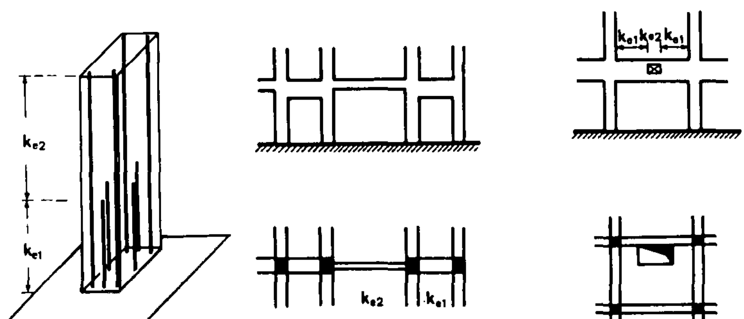


b_{min}

(a) Small section dimensions

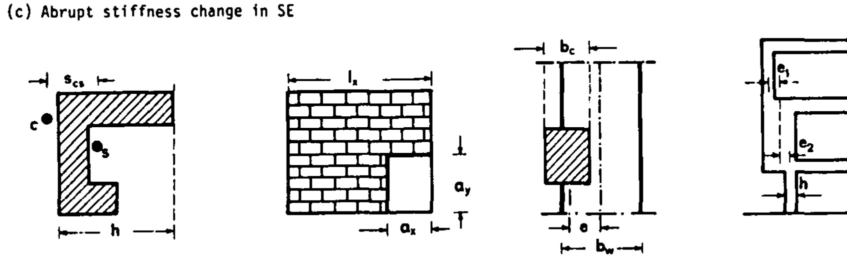
$\lambda = l_o/i \quad (i=\sqrt{J/A})$

(b) Slenderness in SE



$\delta ke=(ke1-ke2)/ke1$

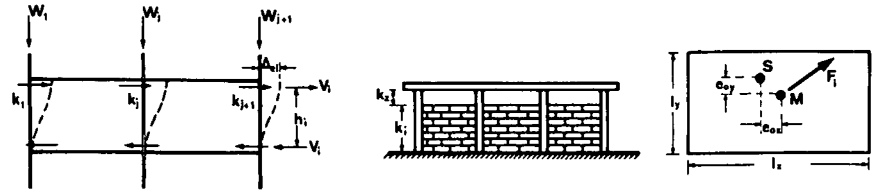
(c) Abrupt stiffness change in SE



$s_{cs} \ngtr 0,15h$ $\quad a \ngtr 0,20l$ $\quad e \ngtr b_c/4$ $\quad e_i \ngtr h/10$

(d) Asymmetry in structural element

Fig. 2.1 Basic overstress parameters |21|



$1/\lambda_s = W_i/(\sum_j k_{i,j}\cdot h_i)$

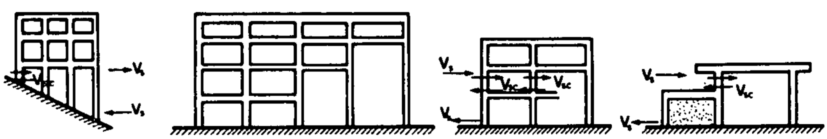
(e) Story slenderness

$\delta ks = (k_1 - k_2)/k_2$

(f) Abrupt story stiffness change

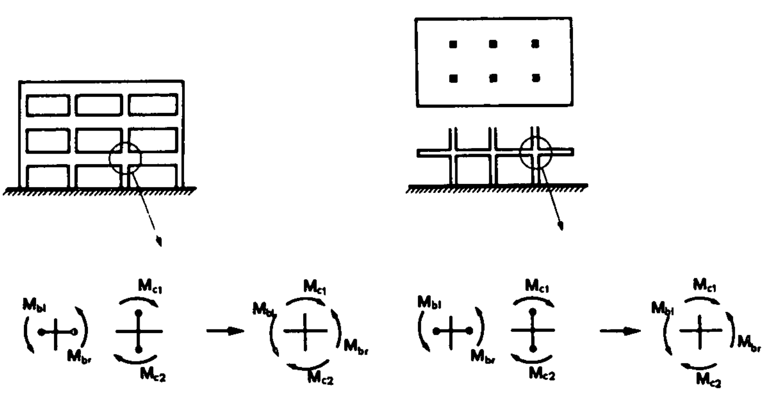
(S: stiffness center)
(M: mass center)
e_o

(g)Story asymmetry



$\sum V_{sc}/V_s$

(h) Interaction between SE of different stiffness



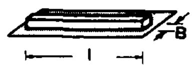
$\sum M_c/\sum M_b = \sum(M_{c1}+M_{c2})/\sum(M_{b1}+M_{br}) > 1$

(i) Strong beams-weak columns

$\sum M_c/\sum M_b = \sum(M_{c1}+M_{c2})/\sum(M_{b1}+M_{br})$

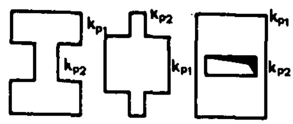
(j) Flat slabs

Fig. 2.1 Basic overstress parameters (continued) |21|



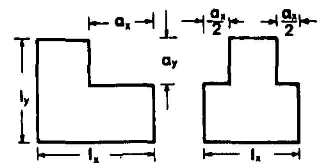
l and l/B

(k) Slenderness
in building plan



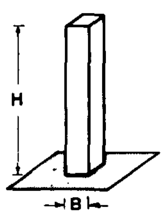
δkp=(kp1-kp2)/kp1

(l) Abrupt stiffness change
in building plan



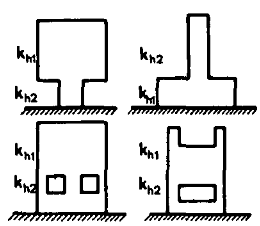
a/l

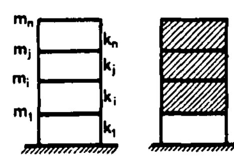
(m) Asymmetry in
building plan



H or H/B or T

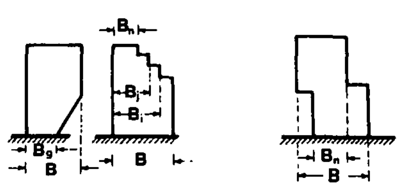
(n) Slenderness in
building elevation





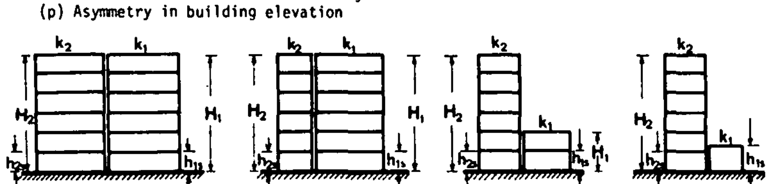
δkh=(kh1-kh2)/kh1 or δkh=(kh1/m1-kh2/m2)/(kh1/m1)

(o) Abrupt stiffness change
in building elevation



B_g/B B_n/B $(B_i-B_j)/B$ B_n/B $(B_i-B_j)/B$

(p) Asymmetry in building elevation



H_2/H_1, h_{2s}/h_{1s}, k_2/k_1

(q) Structure to structure interaction

Fig. 2.1 Basic overstress parameters (continued) 21

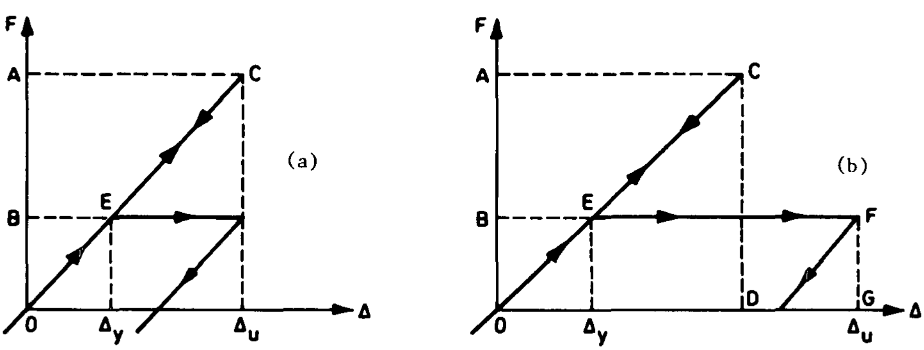
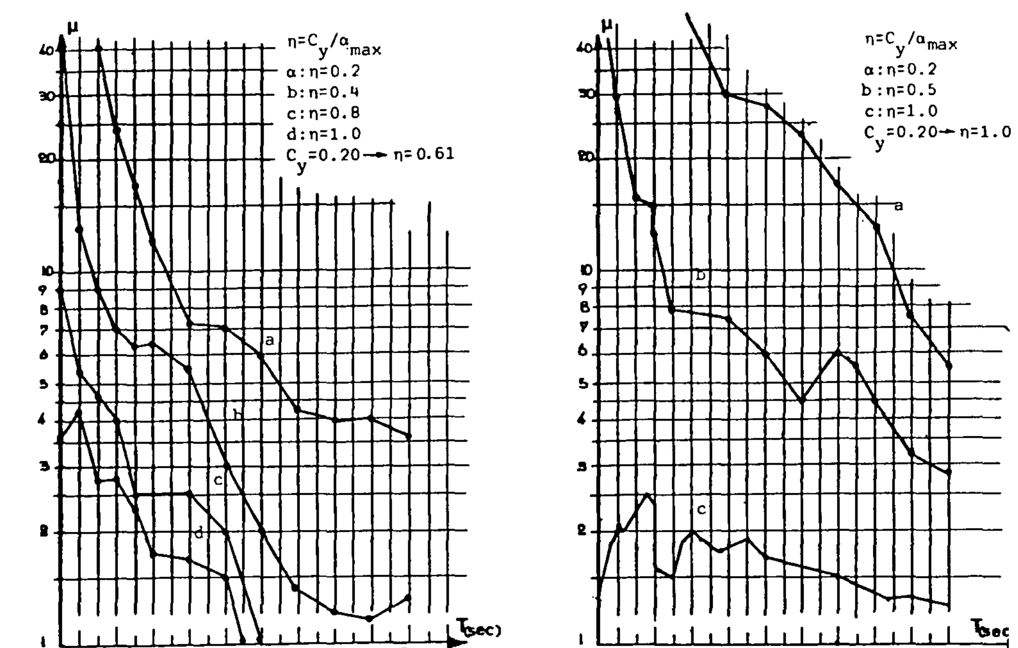


Fig. 2.2 Assumed Responses of Elastic and Elasto-Plastic Systems to Earthquake Ground Shaking



(a) El Centro, 18.05.40, N-S, α_{max}=0,330g
β=5% , EP

(b) Bucharest, 04.03.77, N-S,
α_{max}=0.20g, β=5% , EP

Fig. 2.3 Ductility demand (μ)-Normalized base shear strength (η)-
Fundamental period (T) relation for SDOF systems |21|

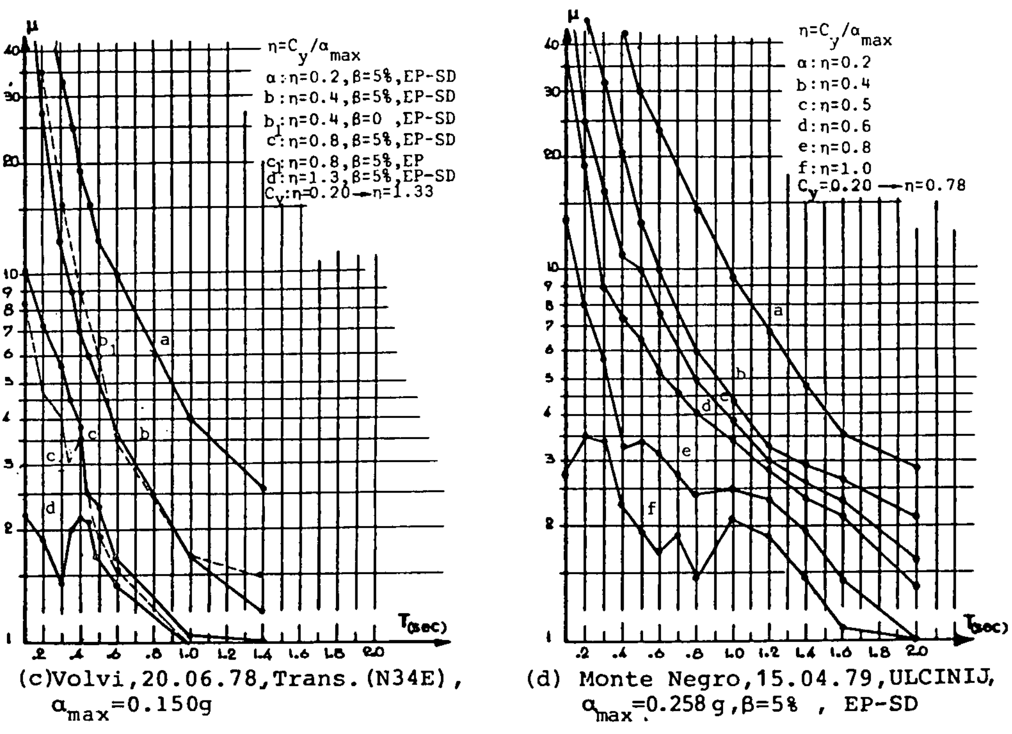
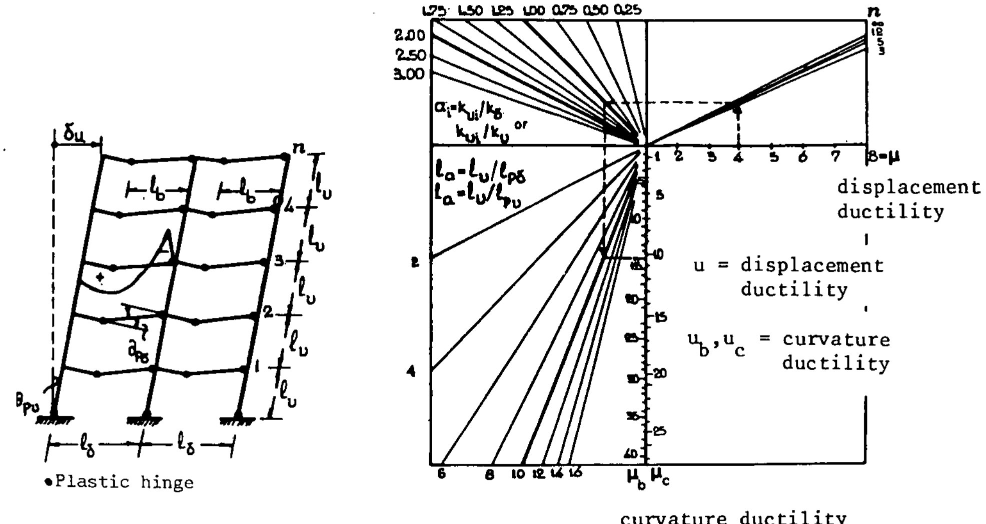
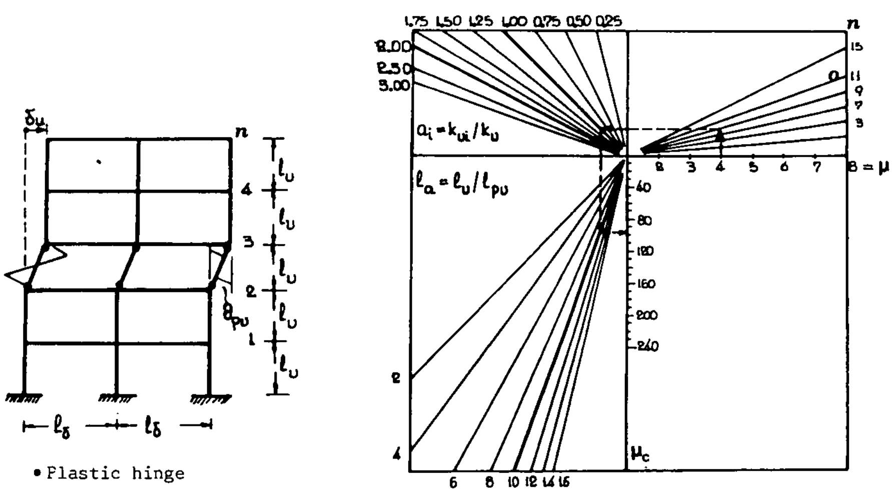


Fig. 2.3 Ductility demand (μ)- Normalized base shear strength (η)-
Fundamental period (T) relation for SDOF systems (continued) |21|

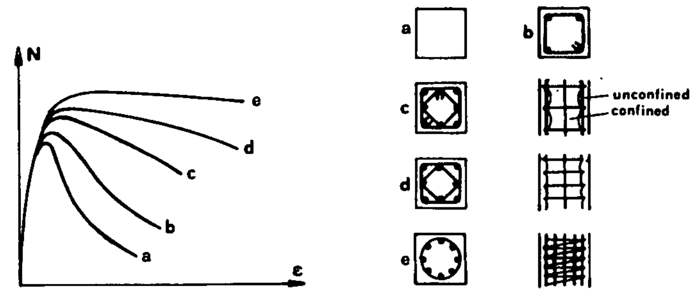


(a) Beam mechanism

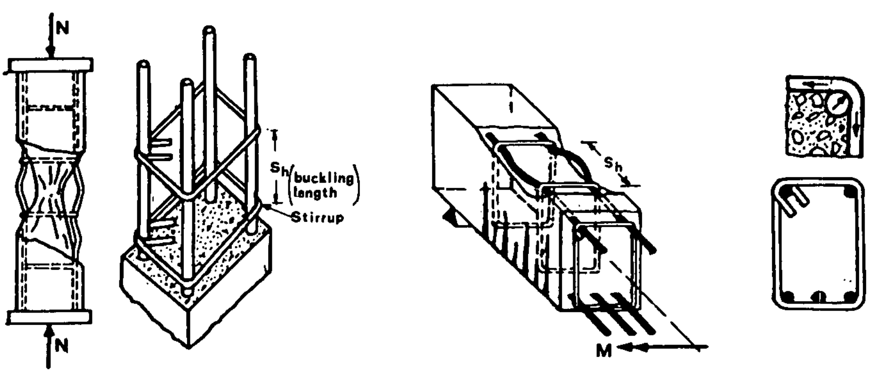


(b) Column mechanism

Fig. 2.4 Overall displacement (μ) and beam (u_b) and column (u_c) curvature ductility relationship based on static sidesway mechanisms

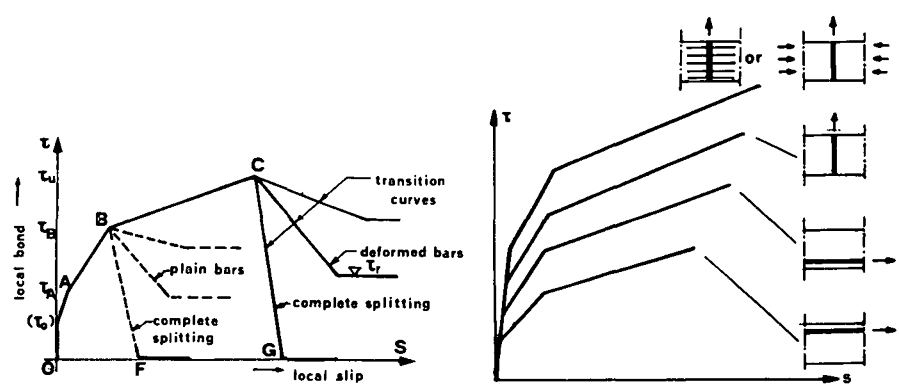


(α) Confinement effect



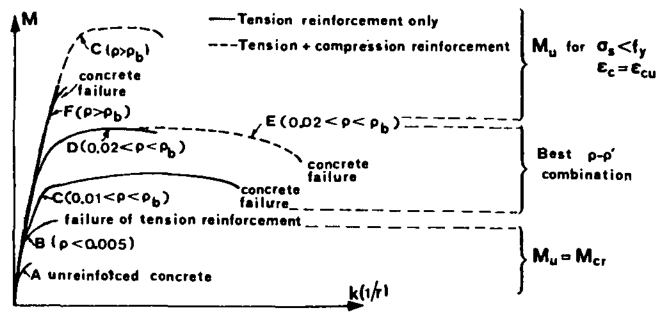
Failure mode Stirrups Failure mode Stirrups

(b) Lateral support of compression reinforcing bars for the cases of axial loading (N) and flexure (M)

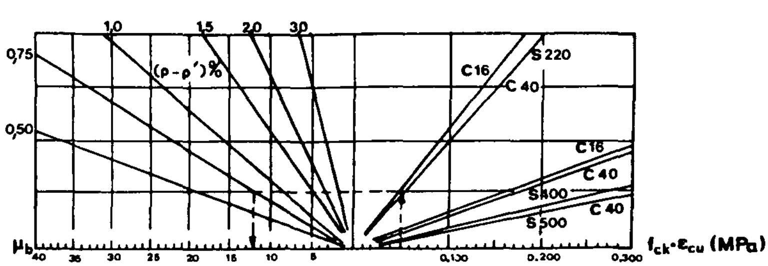


(c) Bond-slip relation for monotonic loading

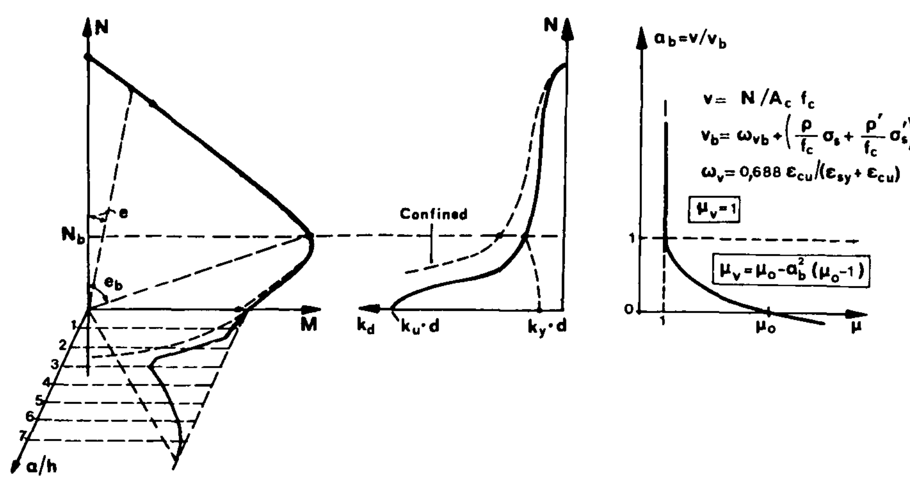
Fig. 2.5 Structural elements, joints and subassemblages



(d) Influence of reinforcement on M-k relation for monotonic loading

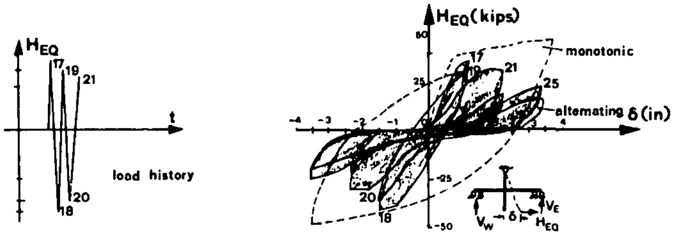


(e) Curvature ductility for monotonic pure flexure |22|

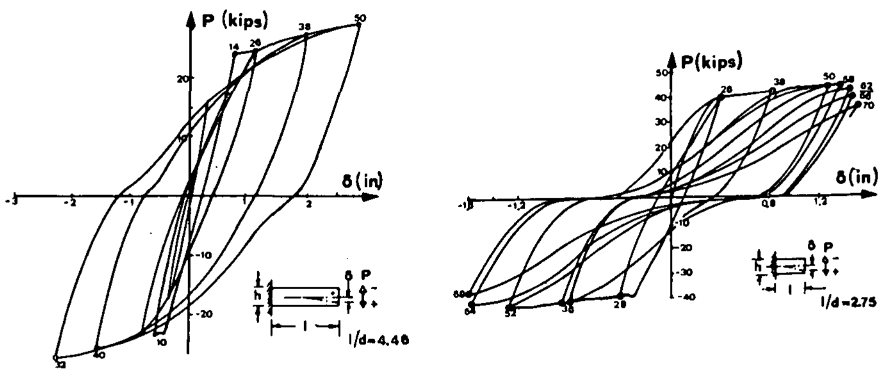


(f) Effect of axial load (N) on strength and ductility and of shear span ($\alpha/h = M/Vh$) on strength

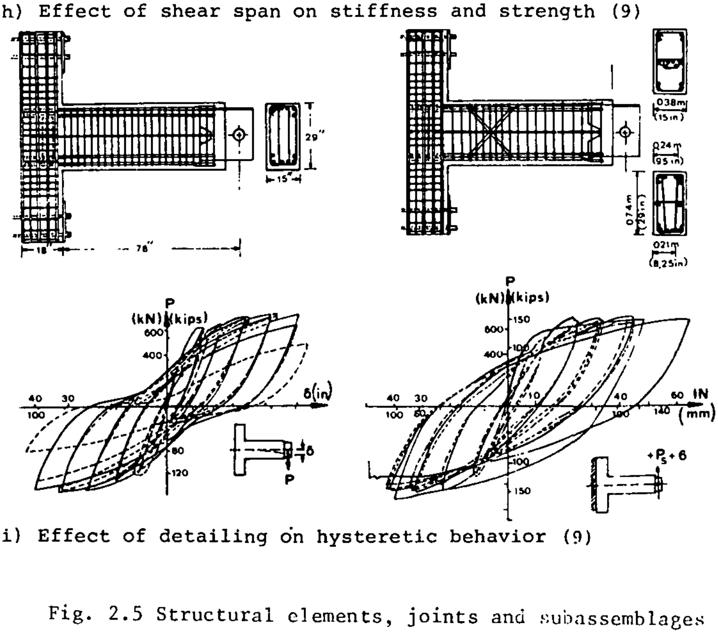
Fig. 2.5 Structural elements, joints and subassemblages (continued)



(g) Effect of alternating loading on stiffness and strength (9)

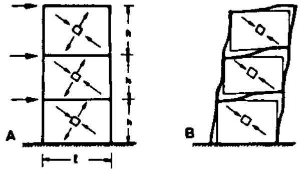


(h) Effect of shear span on stiffness and strength (9)



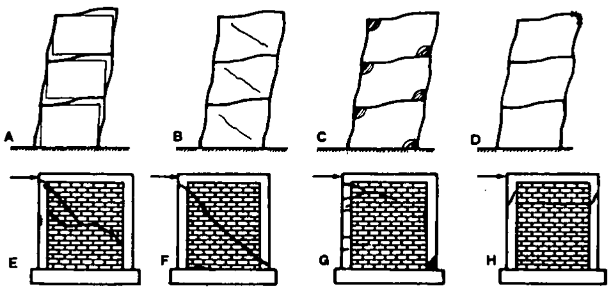
(i) Effect of detailing on hysteretic behavior (9)

Fig. 2.5 Structural elements, joints and subassemblages (continued)



A:Stage I :Uncracked
B: " II:Cracked

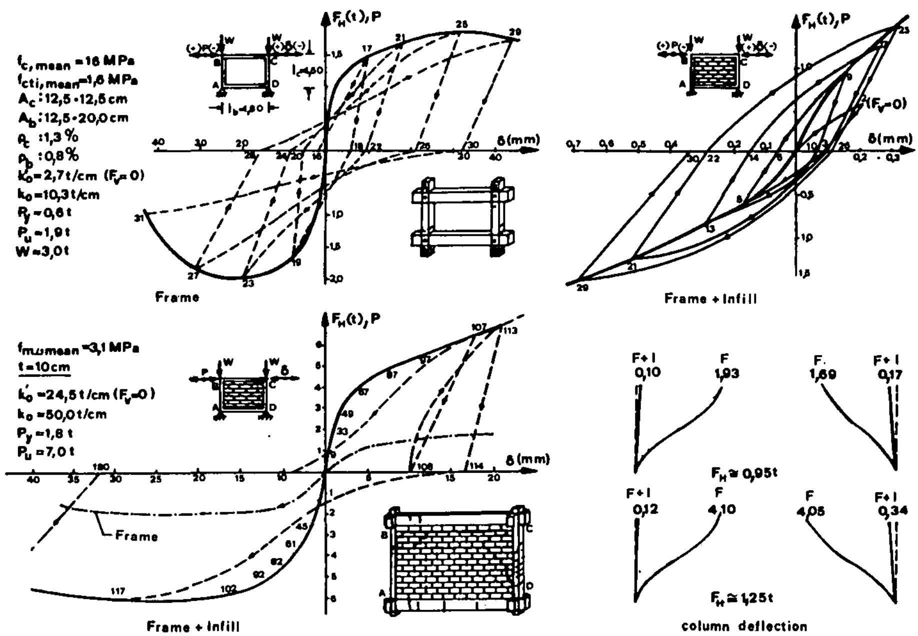
(a) Behavior stages



SLS
A:Infill separation

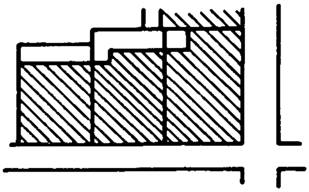
ULS
B:Infill cracking
C: " crushing
D:RC SE failure
E:Shear-Strong frame
F: " -weak "
G:Flexure
H:Slip

(b) Limit states



(c) Comparison of bare and infill frame behavior for alternating loading

Fig. 2.6 Infill walls |22|

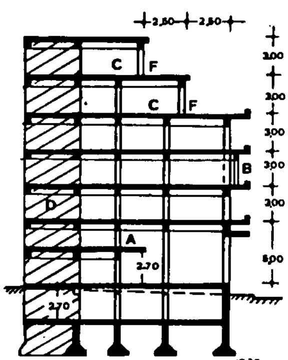


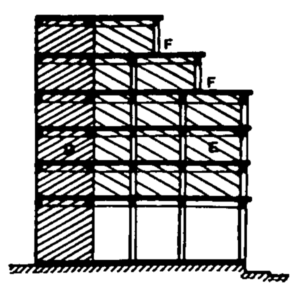
a) plan

b) view of typical case

c) view of severe case

Fig. 2.7 Continuous building system. Structure to Structure interaction





A:Interior balcony
B:Closed balcony
C:Successive penthouses
D:Structural wall
F:Column indirect support

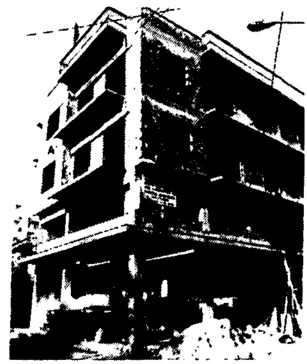
(a) Residential building with stores in the ground floor

D:Structural wall
E:Infill
F:Column indirect support

(b) Residential building with pilotis

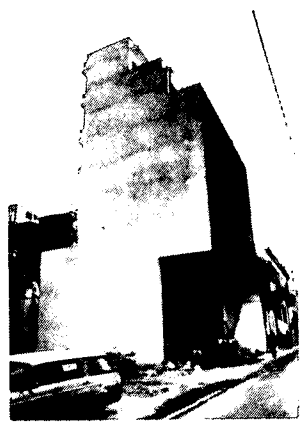
Fig. 2.8 Structural systems with severe overstress problems

(c) Interaction of SE with different stiffnesses due to interior balcony



(d) Closed balcony

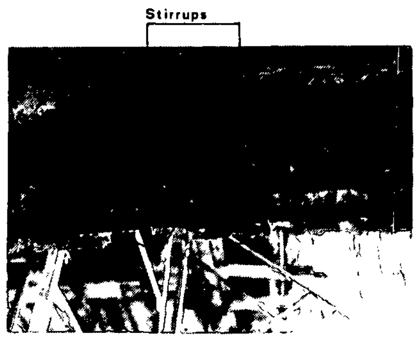
(e) Penthouses (A) indirectly supported columns (B)



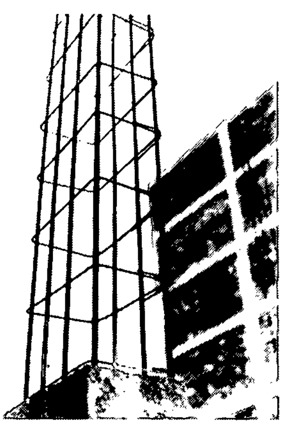
(f) Extremely bad over-all configuration

Fig. 2.8 Structural systems with severe overstress problems (con'd)

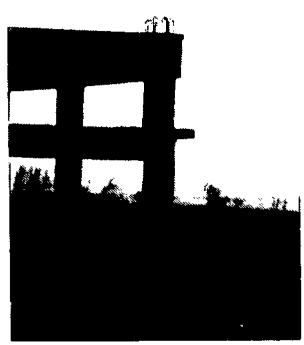
(a) Very poor concrete at upper end of column



(b) No concrete cover + improperly placed stirrups



(c) Insufficient, improperly anchored stirrups

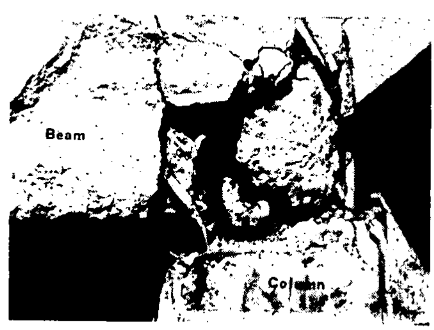


(d) Insufficient splices in construction joint

Fig. 2.9 Examples of RC buildings with severe undercapacity and overstress problems

(a) Insufficient reinforcement. Typical steel tension failure.

(b) Inadequate splicing



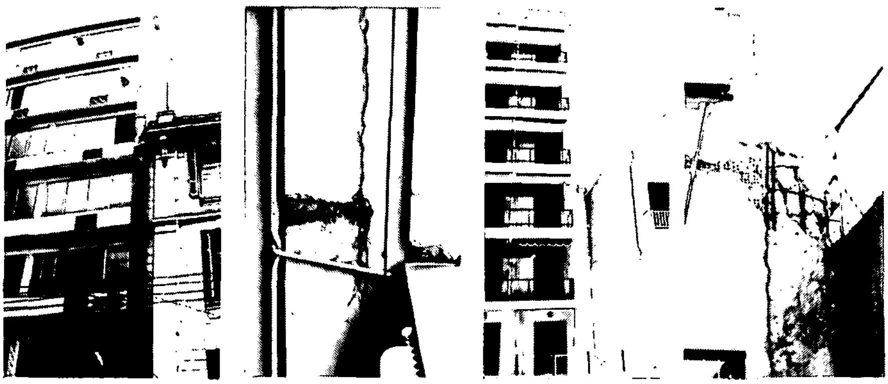
(c) Insufficient anchorage and lack of ties in the joint area

Fig. 2.10 Causes of undercapacity |22|

(d) Insufficient ties and insufficient anchorage length

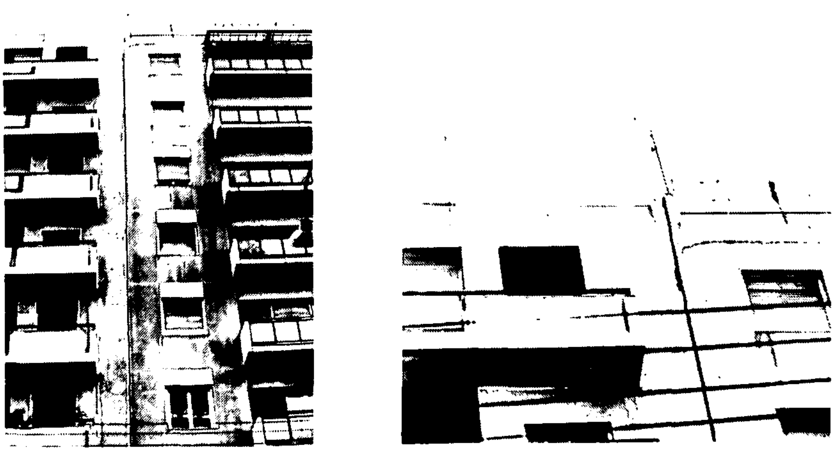
(e) Insufficient and improper structural joint reinforcement

Fig. 2.10 Causes of undercapacity (continued) |22|



Exterior view   Interior view   General view   Detail

(a) Structures with different heights



(b) Structures of the same height

Fig. 2.11 Causes of overstress. Structure to structure interaction |22|

(c) Interaction due to insufficient joint

Fig. 2.11 Causes of overstress. Structure to structure interaction |22| (continued)

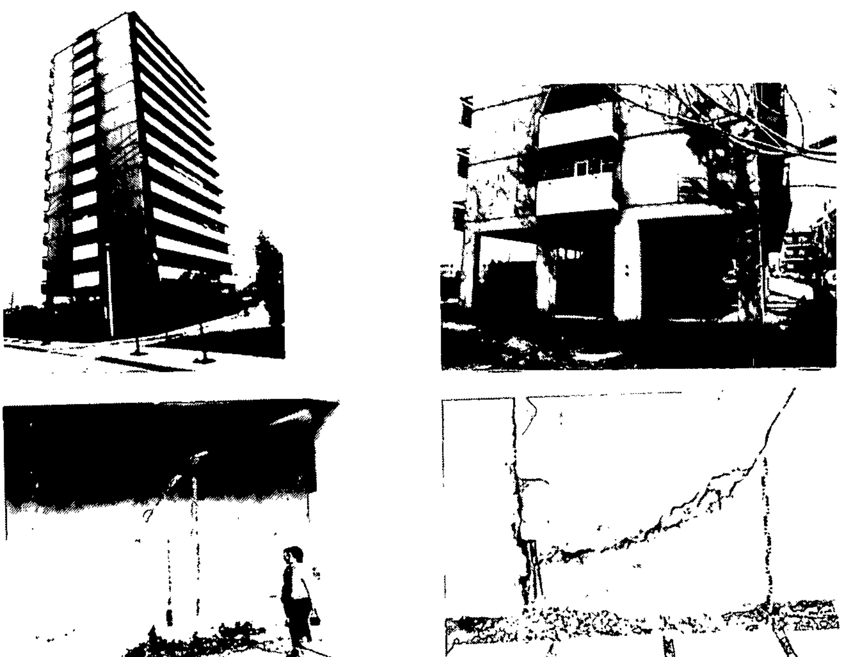
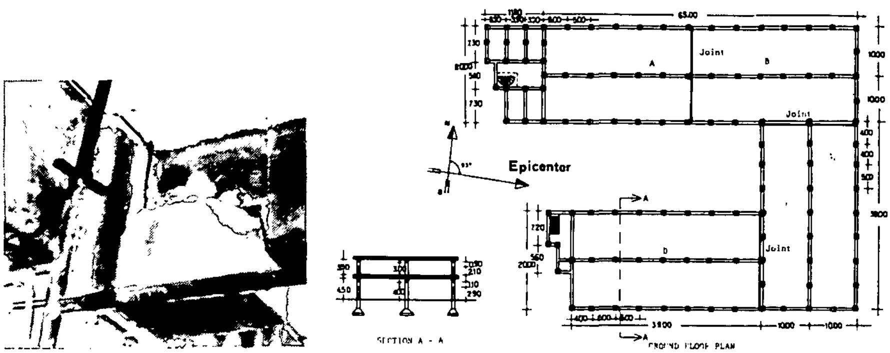


Fig. 2.12 Causes of overstress. Abrupt stiffness change at ground floor level. Damage of main structural wall |22|.

(a) Successive penthouses

(b) Building in sloping ground

Fig.2.13 Causes of overstress. Problems due to torsional motion |22|



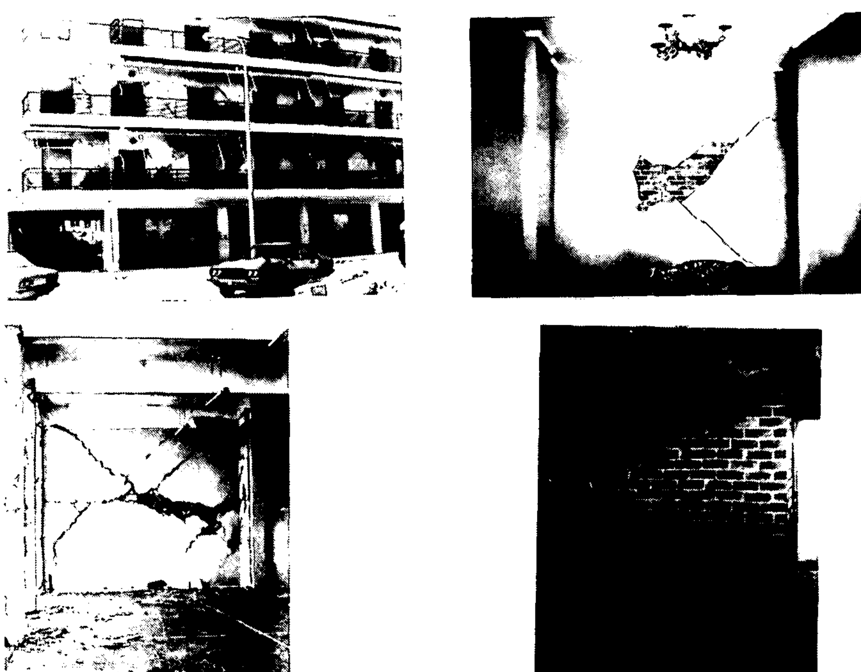
(a) Bad story layout

No damage of interior columns. Heavy damage of all columns in the perimeter. No damage of corner columns

(b) Short columns due to strong infill masonry

Fig. 2.14 Causes of overstress. Bad story layout. Interaction of vertical structural elements of different stiffness |22|

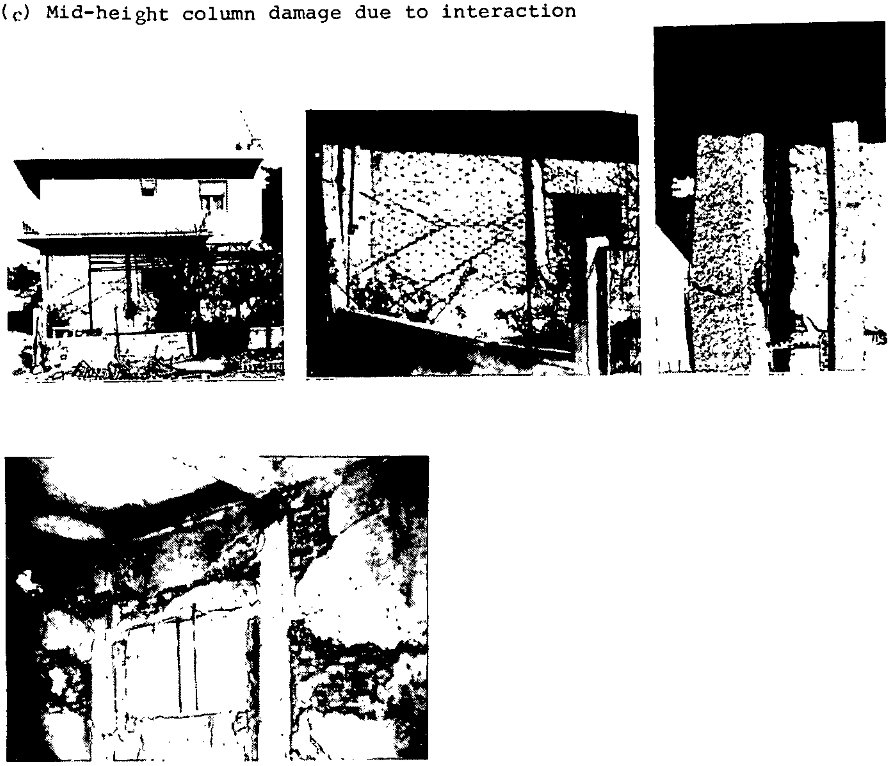
(a) Separation



(b) Infill cracking

Fig.2.15 RC frame-infill interaction |22|

(c) Mid-height column damage due to interaction



(d) In weak frame systems, masonry cracking crosses the columns too

Fig. 2.15 RC frame-infill interaction (continued) |22|

3 - CLASSIFICATION OF STRUCTURES

3.1. General Classification

The provisions of this manual have been prepared with reference to the following three types of structural systems :

i) Frame System

A system in which both vertical and lateral loads are resisted by space frames.

ii) Wall System

A system in which both vertical and lateral loads are resisted by vertical structural walls, either single or coupled.

iii) Dual System

A system in which vertical loads are essentially carried by space frames. Resistance to lateral forces is contributed partly by the frame system and partly by structural walls.

As illustrated ideally in Fig. 3.1, frame systems with stiff beams, under action of forces generated by earthquake motions, tend to deflect as "shear beams", while wall systems tend to deflect as "flexure beams" or cantilevers. The deflected shape of a dual system depends on the relative stiffness of its wall and frame components.

3.2 Categories of Seismic Performance

Generally, it is not economical to design and detail all buildings for the same risk level. Risk level is related to proportioning and detailing requirements of members and joints as well as to other factors. For example, requirements for proportioning and detailing of buildings with high occupancy, located in highly seismic zones should be more severe than for low-rise residential buildings in zones of low seismicity.

Proportioning and detailing requirements affect both ductility and stiffness of members and structure.

Categories:

In this manual buildings have been classified into three seismic performance categories A, B and C, according to the minimum requirements specified for proportioning and detailing. Most severe requirements have been specified for category C.

The requirements specified for each category can also be related to the ductility level provided, with category C having the highest ductility level.

Minimum proportioning and detailing requirements for each category (with the exception of category A) are given in Part 1 of this manual. Requirements for category A are not specified here, because these in general will be covered in nonseismic codes. Basic requirements for each category are summarized below.

Seismic Performance Category A

Structures in this category should be designed mainly in accordance with the nonseismic codes. Only a few additional detailing requirements can be specified.

Seismic Performance Category B

For structures in this category, specific aseismic provisions have to be adopted, enabling the structure to enter the inelastic range of response under repeated, reversed loading, while avoiding premature brittle type of failures.

Seismic Performance Category C

For structures in this category, special requirements for the design, proportioning and detailing should be adopted to ensure the development of selected mechanisms associated with large-energy dissipation capacities.

Buildings should be classified into these three categories, considering the following criteria :

a. Socio-economic importance
b. Seismicity of the region
c. Configuration.

a. Socio-economic importance :

Buildings are assigned to one of the following importance groups depending on the type of occupancy and importance of its serviceability after a major earthquake. Each group is assigned a different Importance Index (I).

Group III - This group includes buildings having essential facilities necessary for post-earthquake recovery. Examples are fire stations, medical centers, power stations, etc.

Group II - In this group, buildings having a large number of occupants or buildings in which occupants' movements are restricted, are included. Examples are public assembly buildings, schools, offices, hotels and apartment houses over 6 stories in height, retail stores with 500 m^2 floor area per level or more than 4 stories in height.

Group I - This group includes all other buildings not included in groups III or II.

b. Seismicity of the region :

The ground motion is usually defined in terms of Effective Peak Acceleration or Effective Peak Velocity-Related Acceleration (A_a and A_v). The Seismicity index is related to the effective Peak Velocity-Related Acceleration coefficient or intensity. Three or four Seismicity Index (SI) numbers can be assigned for the buildings considering acceleration or intensity.

c. Configuration :

Buildings can be classified as "regular" and "irregular buildings" considering both plan and vertical configuration. Buildings which have an approximately symmetrical geometric configuration and which have the centroids of building mass and lateral-force resisting system nearly coincident should be classified as regular.

3.3. Assigning Buildings to Seismic Performance Categories

Buildings in each country may be classified considering the three criteria discussed above. Each of the three seismic performance categories corresponds to a certain ductility level since proportioning and detailing in each category is done accordingly. Therefore A, B and C can also be called Ductility Level 1, 2 and 3. Seismic performance category C (or Ductility Level 3) is assigned to provide the highest level of design performance criteria.

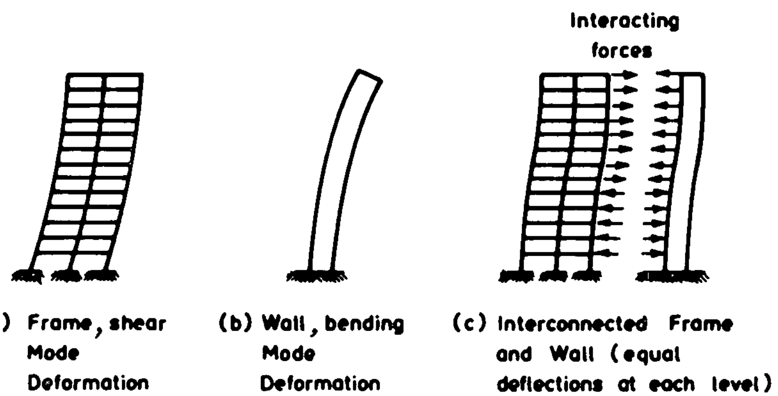


Fig. 3.1 Deflected Shapes and Interaction of Frames and Walls

4 - DESIGN OBJECTIVES

4.1. General

Buildings designed as earthquake-resistant structures should be able to resist frequent, minor earthquakes without any significant damage in the nonstructural components. Such structures should resist the moderate earthquakes without significant structural damage. In case of severe seismic action, the structure should be able to resist the earthquake loading without major failure of the structural system to maintain life and to minimize major economical and cultural losses. For the satisfactory behavior of the structure in the inelastic range, the members and connections should have adequate ductility and energy dissipation capacity.

However, the designer should not solely base his design on ensuring ductile behavior. In many cases the nonstructural damage due to deformations in the inelastic range can be beyond economical limits. Therefore limits should be placed on the lateral displacements to prevent great economical losses and overall structural instability problems.

4.2. Design Requirements

The aim of seismic design is to ensure acceptable small probabilities of exceeding the following Limit States for the expected seismic actions :

a. Limit State of Serviceability (LSS)
(Minimum nonstructural damage)

b. Limit State of Structural Damage (LSSD)
(Limited degree and extend of structural damage)

c. Limit State of Collapse (LSC)
(Avoiding collapse and providing adequate residual capacity for the structure and its components).

Since direct analytical verification of the above requirements is extremely difficult, the following design requirements are set as substitutes of the original design targets :

Stiffness requirements

It was pointed out that excessive deformation in a structure could cause considerable damage to both structural and nonstructural elements. Therefore, in seismic codes, drift limits are specified to minimize the damage. The lateral displacement and the interstory drift depend mainly on the characteristics of the ground motion and the stiffness of the structure. Nonstructural elements such as infill walls influence the stiffness and thus the lateral displacement significantly.

In building codes, drift requirements are specified by limiting the interstory drift. The admissible values of drift parameters depend on the type of nonstructural elements and equipment installed, and on the connections.

Recently some engineers and researchers have considered the drift requirements as the most important criterion in the seismic design of reinforced concrete buildings. Observed behavior of relatively slender multistory R.C. systems suggests that for such systems drift might be the pivotal criterion for earthquake resistant design |19|.

In some types of structures overall displacement at the top of the building should also be checked.

Strength requirements :

Reliability for the Limit State of Structural Damage is ensured by providing critical regions of the structure with adequate strength for the expected design seismic action effects.

Ductility requirements :

Reliability for the Limit State of Collapse is ensured if ductility requirements are satisfied in addition to the strength requirements.

It should be noted that the designer has no reliable means to compute the system ductility of the structure. What is available is the detailing rules arrived at through the study of experimental behavior of members and structures. The designer should be fully aware of such data.

5 - DESIGN CRITERIA

5.1. Introduction

The following items should be considered in producing satisfactory earthquake-resistant structures :

a - Selection of proper site and foundation

In selecting the building site, locations with low seismic activity should be preferred. It is also essential that the site selected should not be close to surface faulting, rockfalls and sliding.

Foundation of good configuration, with adequate stiffness, strength and ductility should be selected.

b - Selection of appropriate structural system with good structural configuration (building, structural system and structural members). Structure to structure interaction should be eliminated whenever possible.

c - Appropriate estimate of basic seismic input and reliable analysis (modeling and analysis of the structural system) is important.

d - Capacity design of the structural system.

e - Adequate stiffness of the structural system (limiting interstory drift).

f - Good detailing of the reinforcement .

g - Good quality control (including control of material quality and construction quality).

Items (a) and (d) were discussed in Chapter 2. Item (f) will be discussed in Chapter 6 and (g) in Chapter 7 (item f has also been discussed in Part 1 of this manual). The rest, i.e. items (b), (c), (e) will be discussed in this chapter.

5.2. Structural Configuration

The term "structural configuration" as used in this manual refers to the geometry of the structure and the type of structural system (such as wall or frame) and elements used. In relation to earthquake effects, non-structural items also affect structural response and are therefore considered to be parts of the structural configuration.

Structural configuration is one of the most decisive parameters of seismic behavior. Bad configuration is likely to lead to severe overstress problems. In design practice of some countries, the architect generally conceives and controls the building configuration. Since configuration is one of the most important influences on the seismic behavior, the structural engineer should cooperate with the architect at the very early stage of design to evaluate alternative configurations.

A number of typical examples of bad and good configurations and possible solutions are given in Fig. 5.1. Cases presented in the column marked as "NO" should be avoided, unless covered by special analysis, design and construction considerations.

Cases shown in Fig. 5.1 are to illustrate mistakes made in choosing the configuration. In some cases the number of rows of structural columns are less than what would be recommended (for example two rows). Such cases were presented due to restrictions in space and are by no means encouraged.

5.3 Design Seismic Actions (Modeling and Analysis)

5.3.1 Seismic Action :

Seismic action to be employed in the design is related to the seismic activity of the region which can be conveniently described by means of seismic risk maps. The seismic risk maps show the distribution of the magnitude of the parameters defining the intensity of the seismic event for a given return period or for given annual probabilities.

The intensity of seismic event can be expressed either by means of a phenomenological scale, such as MSK scale, or by a set of parameters which describe the ground motion, such as "effective peak acceleration" or "effective peak velocity-related acceleration".

Seismic risk maps are prepared considering historical records, if available and reliable, and geological and seismotectonic data.

Ground motion at a site can be defined by specifying its normalized frequency content and duration, in addition to scaling the intensity parameter. For a known or given constant duration, the frequency content can be expressed either through "power density spectrum" or the "elastic response spectrum" relative to any selected value of damping. Both spectrums can be normalized to give a peak ground acceleration of 1g.

5.3.2. Methods of Analysis

Reliable modeling and analysis are helpful towards a good design of structures and structural members to resist seismic action. The designer should realize the errors introduced due to approximations and simplifications made in structural modeling. In modeling and analysis, the effect of masonry infills should be taken into account. Soil-structure interaction should be considered, especially in the case of structural walls with foundations on flexible soils.

Masses to be considered in the determination of the seismic effects will be those of all gravity loads existing at the time of the seismic event. The fundamental combination of load effects to be used is :

$$G + P + E + \Sigma\psi_i Q_{ik} \qquad (5.1)$$

G - includes all permanent loads (nominal)

P - is the prestressing force multiplied by specific factors given in codes (usually 1.0)

E - is the design seismic action

Q_{ik} - are the fractile values of extreme distributions of all variable loads

ψ_i - factors required to pass from the fractile values Q_{ik} to the average values Q_i of their instantanious distribution.

For the analysis of structures under seismic action, different methods can be used. The most commonly used methods are :

a. Equivalent static analysis
b. Modal analysis
c. Time history analysis

The "equivalent static analysis" procedure can be used if the building can be classified as "regular". In this procedure, the actual dynamic (inertia) loads induced in the structure when responding to severe ground shaking are represented by equivalent static loads found from a design response spectrum which is a suitably modified elastic response spectrum. The distribution of horizontal seismic actions assumed,essentially follows that of the first mode of vibration.

The "modal analysis"procedure should be used for more irregular structures or for tall buildings where the equivalent static analysis procedure does not apply.

The procedure uses dynamic analysis, assuming elastic behavior, to determine the inertia forces acting at each floor level. The modal responses are computed using the same design response spectrum as for the equivalent static analysis procedure. The building is modelled as a system of masses lumped at each floor level. The inertia forces at each floor level, separately obtained from each mode of vibration, can be combined by taking the square root of the sum of the squares of the modal values. The forces so found are then reduced to take into account the ductility of the structure. The resulting design forces are then applied to the structure to obtain the internal forces. The effects of torsion due to irregular structural configuration are also included.

In important cases of some unusual structures, or as a research tool, "time history linear or nonlinear dynamic analysis" may be justified. In this type of analysis, the response of the structure to a particular accelerogram record from a severe earthquake is computed by numerical integration for each small time step of the earthquake record. Idealizations for the inelastic internal force -deformation characteristics of the elements of the structure are utilized. For example, idealized moment-curvature hysteretic loops would be necessary for flexural members. Time-history analyses enable both the overall and the local ductility demands to be assessed. However, the results of such analyses are very much dependent on the earthquake record chosen and the structural idealizations assumed. For time history analysis it is necessary to determine seismic parameters such as intensity and frequency content for design earthquake and maximum expected earthquake using results from site investigations.

Only the simplest of the three methods, "equivalent static analysis" will be discussed in more detail in the following paragraphs.

5.3.3. Equivalent Static Analysis :

This type of analysis should be restricted to regular buildings provided that their height does not exceed 80 m. and the fundamental period does not exceed 2 seconds.

In this type of analysis, equivalent lateral forces to be applied at each floor level, in the direction being analyzed, should be determined. The formula recommended by CEB will here be considered |13| .

$$F_i = C_d \gamma_i W_i \tag{5.2}$$

C_d - design seismic coefficient, as defined in Eq. (5.4)

γ_i - distribution factor depending on the height of the floor measured from the base

W_i - gravity load on the i'th floor.

The distribution factor γ_i can be calculated using the following formula proposed by CEB |13|

$$\gamma_i = h_i \frac{\Sigma W_i}{\Sigma W_i h_i} \tag{5.3}$$

where h_i is the height of i'th floor measured from the foundation level.

The distribution of design horizontal seismic forces are shown in Fig. 5.2. When the gravity load W_i on each floor is same, Eq.'s (5.2) and (5.3) indicate that the total design horizontal forces acting is $C_d \Sigma W_i$. As shown in Fig. 5.2(b), in this case the distribution of the total horizontal force along the height of the building follows the shape of an inverted triangle.

It is important to note that the equivalent static horizontal force is assumed to act along each principal axis of the building separately (not concurrently).

The design seismic coefficient depends on many factors. In the CEB-FIP Seismic Appendix |13| C_d is expressed in the following form :

$$C_d = \frac{ISA_{max}\alpha\beta_r}{K} \tag{5.4}$$

where, I - is the importance factor related to the importance groups discussed in Section 3.2(a). Values suggested for the importance factor for each group are given in Table 5.1.

A_{max} - is the peak ground acceleration to be adopted for the seismic zone under consideration (can be expressed as a function of the seismicity index). Usually a country is divided into a number of zones of different seismic activity. An A_{max} value for each zone needs to be allocated.

S - is the site coefficient related to the soil type. Suggested values are given in Table 5.2.

K - is the behavior factor depending on the ductility provided. Suggested K values for each seismic performance category are given in Table 5.3.

α - amplification factor

β_r - is the spectral response factor, which depends on the shape of the design response spectrum and the fundamental period of the structure, T. The variation in β_r is shown diagramatically in Fig. 5.3. The viscous damping assumed in the CEB-FIP Seismic Appendix |13|, to obtain the design response spectrum, is 5% of critical.

TABLE 5.1

Importance Coefficients, I

Importance Group No.	I
III	1.4
II	1.2
I	1.0

TABLE 5.2

Site Coefficients, S

Soil Profile Type	S
S_1	1.0
S_2	1.2
S_3	1.5

TABLE 5.3

Behaviour Factor, K

Structural System	Seismic Performance Category A	B	C
Frame	2	3.5	5
Wall or Dual	2	3	4

The effects of site conditions on building response are established based on soil profile types described below :

Soil profile S_1 - rock of any characteristic or stiff soil conditions where the soil depth is less than 60 m and the soil types overlying rock are stable deposits of sand, gravel or stiffer clays.

Soil profile S_2 - deep cohensionless or stiff clay soil conditions, including sites where the soil depth exceeds 60 m and soil types overlying rock are stable deposits of sand, gravel or stiff clays.

Soil profile S_3 - soft to medium stiff clays and sand, characterized by 10 m or more of soft to medium stiff clay with or without intervening layers of sand or other cohensionless soils.

The effect of torsion about the vertical axis of the building should be included in the analysis. When the building is analyzed by means of two separate planar models, the torsional couples acting at each floor are given by the inertia force acting at that floor multiplied by an eccentricity which can be expressed as a function of the nominal eccentricity (distance between the centre of mass and centre of rigidity at the floor measured perpendicular to the direction of seismic action) and the accidental eccentricity specified in the National Code.

5.4. Principles for Assuring Strength and Ductility

In order to minimize the probabilities of structural collapse and local premature failure, the structure should be evaluated in accordance with the principles described below.

a. Locations of Plastic Hinges : Because ductility of the entire structure is better accomodated by the development of plastic hinges in beams rather than in columns (except in the base), relative strengths of beams and columns should be selected accordingly.

b. Shear Strength : Because design lateral forces are conventionally only a fraction of the actual member forces that may develop during an earthquake, critical members, including joints, must be checked for a shear force corresponding to the development of the characteristic moments at the sections where hinging is expected.

These moments must be calculated using the characteristic values for material strength and the expected axial load resulting in the maximum moment resistance.

c. Anchorage and Splices : Reinforcement at critical sections should be detailed to avoid bond failure.

d. Relative Importance : Individual elements of a structure should be proportioned so that the more important members (such as columns in lower stories) are less vulnerable than members (such as those whose failure would not start a chain reaction).

5.5. Stiffness

Adequate stiffness should be provided for control of nonstructural damage and for avoiding structure to structure interaction. For the purposes of this document adequate stiffness is ensured by establishing and checking :

- . admissible interstory drift
- . admissible overall max. displacement for some type of structures.

Interstory drift limits depends on the type and quality of the infill- For brick infills elastic interstory drift should be limited to 0.25% of the story height ($\Delta_{e\ell} \leq 0.0025\ h_{sx}$). In computing the elastic interstory drift, stiffness of uncracked sections and the design earthquake should be used.

If inelastic analysis is used to compute the drift, the limit given above for the elastic analysis should be magnified by multiplying by an appropriate factor depending on ductility. K factors given in Table 5.3 can be used for magnifying the drift limits.

5.6. Material Quality Requirements

For buildings in the seismic region, the grades of normal or light-weight concrete should be at least C16 or LC16 for buildings in seismic performance categories A and B, and C20 or CL20 for buildings in seismic performance category C.

Steel grades higher than S400 should not be used in seismic regions except in slabs. The actual yield stress should not exceed its nominal value by more than 15% and the ratio of the mean value of strength to the yield stress should not be less than 1.25 for the steel grade S220 and 1.15 for S400.

Cold work steel should not be used in seismic resistant members unless experimental evidence is available.

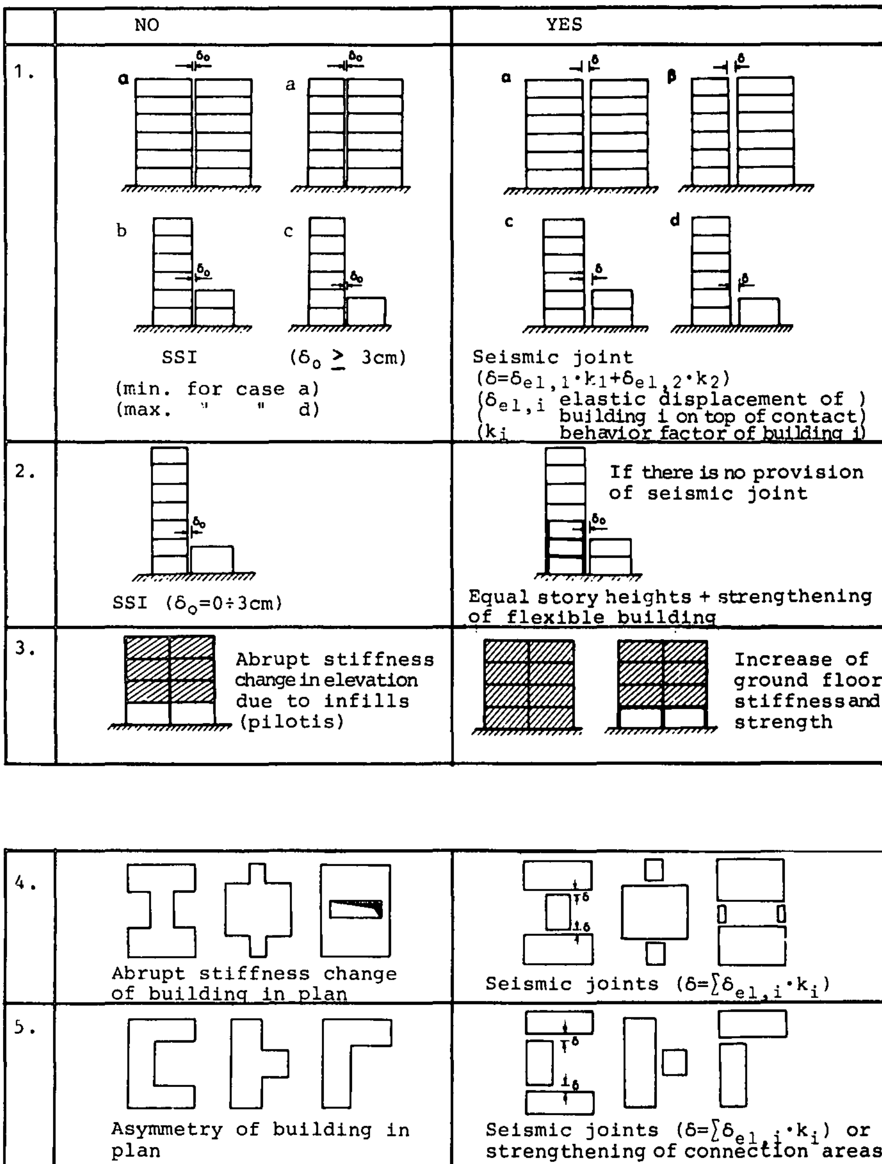


Fig. 5.1 Recommendations for building configuration

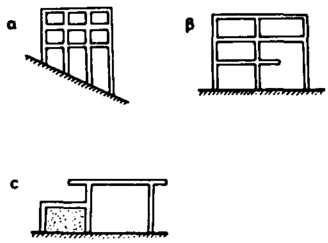
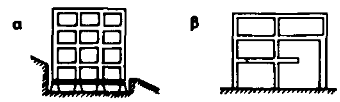
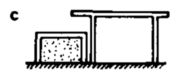
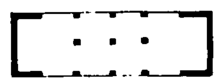
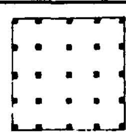
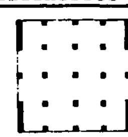
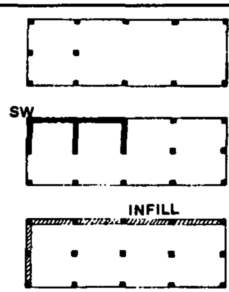
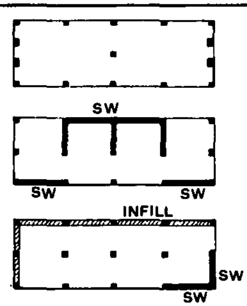
	NO	YES
6.	 Interaction of SE of different stiffness	 Equal stiffness of ground floor SE; Proper design or strengthening of flexible SE  Seismic joint ($\delta=\sum\delta_{el,i}\cdot k$)
7.	 Significant difference between stiffnesses in x & y	 Balanced stiffnesses in x & y
8.	 Equal stiffnesses in x & y	 Different stiffnesses in x & y (Separation of periods in two directions)
9.	 Asymmetric arrangment of vertical SE (Torsional vibrations)	 Symmetric arrangement of vertical SE (Reduction of torsional vibrations)

Fig. 5.1 Recommendations for building configuration (continued)

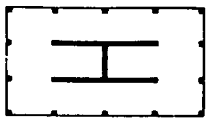
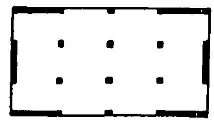
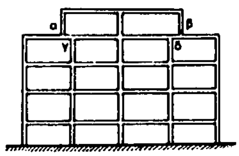
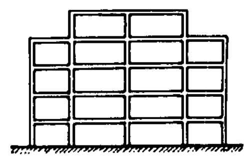
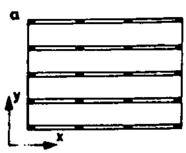
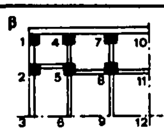
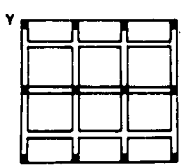
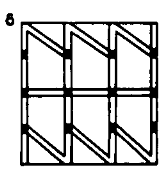
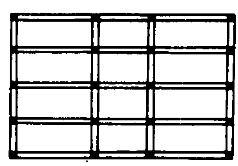
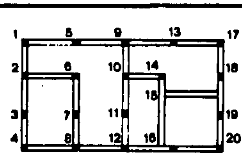
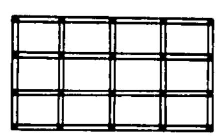
	NO	YES
10.	 Small contribution of SW in torsional resistance	 Increase of torsional resistance
11.	 Indirectly supported columns (Excessive flexibility of columns (α,β) (Overstress of joints (γ,δ)	 Uniform columns
12.	 No frame beams in y  Eccentric loading of columns   Inadequate frame behavior in x	 Good frame behavior in x & y
13.	 Unclear frame behavior	 Good frame behavior in x & y

Fig. 5.1 Recommendations for building configuration (continued)

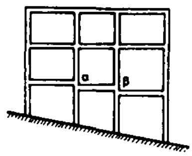
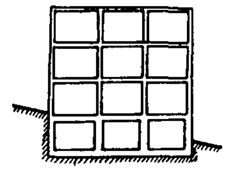
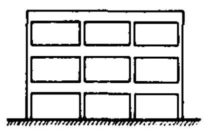
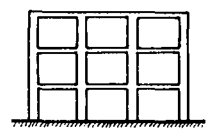
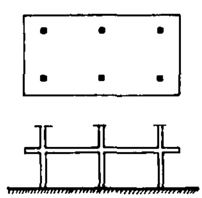
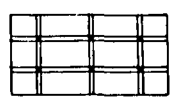
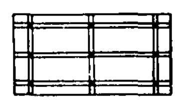
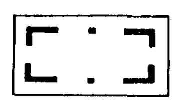
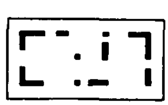
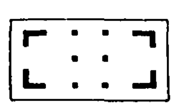
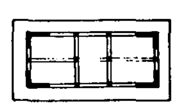
	NO	YES
14.	 Bad frame behavior (Unknown behavior of joints α & β)	 Good frame behavior
15.	 Strong beams-weak columns (Shear beam) (Column failure mechanism)	 Weak (comparatively) beams - strong columns (Flexural beam) (Beam failure mechanism)
16.	 Flat slab (Small ductility) (Eccentric punching)	 Large cantilevers (Should be avoided)  Small cantilevers (Preferable)  SW for 100% of seismic actions Provision of beams,or SW for 100% of seismic actions
17.	 Flat slab Unclear frame behavior	 Flat slab  Plate on beams (Preferable) Good frame behavior

Fig. 5.1 Recommendations for building configuration (continued)

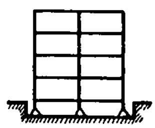
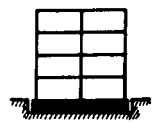
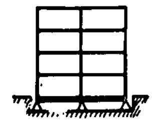
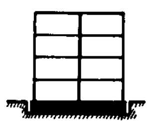
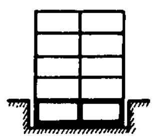
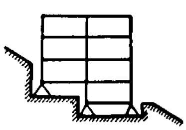
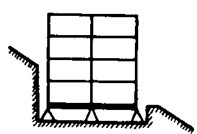
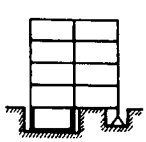
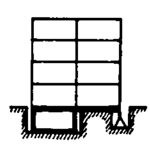
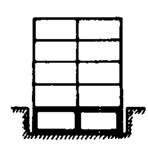
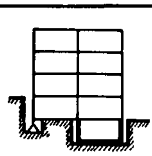
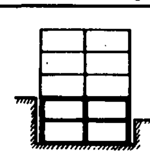
	NO	YES
18.	 Unconnected spread footings (Independent displacements)	  Combined footings or strong connection beams
19.	 Small depth of foundation (Small bearing capacity) (Significant soil-structure interaction)	 Stiff basement for good base fixity
20.	 Foundation at different levels (Asymmetric seismic excitation) (Asymmetric structure)	 Uniform foundation level
21.	 Foundation asymmetry (Asymmetric seismic excitation) (Asymmetric structure)	 Stiff and strong foundation  Uniform foundation (preferable)
22.	 Foundation asymmetry with different foundation levels	 Uniform, stiff and strong foundation

Fig. 5.1 Recommendations for building configuration (continued)

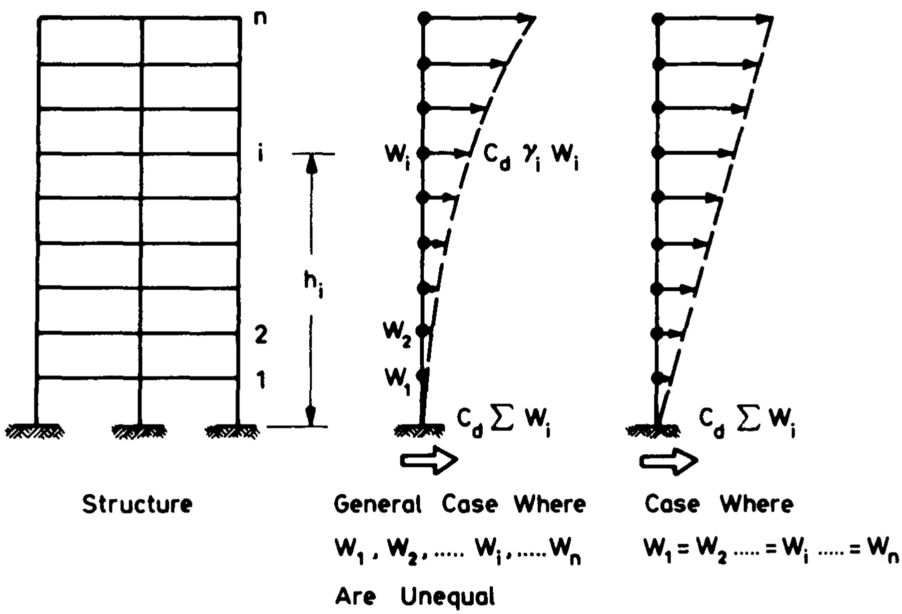


Fig. 5.2 Distribution of Design Horizontal Seismic Action

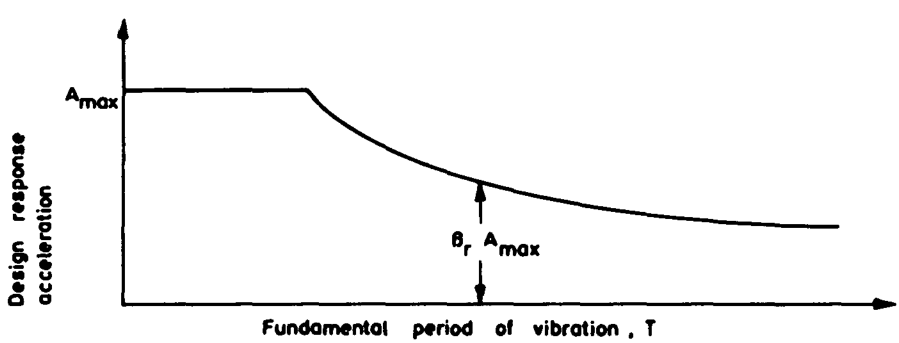


Fig. 5.3 Typical Shape of Design Response Spectrum for Determining the Design Seismic Coefficient

6 - DETAILING

6.1. Introduction

Design is not completed until detailing of reinforcement, infills and architectural elements has been ensured. No matter how correctly and accurately the previous steps of design have been carried out (like selecting of structural configuration, estimation of seismic action, analysis and dimensioning of critical sections) safety cannot be assured unless proper detailing is provided. As a matter of fact, good detailing is by itself the basic criterion for the safety of the structure.

Detailing is the last stage of the design process and a subject for which limited research results, aids and recommendations are available. It is also the stage at which a considerable volume of detailed drawings, lists and specifications must be prepared, so that the contractor and the detailer will be forced and guided to construct exactly the intended structure. In many cases detailing is usually the design stage with the lowest quality.

The quality of detailing depends on :

a. Knowledge of structural behavior.
b. Experience
c. Talent and structural intuition.

Experience and talent are indispensable, but knowledge can be improved.

Reinforced concrete is a complicated construction material and multi-story buildings a new type of construction, so that the designer cannot depend on contractor's and technicians' "good construction tradition". It is the designer who has to provide adequate and clear documents (drawings, lists, specifications, etc.) that will help the contractor to understand without doubt what he is going to build and how he should build it. Then, it is contractor's duty to further clarify things for the detailer.

The brief preliminary comments of this section aim only at improving the existing knowledge of the designer and, to a lesser extent, of the contractor.

Although detailing is one of the most important factors affecting structural behavior under seismic action, detailing by itself cannot provide the desired safety unless good building configuration has been chosen and structural members have been proportioned properly.

It is essential that detailing should be adequate, simple and result in economy of steel. Whenever possible, detailing should be standardized.

Detailing adequacy means that through proper detailing good behavior of the final real structure will be ensured. Knowledge of the expected combinations of physical , chemical and mechanical actions and of real structural behavior in such an environment is indispensable. Simplicity is necessary for ease and economy of construction. Standardisation will minimize the number of possible cases, a fact that will enable deeper study of detailing cases, better designer-contractor-detailer communication and to a certain extent, computerization of detailing documents. Economy is of course a self evident target.

Although all possible actions and their most probable combinations should be taken into account during detailing, one should always bear in mind that seismic actions and fire are the actions that usually impose the most severe detailing requirements.

A summary of minimum requirements for proportioning and detailing of members and joints of seismic resistant reinforced concrete structures are given in Part 1 of this manual. In this chapter general principles for detailing of reinforcement will be given and some typical examples will be presented. Comments on infill and architectural detailing will not be discussed, due to space limitations.

6.2. Reinforcement

6.2.1. Introduction

Typical reinforced concrete design calculations end with some figures giving the necessary area of reinforcement at certain critical regions of the model structure, for some action effects, which are based on simplified analysis of the model structure. Even if a good design code has been followed with care up to this point, the question is how these figures will be translated and incorporated into the structure, so that a safe and economic structure will be the end result of the design and construction process. Good reinforcement detailing is necessary at this point.

Minor quantities of reinforcement (usually less than 0.5% of the total cost of the structure) placed in the right position and with the proper method can increase substantially the overall safety of the structure. In some cases, a small reduction of the quantity of reinforcement can improve workability to an extent that both economy and safety will be increased at the same time. In detailing of reinforcement, quality is at least of the same importance as quantity.

A number of tentative recommendations on reinforcement detailing will be presented here. Much work has still to be done to facilitate the designer's work and it is definitely worthwhile. Valuable information on many aspects of the problem can be found in references 4, 5, 13, 14, 15, 23, and 27.

6.2.2. Role of reinforcement

Reinforcement is used to :

a. Resist tensile stresses, or reduce opening of cracks (tensile reinforcement)
b. Resist compressive stresses (compression reinforcement)
c. Increase the ductility
d. Provide lateral support to compression reinforcement bars (lateral support reinforcement)
e. Help assemblying a stable reinforcement cage (assembly reinforcement)

Tensile and compressive reinforcement are usually termed as the main reinforcement. They influence directly stiffness, strength and ductility of RC structural elements, as well as continuity of the structural system. Their influence on stage I (uncracked) stiffness and strength is usually small, but their influence on stage II (cracked, pre-yield) and on stage III (post-yield) stiffness and strength is predominant.

Confinement, lateral support and assembly reinforcement are usually termed secondary reinforcement. Yet, confinement and lateral support reinforcements (which are usually combined in one) influence materially ductility of the structural elements, while assembly reinforcement is very important for good construction.

Two types of reinforcement can be distinguished based on the method of determination of their quantity, position and direction :

a. Calculated reinforcement
b. Reinforcement by estimate

Calculated reinforcement is the reinforcement whose quantity, position and direction has been determined through some specific calculations, which are assumed to guarantee fullfilment of stiffness, strength, ductility and continuity requirements. It is desirable that, all the required reinforcement should be based on such a procedure of determination. Yet, in medium cost structures not all specific problems are covered by proper analysis and design calculations. In these cases, a certain amount of local reinforcement should be provided as reinforcement by estimate, based on engineering judgement. Assembly reinforcement, in many cases continuity reinforcement and reinforcement for local stress concentrations, are typical examples of reinforcement by estimate.

6.2.3. Detailing requirements

Reinforcement should be detailed for :

a. Serviceability (elastic stage)
b. Strength under monotonic loadings based on failure mechanism
c. Seismic capacity, based on failure mechanism + ductility demand
d. Structural continuity, based on continuity requirement
e. Structural hierarchy, based on failure hierarchy

Although each reinforcing bar serves only a limited number of uses, reinforcement should be provided in every structural region for the simultaneous verification of all the above detailing requirements.

6.2.4. Basic problems

During detailing, the following problems should be taken care of :

a. Environmental protection
b. Bond and anchorage
c. Splices
d. Workability (both of concrete and of the reinforcement itself).

The severity of the workability problem depends on the intensity of seismic actions and on the technological level of the construction personnel. High seismic intensities require large quantity of well-detailed reinforcement, increasing the workability problem, especially for construction personnel of low technological level. A balance between the required level of detailing and the skill of the available technicians is necessary, otherwise a danger exists for very poor final construction.

6.2.5. General recommendations (Fig. 6.1)

A limited number of simple-to-state but sometimes difficult-to-apply rules should always be followed during reinforcement detailing :

a. Main reinforcement for space behavior with definite structural hierarchy, covering all basic detailing requirements
b. Adequate main reinforcement ratios
c. Double (tensile + compression) main reinforcement
d. Good anchorage of main reinforcement
e. Sufficient and well anchored hoops (especially in joints and critical regions)
f. Sufficient concrete cover
g. No abrupt changes of main reinforcement
h. No splices in critical regions
i. Simple overall solutions
j. Concrete workability
k. Standard details
l. Maximum possible economy.

No matter what simplifying assumptions for modeling and analysis have been made during the initial design steps, detailing needs should be identified based on an at least qualitative analysis of a realistic model of the structure (Fig. 6.1a). Three dimensional structural behavior should be anticipated, the position of construction joints (j) should be defined and the whole structure should be divided in regions of certain type and hierarchy.

Structural system of structural regions should be established (Figs. 6.1a and b), in order that a correct structural system will be designed through proper detailing.

Three types of structural regions :

a. Type I : Linear (1D) region
b. Type II : Plane (2D) region
c. Type III : 3-D region

should be distinguished and each one of them should be detailed accordingly.

Double (tensile + compression) main reinforcement should be provided, at least at the critical (high hierarchy rank) structural regions, so that adequate ductility and safety for the case of unexpected load reversals will be ensured.

Good anchorage of reinforcement is a basic requirement for overall structural safety. Normal methods of anchorage are :

- straight anchorage;
- curved anchorage : hooks (at 135^o to 180^o), bents (at 90^o to 135^o), loops;
- anchorage by mechanical devices.

The basic anchorage length is,

$$\ell_b = \frac{\emptyset}{4} \frac{f_{yk}}{f_{bd}} \tag{6.1}$$

f_{yk} - characteristic yield strength of reinforcement

f_{bd} - design bond strength given in National codes (or CEB-FIP Model Code)

The required anchorage length ℓ_{bn} can be expressed as a function of ℓ_b.

$$\ell_{bn} = \alpha \ell_b \leq \ell_{b,min}. \qquad (6.2)$$

$\alpha = 1.0$ for straight bar anchorage (tension or compression)
$\alpha = 0.7$ for curved bars in tension
$\alpha = 1.0$ for curved bar anchorage in compression
$\ell_{b,min} = 0.3\ell_b$ but not less than $10\emptyset$ and 100 mm (for bars in tension)
$\ell_{b,min} = 0.6\ell_b$ but not less than $10\emptyset$ and 100 mm (for bars in compression

Free bending diameter of curved bars should not be less than 5 bar diameters.

Curved bars should have a straight tail portion equal to :

$5\emptyset$ for 135^o and 180^o bents of plain bars
$10\emptyset$ for 90^o bents of deformed bars and 135 bents of the hoop reinforcement
$20\emptyset$ for the development length of beam.

Abrupt changes in the main reinforcement should be avoided because such changes cause severe stress concentration problems.

Splices are weak points where severe bond and stress concentration take place. Therefore splices in the critical regions should be avoided as much as possible. To decrease the stress concentration, lap splices should be staggered.

The required lap length can be calculated from the following formula,

$$\ell_s = \alpha_1 \ell_b \leq 15\emptyset \leq 200 \text{ mm} \qquad (6.3)$$

α_1 is a coefficient which depends on the percentage of reinforcement lapped in any one section. Values of α_1 can be found in CEB-FIP model code or the National Codes. Permissible percentage of lapped bars in any one section are given in Table 6.1.

TABLE 6.1

Permissible percentage of Lapped Bars in Any One Section

Quality Bond	Load Effects	
	Static	Seismic
Deformed bars	100%	50%
Plain bars $\leq \emptyset 16$ mm	50%	25%
Plain bars $< \emptyset 16$ mm	25%	25%

Simple and standard detailing solutions will improve the understanding of the placer, ensuring better construction and will permit good workability of concrete and the reinforcement.

The need for economical solutions is of course self evident.

6.2.6. Responsibilities and documents

In a large scale complex structure the following persons involved in the detailing process can be distinguished :

a. Designer (the engineer responsible for designing the structure)
b. Supervisor (the engineer responsible for supervising the construction as the Owner's representative)
c. Contractor (the engineer responsible for the construction of the work)
d. Detailer (the person (engineer or technician) responsible for preparation of all fabrication and placing documents)
e. Fabricator (the person responsible for cutting and bending of all reinforcing bars)
f. Placer (the technician responsible for proper placing of all reinforcing bars).

In most of the cases, some of these responsibilities are combined (e.g. designer + supervisor, contractor + detailer or fabricator + placer).

It is imperative that each person involved clearly understands his responsibilities and produces the detailing documents that are needed by the person involved in the following step. The designer and detailer produce the most basic detailing documents and for this reason, they need some standard detailing aids. Aids for typical details, for detailing structural elements and for preparation of detailing documents are needed.

Contract specifications should specify the types of detailing documents that are to be produced and the persons responsible for their preparation.

6.2.7. Aids for detailing structural elements

All detailing requirements conforming to the applied code for each type of structural element should be presented in tabulated form in a single page and illustrated by proper drawings, so that the work of the designer and the detailer will be facilitated. Such aids are needed for the case of :

a. Slabs
b. Beams
c. Columns
d. Structural walls
e. Stairs
f. Foundation elements
g. Beam-beam joints (indirect supports)
h. Beam-column joints
i. Plane element joints
k. Flat slab-column joints
l. Coupling beams.

Examples of application of the above detailing requirements should also be provided for each one of the types of structural elements.

6.2.8 Detailing documents

Standards, aids and examples of detailing documents are needed, so that the work of the designer and detailer will be facilitated and better communication between the persons involved in the detailing process will be established.

Simplicity, standardisation, reproducibility and durability are the desirable characteristics of the detailing documents. Standard dimensions, symbols and notations, proper scales and legible lettering in the final size of the documents should always be used.

Engineering drawings should contain adequate notes and other essential information, in a form that can be quickly and correctly interpreted by an engineer, illustrating clearly all detailing requirements. They should show type and grade of steel, service live load, concrete strength, concrete dimensions, lap lengths, concrete cover for reinforcement, required joints, and any other information necessary for the preparation of placing drawings.

Placing drawings should contain all information necessary for complete fabrication and placing of reinforcing steel and bar supports, a form clearly understandable by a technician and should have the designer's approval.

Combination engineer-placing drawings should cover simultaneously the requirements of engineering and placing drawings.

Bar lists (schedules) are prepared and distributed either separately or on the drawings. They constitute a compact summary of all bars, with the number of pieces, shape, size, lengths, marks and bending details, from which shop orders can be easily and readily written.

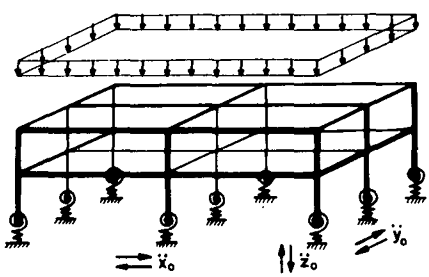
Detailing specifications should contain all information not included in drawings and bar lists (schedules) necessary for complete good fabrication and placing of reinforcing bars and bar supports.

6.2.9. Typical details

Typical details for beams, columns and structural walls are presented in Part 1 of this manual with minimum proportioning and detailing requirements. In this Chapter some typical details are given for beam-column joints in Figs. 6.2-6.5 and for joints of plane elements (walls) in Fig. 6.6.

Criterion	Case	Detailing tool
Structural region type	• Linear (I) • Plane (II) • 3-D (III)	
Design level	• Serviceability • Strength under monotonic loading • Seismic capacity	• Elastic stress distribution • Failure mechanism • Failure mechanism + ductility demand
Detailing requirement	• Structural continuity • -"- adequacy • -"- hierarchy	• Action effects • Failure mechanism chain

Target: Amount + position + direction of reinforcement



S.C.: Structural Continuity
R.T.: Region Type
S.H.: Structural Hierarchy

Simplified overall model

Wrong assumptions | Right assumptions

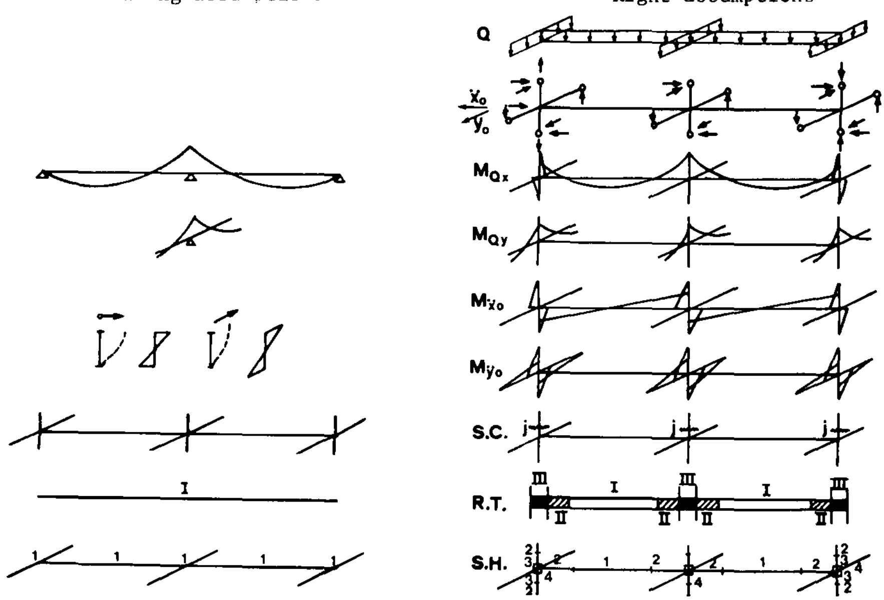
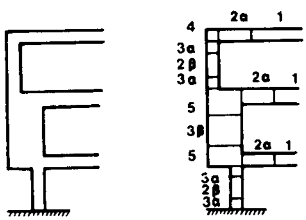
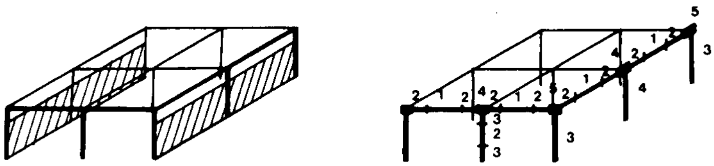


Fig. 6.1a General recommendations for detailing



Structural region		
Number	Type	Action effects
1	Linear (I)	M
2a	-"- (I)	M + V
2β	-"- (I)	N + V
3a	-"- (I)	N + M+V
3β	Plane (II)	-"-
4	-"- (II)	-"-
5	-"- (II)	-"-

Assymetry in structural element



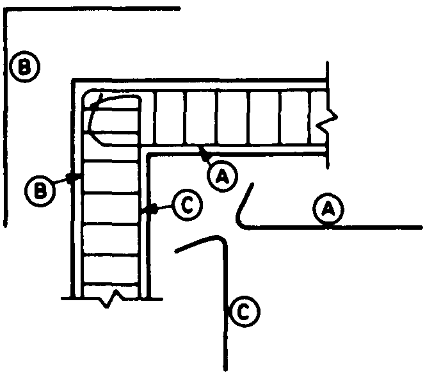
Interaction between SE of different stiffness

Interaction between SE of different stiffness

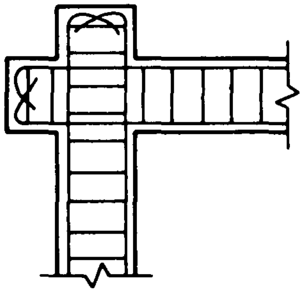
Abrupt stiffness change in building plan

Abrupt stiffness change in building elevation

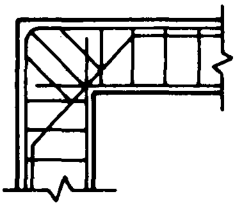
Fig. 6.1.b General recommendations for detailing (continued)



1- Typical simplified detailing

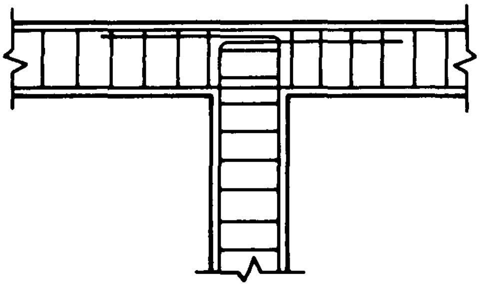


2 - Joint with stubs



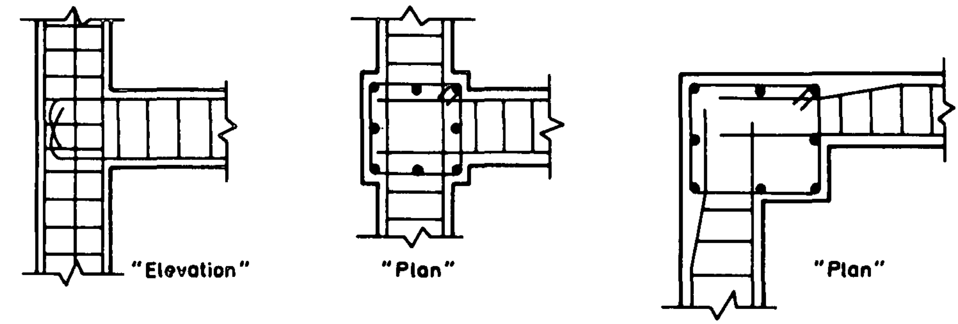
3 - Joint with inclined hoops

Figure 6.2 Detailing of Beam - Column L - Joints



4 - Typical detailing

Figure 6.3 Detailing of Beam - Column T - Joints

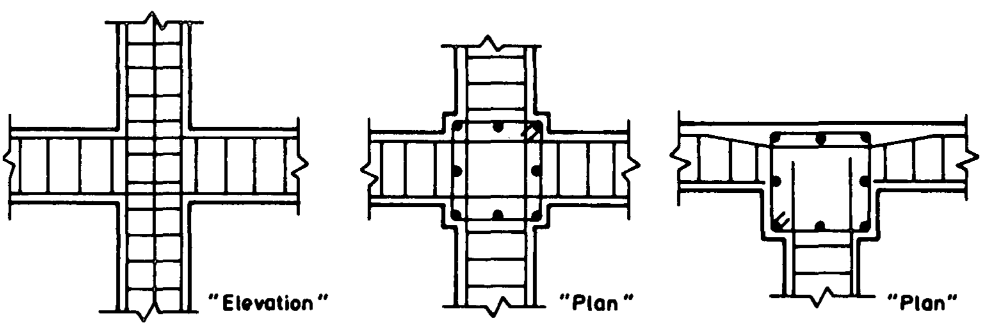


5 - Typical joint detailing

6 - Joint well confined by beams

7 - Joint not well confined by beams

Figure 6.4 Detailing of Beam - Column ⊢ Joints

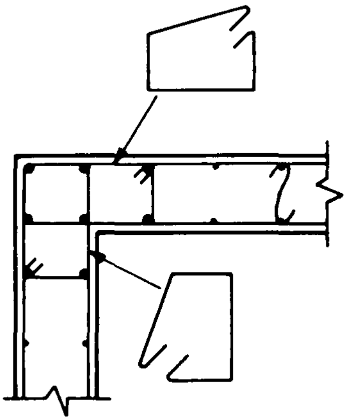


8 - Typical joint detailing

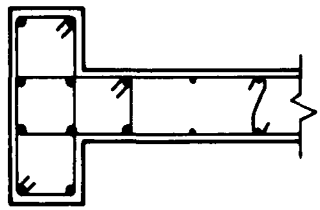
9 - Joint well confined by beams

10 - Beam longitudinal steel detailing for edge joints

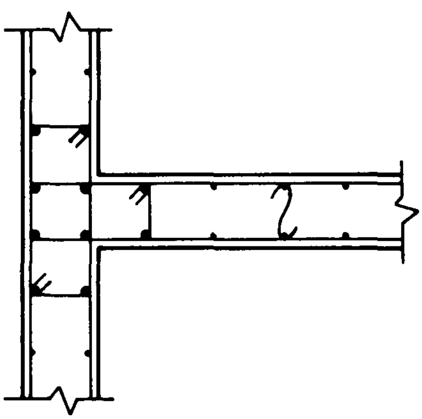
Figure 6.5 Detailing of Beam - Column + joints



(a)



(b)



(c)

Fig. 6.6 Detailing of plane element joints

7 - QUALITY CONTROL

7.1 Introduction

With the exception of precast construction, reinforced concrete is made on the work site. Quality of steel and concrete, sizes of members, placement of the bars, quality of formwork and curing can vary considerably. Therefore continuous inspection and supervision on the job are extremely important in order to produce a reinforced concrete building which will have the material quality, member sizes and steel detailing as specified by the designer. A perfect design can not produce a serviceable and safe structure unless the design requirements are satisfied in the construction stage.

Design should also be checked by qualified engineers, at least to eliminate the possibility of gross errors.

For buildings to be built in seismic regions, nature and location of the nonstructural components are also very important, since they influence the dynamic characteristics of the structure significantly. Therefore, the design engineer should work with the architect to make the final decisions on the nature and location of non-structural elements. The location of such elements should be checked by the inspecting engineer. Building regulations should clearly state that the location of non-structural elements cannot be changed without the permission of building authorities.

It can be said that the dynamic response characteristics of an earthquake resistant building are established at the design and construction stages. For satisfactory behavior, a good design (including building configuration) is essential, however not adequate. The structure which is expected to resist the seismic action is not the structure on the design drawings but is the physical structure which is constructed.

There are mainly three engineering groups responsible for the structure built; (a) design engineers, (b) inspecting engineers (both for the design and the construction), and (c) contractor. Although the responsibility during construction must be coordinated, the role each party is expected to play and the responsibility of each party should be well defined.

It should be noted that the recommendations made in this chapter are intended to be just guidelines. Requirements given in National Codes on quality control will govern.

7.2. Quality Control in the Design Stage

The structural design of a seismic resistant building should be checked either by the building authorities or by qualified proof engineers. A great many of the gross errors which can cause substantial decrease of the overall structural safety can be detected and corrected through quality control of the design.

For structures to be built in seismic areas, the overall configuration and configuration of elements, basic assumptions, and detailing of critical sections should be checked. Checking the design should not only be a formality but the proof engineer should legally share the responsibility with the designer.

7.3. Quality Control at the Construction Stage

Quality control at the construction stage is extremely important, because as was mentioned earlier the behavior of the structure depends on the physical properties which are produced during the construction and not on the properties which are seen on the design drawings. It should also be recalled that a great percentage of the failures observed have been caused by mistakes made during the construction stage including poor material quality.

For buildings to be built as seismic resistant structures, a qualified experienced engineer should be responsible for inspecting the construction. The inspecting engineer should share the legal responsibility with the contractor.

Materials should be inspected and tested at delivery and before use. Samples should be taken from concrete regularly and tested to determine the strength. Formwork, placement of reinforcement and details should be checked by the inspecting engineer. Concrete casting should start only after the forms and reinforcement is inspected and approved by the inspecting engineer.

The quality, location and anchorage of non-structural elements should also be checked in accordance with structural, mechanical and electrical drawings and specifications.

The designer should specify the quality requirements, the contractor should exercise the control to achieve the desired quality and the owner should monitor the construction process through inspection to protect the public interest in safety of buildings. It is essential that each party recognize its responsibilities, understand the procedures, and be capable of carrying them out. Because the contractor and the specialty subcontractor are doing the work and exercising control on quality, it is essential that the inspection be performed by someone not in their direct employ and also be approved by the building authorities. When the owner is also the builder, he should engage independent agencies to conduct these inspection rather than trying to qualify his own employees.

For important buildings a "quality assurance plan" should be drafted. The quality assurance plan should be prepared by the persons responsible for the design of each seismic system subject to quality assurance whether it is architectural, electrical, mechanical or structural in nature. The quality assurance plan may be a very simple listing of those elements of each system which have been designated as being important enough to receive special inspection and/or testing. The extent and duration of inspection must be set forth as well as the specific tests and the frequency of testing.

The building authorities must approve the quality assurance plan and must obtain from each responsible contractor a written statement that he understands the requirements of the quality assurance plan and that he will exercise control to obtain conformance.

The success of a quality assurance plan depends upon the intelligence and knowledge of the inspector and the accuracy and thoroughness of his reports. It should be emphasized that both the inspector and the contractor are required to submit to the building authorities a final certification as to the adequacy of the completed work. In the final certificate the contractor must take full responsibility for every detail of the construction and should state that the work has been completed in accordance with approved plans and specifications. The inspector can only attest to the work he has personally inspected and shares the responsibility with the contractor.

The inspector should inspect the items listed in the quality assurance plan in accordance with the specification related to that item . He should check the concrete materials and reinforcing steels regularly by testing. He should also inspect the mixing, placing and curing of concrete. Strength of concrete should be checked regularly. The details of material quality control and related testing follows the procedures long established by standards. The guidelines and details can be found in the publications of international organizations such as CEB, ACI, ASTM and RILEM. Related provisions can also be taken from the National Codes.

At any stage of the construction if there is a disagreement between the contractor and the inspector, the design engineer is called to make the final decision by taking the responsibility.

If the concrete quality is found out to be lower than the specified strength, the inspector can order the contractor to stop the construction. In such cases the design engineer is informed and the design engineer decides whether the contractor can go on with or without any precautions. Even if the engineer decides that the structure is safe with lower concrete strength, the contractor should pay a penalty for producing poor quality concrete.

In the specifications, the responsibilities and authorities of each party should be clearly defined. The specification should also include penalty clauses for the poor quality.

The importance of quality control in the construction of seismic resistant structures cannot be overemphasized. During past earthquakes, building failures which are directly traceable to poor quality control are numerous. In many cases reports on such failures point out that the failure would not have taken place if proper inspection had been exercised.

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PART 3

DESIGN EXAMPLES

RUMANIAN DESIGN EXAMPLE
RC SHEAR WALL STRUCTURE

1. INTRODUCTION

1.1 General

This section presents a complete design example for a reinforced concrete shear-wall structure of a multistory building. The general organization of the design process, i.e. the load calculations, the method of analysis and the design of R.C. elements are those used in the Rumanian design practice. The notations used herein are those of the CEB Model Code 6 . The design equations were transformed for allowing the use of the CEB Model Code design values for steel and concrete strengths.

In this example the design is made for a "special" load combination which takes into account the seismic action, in the form of a code loading without an explicit analysis of the structural response in the post-elastic range of behavior. It is assumed that by proper detailing the structure is provided with sufficient deformation capacity for the post-elastic range.

The most important requirements to be observed in design in order to ensure such a behavior are:

a) the design shear forces in structural elements are directly associated with the ultimate bending moment capacities;
b) the shear force intensity is limited in cantilever walls and coupling beams;
c) the axial compressive stress in vertical elements is limited;
d) the splicing and anchorage length of bars is increased.

Requirements "a" and "b" aim at avoiding premature brittle failure of R.C. elements under shear-force action, before developing important flexural deformations.

Requirement "c" is aimed at providing vertical elements with sufficient curvature (flexural) ductility.

Requirement "d" aims at preserving bond between steel and concrete under high-intensity cyclic loading.

The design process of R.C. structures generally consists of four principal steps:

- the choice of the structural layout, the schematic structural model and loading for analysis and the preliminary proportioning of the structural elements;
- computation of internal forces;
- dimensioning of the cross sections;
- detailing of reinforcement.

An elastic analysis can usually be based on the concrete cross sections as resulting from the preliminary proportioning. The post-elastic analysis, either explicitly or by means of simple relations as those assumed by the fulfillment of requirements "b" and "c", previously mentioned, also implies the need of a preliminary dimensioning of the reinforcement (mainly, the longitudinal reinforcement). Such preliminary dimensioning is currently done by means of elastic analysis. In case of shear-wall structures these steps are affecting one another in greater measure than for other types of structures. This is mainly due to greater difficulties in finding preliminary proportions that would not require subsequent alterations.

The detailing rules are aimed at limiting the development of inelastic deformations in the coupling beams and in the lowest part of the shear-wall. Thus the demand of rotation in plastic hinges is reduced and the supplementary transverse reinforcement for confinement of concrete is concentrated in very limited zones. For this purpose, the ratio between the ordinates of the design-resisting moments diagram and the ordinates of the induced-moments diagram must be minimum in the lower part of the shear wall.

In the potential plastic-hinge zones the concrete capacity to transfer shear is drastically reduced due to high-intensity cyclic loading. In the design of coupling beams, according to the Rumanian Regulations, the capacity of concrete to carry shear force is neglected. In the potential plastic hinge zone of vertical elements, this capacity is considered to be about 70% of the capacity of the concrete in the rest of the shear-wall.

For a better understanding of the successive steps, a flow chart of the main operations is presented:

The choice of the structural layout, the schematic structural model and loading for analysis and the preliminary proportioning of the structural elements with observance of requirement "c".
↓
Computation of internal forces under code lateral loading in an elastic analysis.
↓
Design of reinforcement in the walls and coupling beams; using the effective area of steel compute the design resisting moments.
↓
Computation of the design shear forces corresponding with the flexural capacity and checking of concrete sections using the limitations set under "b".
↓
Is it necessary to alter the concrete cross sections or/and the general layout? YES → (return to first step)
NO ↓
Design of reinforcement for shearing forces in walls and coupling beams.
↓
STOP

1.2 Method of Analysis

The analysis of shear-wall structures for multistory buildings under the action of horizontal forces is generally made using an available computer program developed for such structures. In many cases one can use, with good results, manual methods for determining the distribution of bending moments, shear and axial forces in the elements of shear-wall structures, without consumming too much time. This is the case for structures which can be considered as "monotonous" over the building's height, i.e. observing the following conditions: (Fig. 1)

- the story height is the same for every floor;
- the shear wall thickness is kept constant from bottom to top;
- the openings are superposed and have the same dimensions at every floor;
- the coupling beams are identical at every floor.

Accepting the horizontal load distribution over the building height as uniform (for wind loading) or triangular (for seismic loading) a simplifying assumption may be made; namely, the deformed shape of a cantilever wall and a coupled shear wall are affinitive and consequently it is sufficient to impose the deflections equality condition at a single level, the "characteristic" level. Furthermore, if the number of stories is greater than six, one can replace the real coupling beams by an equivalent continuous medium thus obtaining a fictitious shear wall having the same stiffness as the real shear wall (Fig. 2). This enables a system of linear equations corresponding to the real structure to be reduced to a single differential equation with constant coefficients. This equation has been solved for most practical cases and the results are presented in the form of graphs or tables which enable finding the values of the internal forces. The method used in several papers with different patterns [1], [2] , [3] , etc. The current design in Rumania uses the variant presented in [7].

If the number of stories is less than six, the above method becomes inaccurate. However, if it is satisfactory to consider only the first mode of vibration, the method can also be used for structures with more than eleven stories. However, for high-rise, significantly nonsymmetrical buildings and for buildings presenting a great variation in the mass or stiffness distribution over the height, this method can be used only for preliminary design.

Taking into account the strictly practical character of this design example, the derivation of pertinent equations is not presented herein. However, the paper contains:

- symbols and design equations which permit the use of design charts;
- design charts for the determination of forces and stiffnesses of coupled shear walls and tables for the computation of forces and deflections in the cantilever shear walls.

1.3 Steps in the Analysis

Considering the assumptions previously presented, the design of the internal forces induced by horizontal loading in R.C. shear-wall structures involves the following operations:

a) Computation of equivalent moments of inertia for each shear wall in the structure; by definition, the equivalent moment of inertia of a cantilever wall, which, under the same actions has the lateral flexural displacement, at a certain level, equal to the displacement (caused by bending moments, axial forces and shearing forces) of the actual shear wall. The displacement equality is usually imposed at the floor level nearest to 0.8 H, called "characteristic" level [2].

b) Determining of the location of the centroid of stiffnesses.

c) The total seismic force determined for the structure is distributed to each shear wall proportionately to its stiffness, respectively with its equivalent moment of inertia, taking into account the effects of both translation and general torsion.

d) Determination for each shear wall of the bending moments, axial forces and shear forces considering the lateral force, triangularly distributed over the height of the building.

2. DEFINITIONS AND NOTATIONS

A	area of shear-wall section
Af	area of flanges for T, L, I shaped shear-wall sections
Aw	cross-sectional area of web
Aw,v	effective cross-section area of shear wall for computation of shear displacements
Ab,v	the same, for coupling beams
As	area of longitudinal tension reinforcement
A's	area of longitudinal compression reinforcement
As α	area of coupling beam reinforcement inclined at an angle α to horizontal
As,w	area of a stirrup leg, of coupling beam
Ash	total horizontal steel area over story height
As,v	total vertical steel area in shear wall web
b_a	design web width of shear wall
B_f	actual flange width
b_f	effective flange width
b_w	web width of both shear wall and coupling beam
c	concrete cover
l_a	length of the deformable zone of shear wall situated at coupling beam ends
d	effective depth, the distance between the extreme compression fiber to the centroid of reinforcement
d_i	distance between the extreme compression fiber to centroid of current web reinforcement bar, i
E_c	modulus of elasticity of concrete
E_w	conventional modulus of elasticity of concrete in shear walls
E_b	conventional modulus of elasticity of concrete in coupling beams
G_c	transverse modulus of elasticity
f_{cd}	design compressive strength of concrete
f_{yd}	design yield strength of steel
h	depth of shear-wall cross section
h_b	depth of coupling beam
h_f	flange thickness
h_{sx}	story height
H	total height of shear wall
k	factor accounting for cross-section shape influence on shear displacements computation

l_d	design span of coupling beam
l_l	distance from the centroid of the shear wall cross section to that of the shear wall located immediately to the left
l_{on}	clear span of coupling beam
l_r	the same, for shear wall located immediately to the right
L	span of coupling beam (distance between the centroids of coupled walls)
M_d	design bending moment resulting from code loads
$\overline{M}_d$	adjusted design bending moment
M_r	ultimate flexural strength of either shear wall or coupling beam cross section
n	number of stirrup legs or number of stories
N_d	design axial force
s_n	stirrup spacing
V_d	design shear force
V_{du}	design shear force associated with M_r
V_{cd}	shear force carried by concrete
X	neutral axis depth
$\overline{X}$	conventional value of neutral axis depth (assuming a rectangular stress block)
X_b	neutral axis depth for balanced failure
$\overline{X}_b$	conventional value of neutral axis depth for balanced failure
α	inclination angle of diagonal reinforcement in coupling beam
δ	$b - b_a$ = dimension by which shear-wall thickness is conventionally reduced for design purposes
$\Delta b_l, \Delta b_r$	left side flange and right side flange, respectively, of the effective flange width: $b_f = b + \Delta b_l + \Delta b_r$
ρ	reinforcement ratio (percentage)
$\xi = x/d$	relative depth of compression zone
$\overline{\xi}$	conventional value of relative depth of compression zone
$\xi_b = x_b/d$	relative depth of compression zone at balanced failure
$\overline{\xi}_b$	conventional value of relative depth of compression zone at balanced failure
$\chi_i = d_i/d$	relative distance between the extreme compression fiber to the centroid of current intermediary reinforcement bar, i
σ_s	stress in tension reinforcement A_s
σ'_s	stress in compression reinforcement A'_s
σ_{si}	stress in current intermediary reinforcement bar at failure
ε_s	strain in tensile reinforcement A_s
ε'_s	strain in compression reinforcement A'_s

ε_{cu}	ultimate compression strain of concrete
τ_{Rd}	nominal ultimate shear stress
γ	magnification factor of design shear force
μ_{φ}	curvature ductility factor
μ_{Δ}	structure ductility factor

3. DESIGN EXAMPLE

The present example deals with the design of an R.C shear wall-structure. The building is a nine-story block of flats sited in Bucharest (seismicity index equal to 8 on the MSK scale) and considered to correspond to conditions of first level of safety. The horizontal and vertical sections of the structures are presented in Fig. 3. The entire R.C. structure is cast-in-situ.

With no other rigorous definitions available, usually structures included in the first level of safety are considered as corresponding in principle to those in regions with a seismicity index on the MSK scale greater than 7. For the second level of safety, correspondingly, a seismicity index less or equal to 6 on the above mentioned scale is being considered.

According to the present codes and based on existing data, the foundation soils in Bucharest are considered not to influence unfavorably the structural response to the seismic action. In the rest of the country the code seismic forces are 30 percent increased for low rigidity soils (soft clays, sand) and 20 percent reduced for stiff soils (rocks).

3.1 Design of Seismic Forces

According to the Rumanian design provisions, the seismic action is defined as an "exceptional" action, thereby meaning that it is likely to occur very rarely, if at all, with significant intensity during the lifetime of a building. The seismic base shear is computed as follows:

$$S = k_s \beta \psi \varepsilon G \tag{1}$$

where G is the total gravity load of the building

k_s represents the ratio between the expected peak value of horizontal ground acceleration and the acceleration of gravity; as a function of the MSK scale of intensity 6, 7, 8, or 9, k_s is equal to 0.07, 0.12, 0.20 and 0.32 respectively.

β is a dynamic amplification factor dependent on the period of vibration of the structure (T) and on the type of foundation soil; for normal soil where $\beta = 3/T$:

$$0.75 \leq \beta < 2 \tag{2}$$

ψ is a reduction factor accounting primarily for the capability of the structure to deform inelastically; the Code (P 100) requires $\psi = 0.3$ for shear-wall buildings with no more than 5 stories, $\psi = 0.25$ for shear-wall buildings with more than 5 stories and for one-bay multistory frames and $\psi = 0.20$ for multi-bay, multistory frames.

ε is a factor which accounts, at each mode of vibration, for the equivalence between the real structure and the SDOF system.

The load combinations considered in design are:

- "fundamental" load combination which is the summation of the design values of the dead loads (i.e. specified values multiplied by load factor) and the design values of the live loads;
- "special" load combinations (specified values of the dead loads plus reduced live loads plus one "exceptional" load).

3.1.1 Floor Weight

To determine the concentrated mass of one floor, the loads are considered as follows:

- dead loads (g) with their specified values (weight per unit volume for reinforced concrete is 25000 N/m³; for aerated concrete, the value 6900 N/m³ is taken);
- live loads (p) with their specified values factored by a reduction coefficient (n^d), to obtain the long term (or frequent) part of the live loads. In this case, the results are:

in the flat: $P_{1d} = P.n^d = 1500 \times 0.4 = 600$ N/m²
in corridors and stairs: $P_{1d} = P.n^d = 3000 \times 0.4 = 1200$ N/m²

The total load per unit area of the floor results:

- in the living areas:			
. dead weight of the slab (0.12 m thickness)		3000	N/m²
. flooring (5 cm equalizing pad + plastic carpet)		1230	N/m²
. live load		600	N/m²
	Total	4830	N/m²
- in the corridors and stairs:			
. dead weight of the slab (0.12 m thickness)		3000	N/m²
. flooring		1480	N/m²
. live load		1200	N/m²
	Total	5680	N/m²
Accordingly, the total story weight is:			
- living areas 4.83 x (3.45 + 2.70 + 3.15) x 4 x 5.85		1051	KN
- corridors 5.68 x (2.65 x 13.35)		201	KN
- R.C. shear walls 50.93 m³ x 25.00 KN/m³		1273	KN
- aerated concrete partitions (finishes included)		525	KN
	Total	3050	KN

3.1.2 Roof Floor Weight

The loads on the roof are considered as follows:

. dead weight of the slab 0.12 x 25000		3000	N/m²
. water driving concrete (mean value) 0.09 x 24000		2160	N/m²
. heat insulation of aerated concrete 0.10 x 6900		690	N/m²
. water insulation (3+4) x 20 + 2 x 20		180	N/m²
. snow 100 x 0.4		400	N/m²
	Total	6430	N/m²

The snow loading is computed from $q_{ES} = q_s \cdot n^d$, where n^d is the above mentioned reduction factor and q_s is the specified value of the snow load depending on the geographical site of the building.

Accordingly, the total roof weight is:

- roof 6.43 x 19.55 x 14.85	1866.7	KN
- shear walls 25.48 m³ x 25.00	636.9	KN
- attic 0.2 x 0.8 x 25 x 69.60	278.4	KN
Total	2782	KN

Total weight of the structure (8 floors plus roof): Q = 8 x 3050 + 2782= = 27,182 KN

3.1.3 The Design Seismic Load of the Structure

The seismic base shear for shear wall structures of more than 5 stories and for normal foundation soil is:

$$S = 2 \times 0.75 \times 0.25\, k_s Q = 0.38\, k_s Q,$$

where Q is the total gravity load and k_s is a factor accounting for the seismic intensity of site and seismic hazard exposure ($k_s = 0.2$ for Bucharest).

Consequently, for each separate direction of the building:

$$S = S_x = S_y = 0.38 \times 0.20 \times 27{,}182 = 2065 \text{ KN}$$

The vertical distribution of the seismic force is shown in Fig. 4, with:

$$q_{Ex} = q_{Ey} = \frac{2S}{H} = \frac{2 \times 2065}{24.75} = 166.87 \text{ KN/m},$$

E_i = design seismic force, concentrated at level "i"
q^i_{Ex}= distributed loading, equivalent to the system E_i

3.2 Distribution of the Design Seismic Force to Shear Walls

3.2.1 Geometrical and Stiffness Characteristics of Shear Walls

3.2.1.1 Shear Wall DT1 (Fig. 5)

To the left, the flange is edged by another wall at a distance l_c = 3.45 m. Hence, b_r = (3.45)/2 = 1.725 m and $b_f = b_w + \Delta b_r$ = 0.15 + 1.725 ≅ 1.875 m. Conditions b_f < H/5 =(24.75)/5 = 4.95 m and $b_f < h^r$ = 6.30 m are satisfied.

$$A = 0.15\ (0.30 + 0.70 + 5.85 + 1.875) = 1.309 \text{ m}^2$$

The position of the centroid is given by:

$$Y_G = \frac{0.15}{1.309} (0.30 \times 0.075 + 0.70 \times 0.225 + 5.85 \times 3.225 + 1.875 \times 6.225) = 3.50 \text{ m}$$

Hence, $I = \frac{0.30 \times 0.15^3}{12} + \frac{0.70 \times 0.15^3}{12} + \frac{0.15 \times 5.25^3}{12} + \frac{1.875 \times 0.15^3}{12} +$

$+ 0.15 \times 0.3 \times 3.425^2 + 0.15 \times 0.70 \times 3.275^2 + 0.15 \times 5.85 \times$
$\times 3.275^2 + 0.15 \times 1.875 \times 2.725^2 = 6.20 \text{ m}^4$

For I-shaped cross sections:

$$A_{w,v} = \frac{b_w\ h}{1.0} = \frac{0.15 \times 6.30}{1.0} = 0.945 \text{ m}^2$$

For Table A5 (Appendix A2) in the case of 9-story buildings, $\beta_s = 12.13$

With allowance make for the shear displacements, the equivalent moment of inertia is:

$$\bar{I}_{es} = \frac{I}{1 + \beta_s \frac{I}{A_{w,v} \cdot H^2}} = \frac{6.200}{1 + 12.13 \frac{6.200}{0.945 \times 24.75^2}} = 5.487 \text{ m}^4$$

3.2.1.2 Shear Wall DT2 (Fig. 6)

The transverse shear wall DT2 has a nonsymmetrical cross section with a single row of openings. The ratio Eb/Ec is taken equal to 0.25[(x)].

a) Data for the coupled wall DT2:

For wall DT2.1, $b_f = 0.95$ m; $A_1 = 0.15 \times (0.95 + 3.25) = 0.630 \text{ m}^2$

For T-shaped cross sections, the area considered to carry shear is:

$$A_{w,v_1} = \frac{b_w \cdot h}{1.1} = \frac{0.15 \times 3.40}{1.1} = 0.464 \text{ m}^2$$

Position of the centroid:

$$Y_{G_1} = \frac{0.15}{0.630} (0.80 \times 0.075 + 3.40 \times 1.70) = 1.39 \text{ m}$$

Hence, the moment of inertia is:

$$I_1 = \frac{0.15}{3} (1.39^3 + 2.01^3) + 0.15 \times 0.80 (1.39 - 0.075)^2 =$$
$$= 0.746 \text{ m}^4$$

(x) According to the Rumanian design provisions, the modulus of elasticity of the concrete in the coupling beams is reduced by a factor taking the values 0.15 and 0.5 to allow for a reduction in the rigidity of these elements due to cracking. Computations have to be made separately for both specified values leading to the values of the internal forces, the most unfavorable of which are to be used in the design of wall sections and of beam sections respectively. In the present example, a 0.25 ratio between the modulus of elasticity of concrete in the beams and that in the walls was considered.

For wall DT2.2, $b_f = b_w + \Delta b_l + \Delta b_r$. To the right the flange is bordered by another shear wall at a distance $l_r = 3.15$ m, whereas to the left it is bordered by a window opening at a distance $l_l = 0.90$ m; consequently,

$$b_r = 3.15/2 = 1.575 \text{ m}; \quad b_l = 0.90 \text{ m, hence}$$

$$b_f = 0.15 + 3.15/2 + 0.90 = 2.625 \text{ m}$$

The following conditions are met:

$$b_f = H/5 = 24.75/5 = 4.35 \text{ m};^{*} \qquad b_f < l_w = 6.15 \text{ m}$$

$$A_2 = 0.15\ (1.80 + 2.625) = 0.663 \text{ m}^2$$

$$A_{wv2} = \frac{0.15 \times 1.95}{1.1} = 0.266 \text{ m}^2$$

$$Y_{G2} = \frac{0.15}{0.663}(1.80 \times 1.05 + 2.625 \times 0.075) = 0.47 \text{ m. Hence,}$$

$$I_2 = \frac{0.15}{3}(0.47^3 + 1.48^3) + 0.15 \times 2.475 \times (0.47 - 0.075)^2 = $$
$$= 0.225 \text{ m}^4$$

b) Data for the coupling beam:

$$L = 6.15 - (1.39 + 0.47) = 4.29 \text{ m}$$

$$l_{on} = 0.80 \text{ m}; \ a_c = 0.35 \times 0.60 = 0.21 < 0.40 \text{ m}$$

$$l_d = 0.80 + 2 \times 0.21 = 1.22 \text{ m}$$

$$\Delta b_l = \Delta b_r = 0.15\ l_{on} = 0.15 \times 0.80 = 0.12 \text{ m and } h_b = 0.60 \text{ m}$$

$$b_f = b_w + \Delta b_l + \Delta b_r = 0.15 + 2 \times 0.12 = 0.39 \text{ m}$$

The moment of inertia of the coupling beam:

$$I_{bl} = c\,\frac{b_w h_r^3}{12} = 1.446\,\frac{0.15 \times 0.60^3}{12} = 0.0039 \text{ m}^4, \text{ where } c = 1.446$$

indicates the contribution of the slab to the moment of inertia of the beam section and is based on the conditions:

$$\frac{b_f}{b_w} = \frac{0.39}{0.15} = 2.6 \text{ and } \frac{h_f}{h_r} \doteq \frac{0.13}{0.60} = 0.217$$

$$A_{b,v} = \frac{0.15 \times 0.60}{1.1} = 0.082 \text{ m}^2$$

Shear displacements are allowed for by

$$\mu = \frac{1}{1 + \dfrac{30\ I_{bl}}{A_{b,v}\ l_d^2}} = \frac{1}{1 + \dfrac{30 \times 0.0039}{0.082 \times 1.22^2}} = 0.51$$

---

* The effective flange width of the wall b_f shall be less than 1/5 of the shear wall height h_w.

c) Equivalent moment of inertia (see Appendix A1):

The allowed axial deformability of the walls is based on the coefficient γ, computed from the expression A1.11 (see Appendix A1):

$$\gamma = 1 + \frac{I_1 + I_2}{L^2} \left(\frac{1}{A_1} + \frac{1}{A_z}\right) = 1 + \frac{0.746 + 0.225}{4.29^2} \times \left(\frac{1}{0.630} + \frac{1}{0.663}\right) =$$

$$= 1.163$$

The monolithic coefficient $\alpha^{(x)}$ is computed using (A1.10)

$$\alpha = \sqrt{12\ \gamma \frac{E_b}{E_w} \times \frac{1}{I_1 + I_2} \times \frac{\mu . I_{b1} \times L^2}{h_{sx} . l_d^3}} \times H, \text{ or}$$

$$\alpha = \sqrt{12 \times 1.163 \times 0.25 \times \frac{1}{0.746 + 0.225} \times \frac{0.51 \times 0.0039 \times 4.29^2}{2.75 \times 1.22^3}} \times$$

$$\times\ 24.75 = 4.00$$

$$\Sigma A_{w,v_j} = 0.464 + 0.266 = 0.730 \text{ m}^2$$

$$I_0 = \frac{\gamma}{\gamma - 1} (I_{cw1} + I_{cw2}) = \frac{1.163}{0.163} \times (0.746 + 0.225) = 6.921$$

From chart A2.2.2 for $\alpha = 4$ (see Appendix A2):

$$\psi_{s(0)} = 0.440;\ \psi_{s(0.778)} = 0.082$$

From Table A5 in Appendix A2, $Y_s^M = 0.128$ and $\beta_s = 12.13$

The moment of inertia of an equivalent cantilever wall having the same rigidity as the coupled wall is according to (A1.18):

$$I_{es} = \frac{I_1 + I_2}{\dfrac{\gamma - 1}{\gamma} + \dfrac{\psi_{s\ (a,0)} - \psi_{s\ (a,0.778)}}{\gamma . \alpha^2 . Y_s^M}} =$$

$$= \frac{0.746 + 0.225}{\dfrac{1.163 - 1}{1.163} + \dfrac{0.440 - 0.082}{1.163 \times 4.0^2 \times 0.128}} = 3.343$$

and allowing for the influence of the shear deformability (see A1.21):

$$\overline{I}_{es} = \frac{I_{es}}{1 + \beta_s \dfrac{I_{es}}{(\Sigma A_{wv}) . H^2}}$$

(x) Factor α is named by some authors "monolithic coefficient" as it synthetically reflects the influence of the openings on the deformability of the cross section and consequently of the entire wall: when $\alpha = 0$, the coupling beams are pin-jointed to the walls; when $\alpha \leq 1$ the coupled shear wall is termed as having "large openings" and can be included in the foregoing case; when $1 < \alpha < 10$, the coupled shear wall is termed as having "middle range openings" and frame-type analysis is required; when $\alpha \geq 10$, the coupled shear wall is termed as having "small openings" and the influence of the openings as to deformability may be neglected.

Hence, $$\bar{I}_{es} = \frac{3.343}{1 + 12.13\,\dfrac{3.43}{0.730 \times 24.75^2}} = 3.065\ m^4$$

3.2.1.3 Shear Wall DT3 (Fig. 7)

The flange is edged by shear walls situated at a distance:

$$l_l = l_r = 3.15\ m$$

$$\Delta b_l = \Delta b_r = 3.15/2 = 1.575\ m$$

$$b_f = b_w + \Delta b_l + \Delta b_r = 0.15 + 2 \times 1.575 = 3.30$$

The following conditions are met:

$$b_f \leq H/h = 24.75/5 = 4.95\ m$$

$$b_f \lessgtr h = 6.15\ m\text{, hence}$$

$$A = 0.15 \times (6.00 + 3.30) = 1.395\ m^2$$

$$Y_G = \frac{0.15}{1.395}\,(3.30 \times 0.075 + 6.00 \times 3.15) = 2.06\ m$$

$$I = \frac{0.15}{3}\,(4.09^3 + 2.06^3) + 0.15 \times 3.15 \times (2.06 - 0.075)^2 =$$
$$= 5.79\ m^4$$

$$A_{w,v} = \frac{0.15 \times 6.15}{1.1} = 0.84\ m^4$$

$$\bar{I}_{es} = \frac{I}{1 + \beta_s \dfrac{I}{A_{w,v}\ H^2}} = \frac{5.79}{1 + 12.13\,\dfrac{5.79}{0.84 \times 24.75^2}} = 5.033\ m^4$$

3.2.1.4 Longitudinal Shear Wall DL1 (Fig. 8)

The shear wall DL1 has two symmetrical rows of openings.

a) Data for the coupled walls:

Since all transverse shear walls intersecting the longitudinal shear wall are free at the other end, their design width for integer action with this wall is: $\Delta b_l = 10\ b = 10 \times 0.15 = 1.50$ m.

The breadth of the flange for both types of walls is:

$$b_f = b_w + \Delta b_l = 0.15 + 1.50 = 1.65\ m$$

The following conditions are met:

$b_f < H/5 = 4.95$ m and $b_f < h = 4.65$ and 8.55 m (for side wall and central wall, respectively).

For the coupled wall no. 1 (side wall):

$$A_l = 0.15 \times (4.65 + 2 \times 1.50) = 1.148\ m^2$$

$$A_{w,v_1} = \frac{0.15 \times 4.65}{1.1} = 0.634 \text{ m}^2$$

$$X_{G_1} = \frac{0.15}{1.148} (4.65 \times 2.825 + 1.50 \times 0.075 + 1.50 \times 3.675) = 2.15 \text{ m}$$

$$I_1 = \frac{0.15}{3} (2.15^3 + 2.50^3) + 0.15 \times 1.50 \times (2.15 - 0.075)^2 +$$

$$+ 0.15 \times 1.50 \times (3.675 - 2.15)^2 = 2.77 \text{ m}^4$$

For the coupled wall no. 2 (central wall):

$$A_2 = 0.15 \times (8.55 + 3 \times 1.50) = 1.957 \text{ m}^2$$

$$A_{w,v_2} = \frac{0.15 \times 8.55}{1.2} = 1.069 \text{ m}^2$$

$$I_2 = \frac{0.15 \times 8.55^3}{12} + 2 \times 1.5 \times 3.30^2 = 12.71 \text{ m}^4$$

b) Data for the coupling beams:

$$L = 4.65 + 0.90 + 8.55/2 - 2.15 = 7.675 \text{ m}$$

$$l_{on} = 0.90 \text{ m}; \; Q = 0.35 \times 0.60 = 0.21 \text{ m} < 6.40 \text{ m}$$

$$l_d = 0.90 + 2 \times 0.21 = 1.32 \text{ m}$$

$$\Delta b_l = \Delta b_r = 0.15 \; l_o = 0.15 \times 0.90 = 0.135 \text{ m}; h_b = 0.60 \text{ m}$$

$$b_f = b_w + \Delta b_l + \Delta b_r = 0.15 + 2 \times 0.135 = 0.42$$

$$I_b = 1.532 \frac{0.15 \times 0.60^3}{12} = 0.00414 \text{ m}^4$$

$$A_{bv} = \frac{0.15 \times 0.60}{1.1} = 0.082 \text{ m}^2$$

$$\mu = \frac{1}{1 + \dfrac{30 \; I_b}{A_{bv} \cdot l_d^2}} = \frac{1}{1 + \dfrac{30 \times 0.00414}{0.082 \times (1.32)^2}} = 0.535$$

c) Equivalent moment of inertia:

Due to symmetry, the axial force in the central wall is zero. Only the areas of the lateral walls are taken into account when computing γ. Hence, with $A_1 = A_z = 1.148$ m² and $L_{1z} = 19.65 - 2 \times 2.15 = 15.35$ m, according to formula (A1.11):

$$\gamma = 1 + \frac{\Sigma I_j}{\Sigma L_j^2} \times \frac{2}{A_1} = 1 + \frac{2 \times 2.77 + 12.71}{(15.35)^2} \times \frac{2}{1.148} = 1.135$$

Hence, the monolithic coefficient, using (A1.10) is:

$$\alpha = \sqrt{12 \quad \gamma \frac{E_b}{E_w} \quad \frac{1}{h_{sx} \; \Sigma I_j} \quad \Sigma \frac{\mu I_{blj} \; L^2_j}{l^3_{dj}}} \; H$$

$$\alpha = \sqrt{12 \times 1.135 \times 0.25 \times \frac{1}{2.75 \times (2 \times 2.77 + 12.71)}}$$

$$\times \frac{2 \times 0.535 \times 0.0414 \times 7.675^2}{1.32^3} \times 24.75 = 2.165$$

$$A_{w,v_j} = 2 \times 0.634 + 1.069 = 2.337 \text{ m}^2$$

From chart A2.2.2 $\psi_{s(0)} = 0.310$; $\psi_{s(0.778)} = 0.080$ and from Table A5 $Y_s^M =$ $= 0.1280$; $\beta_s = 12.13$.

Hence, the value I_{es} of the equivalent moment of inertia (using in principle formula A1.19) results in:

$$I_{es} = \frac{\Sigma I}{\frac{\gamma - 1}{\gamma} + \frac{\psi_{s(0)} - \psi_{s(0.778)}}{\gamma\ a^2\ Y_s^M}} = \frac{12.71 + 2\ (2.77)}{\frac{1.135 - 1}{1.135} + \frac{0.31 - 0.080}{1.135 \times 2.165^2 \times 0.128}} =$$

$$= 39.88 \text{ m}^4 \text{ and according to (A1.21),}$$

$$\overline{I}_{es} = \frac{39.88}{1 + 12.13 \times \frac{39.88}{2.337 \times 24.75^2}} = 29.81 \text{ m}^4$$

3.2.2 Distances from the Centroids of Individual Shear Walls to the Overall Centroid of Stiffness (see Fig. 9)

shear wall DT1:

with respect to the web axis of the wall:

$$X_G = \frac{1}{1.1309} (0.15 \times 0.30 \times 0.225 - 0.40 \times 0.15 \times 0.275 \times 0.15 \times$$
$$\times 0.94) = - 0.19 \text{ m}$$

with respect to 0_y:

$$X_{DT1} = 3.60 + 2.85 + 3.30 - 0.19 = 9.56 \text{ m}$$

shear wall DT2:

with respect to the web axis:

$$X_G = \frac{2.625 \times 0.15 \times 0.34}{1.293} = + 0.10 \text{ m}$$

with respect to 0_y:

$$X_{DT2} = 2.85 + 3.30 + 0.10 = 6.25 \text{ m}$$

shear wall DT2':

$$X_G = - 0.10 \text{ m}$$

$$X_{DT2'} = 3.30 - 0.10 = 3.20 \text{ m}$$

shear wall DL:

with respect to the web axis:

$$Y_G = \frac{7 \times 1.50 \times 0.15 \times 0.825}{4.253} = 0.30 \text{ m}$$

with respect to O_x:

$$Y_{DL} = 1.40 + 0.30 = 1.70 \text{ m}$$

3.2.3 Distribution Coefficients of the Horizontal Loading

Only the action of a transverse seismic force was considered.

Shear walls parallel to the direction (Y) of the horizontal loading:

Type	Number of elements (n)	$\bar{I}_{esy}$ (m^4)	$n\bar{I}_{esy}$ (m^4)	$\frac{\bar{I}_{esy}}{\Sigma I_{esy}}$	X (m)	$n\bar{I}_{esy}.X^2$ (m^6)	$\frac{\bar{I}_{esy}\ X}{\Sigma I_{esy}.X^2 + \Sigma I_{esy}.Y^2}$ (*)
DT1	4	5.487	21.95	0.097	9.56	2006.1	0.0188
DT2	4	3.065	12.26	0.054	6.25	478.9	0.0069
DT2'	4	3.065	12.26	0.054	3.20	125.5	0.0035
DT3	2	5.033	10.07	0.089	0	0	0
			Σ=56.54			Σ=2610.5	

Shear walls perpendicular to the direction (Y) of the horizontal loading:

Type	Number of elements	$\bar{I}_{esx}$ (m^4)	$n\bar{I}_{esx}$ (m^4)	Y (m)	$n\bar{I}_{esx}.Y^2$ (m^6)	$\frac{\bar{I}_{esx}.\ Y}{\Sigma\bar{I}_{es}.X^2 + \Sigma\bar{I}_{esx}.Y^2}$ (*)
DL	2	29.81	59.62	1.70	172.3	0.0182

(*) $(\bar{I}_{esx}\ Y^2 + \bar{I}_{esy}\ X^2) = 2610.5 + 172.3 = 2782.8\ m^6$.

3.2.4 Horizontal Forces Distributed to Shear Walls

For the transverse shear walls:

$$S_{yj} = S\left[\frac{\bar{I}_{esy}}{\Sigma\bar{I}_{esy}} \pm \frac{e_o \times \bar{I}_{esy}}{\Sigma\bar{I}_{esy}\ X^2 + \Sigma\bar{I}_{esx}\ Y^2}\right], \text{ where } e_o = 0.05B = 1.0 \text{ m}^{(x)}$$

(x) The earthquake design provisions in Rumania require the consideration of an additional eccentricity equal to 0.05 B for the seismic force (B is the dimension of the building normal to the direction of the seismic force). For a symmetrical building, the design eccentricity is equal to this additional eccentricity.

$$\text{DT1:}\quad S_Y = 2065\ (0.097 \pm 1.00 \times 0.0188) = \begin{bmatrix} 239.1\ \text{KN} \\ 161.5\ \text{KN} \end{bmatrix}$$

$$\text{DT2:}\quad S_Y = 2065\ (0.054 \pm 1.00 \times 0.0069) = \begin{bmatrix} 125.8\ \text{KN} \\ 97.2\ \text{KN} \end{bmatrix}$$

$$\text{DT2':}\quad S_Y = 2065\ (0.054 \pm 1.00 \times 0.0035) = \begin{bmatrix} 118.7\ \text{KN} \\ 104.3\ \text{KN} \end{bmatrix}$$

$$\text{DT3:}\quad S_Y = 2065 \times 0.089 = 183.8\ \text{KN}$$

For the longitudinal shear walls:

$$S_x = \pm\, S \frac{e_o\, Y\, \bar{I}_{esx}}{\Sigma(\bar{I}_{esx}\, Y^2 + \bar{I}_{esy}\, X^2)}$$

$$S_x = \pm\, 2065 \times 1.00 \times 0.0182 = 37.6\ \text{KN}$$

Fig. 9 presents the distribution of the horizontal loading considered.

3.3 Computation of the Bending Moments in the Structural Elements

The computations of internal forces (bending moments) for shear wall (DT3) and shear wall (DT2) are presented herein.

3.3.1 Shear Wall DT3

The computation of the bending moments is made using relation (A1.8):

$$M_s(\xi) = V_{os}\,.\,H\,.\,\frac{2 - 3\xi + \xi^2}{3} = V_{os}\,.\,H\,.\,K_s^M\,(\xi)$$

$$= 183.8 \times 24.75 \times K_s^M\,(\xi) = 4549\,K_s^M\,(\xi)$$

The bending moments diagram is presented in Fig. 10.

3.3.2 Shear Wall DT2

The values of coefficient φ_s can be taken from chart A2.2.4 and the values of of coefficients ψ_s from chart A2.2.2 (case $\alpha = 4.00$).

In Table A7 the values of K_s^M for a 9-story building (n=9) are presented.

a) Bending moments in the equivalent semi-structure (x) (see A1):
   - in coupling beams:

$$M_{bj} = \frac{V_{os}\, h_{sx}}{2\gamma}\,.\,\varphi_s = \frac{125.8 \times 2.75}{2 \times 1.163}\,.\,\varphi_s = 148.3\,\varphi_s$$

(x) The design method used herein implies the reduction of both nonsymmetrically coupled walls and of coupled walls with several rows of openings to a symmetrical coupled wall with a single row of openings.

- in coupled walls:

$$M_{jm} = \frac{V_{os}\,H}{2}\left(K_s^M - \frac{\psi_s}{\gamma}\right) = \frac{125.8 \times 24.75}{2}\left(K_s^M - \frac{\psi_s}{1.163}\right) =$$

$$= 1556.8\; K_s^M - 1338.6\; \psi_s$$

Systematic computations are presented in the following Table:

Level	$\xi = \frac{X}{24.75}$	φ_s $(\alpha=4.0)$	M_{bj} $=1483\varphi_s$	K_s^M	$1556.8 \times K_s^M$	ψ_s $(\alpha=4.0)$	$1338.6\psi_s$	M_{jm} $=1556.3K_s^M$ $-1338.6\psi_s$
0	0	0	0	0.667	1038.4	0.440	589.0	449.4
1	0.111	0.315	46.7	0.556	856.6	0.430	576.6	290.0
2	0.222	0.482	71.5	0.448	697.5	0.378	506.0	191.5
3	0.333	0.566	83.9	0.346	538.7	0.320	428.4	110.3
4	0.444	0.578	85.7	0.251	390.8	0.255	341.3	49.5
5	0.555	0.545	80.8	0.161	250.6	0.196	262.4	11.8
6	0.666	0.492	73.0	0.099	154.1	0.138	184.7	-30.6
7	0.777	0.420	62.3	0.046	71.6	0.082	109.8	-38.2
8	0.888	0.370	54.9	0.012	18.7	0.034	45.5	-26.8
9	1.000	0.340	50.4	0	0	0	0	0

b) Bending moments in the real structure:

- in coupled walls:

$$\frac{2I_1}{I_1+I_2} = \frac{2 \times 0.746}{0.971} = 1.537; \quad \frac{I_2}{I_1+I_2} = \frac{2 \times 0.225}{0.971} = 0.643$$

$$M_{1j} = 1.537\; M_j;\; M_{2j} = 0.463\; M_j$$

Level	M_j (KN m)	M_{1j}	M_{2j}
0	449.4	690.7	208.1
1	290.0	445.7	134.3
2	191.5	294.3	88.7
3	110.3	169.5	51.1
4	49.5	79.1	22.9
5	-11.8	-18.1	-5.5
6	-30.6	-47.0	-14.2
7	38.2	-58.7	-17.7
8	-26.8	-41.2	-12.4
9	0	0	0

Axial force in shear wall DT1

From one story:

Shear wall (0.3 x 0.3 ÷ 0.15 x 0.40 + 5.85 x 0.15 + + 0.15 x 1.725) x 2.75 x 25000	=	88430 N
autoclaved aerated concrete [(5.85 + (1.32 + 1.32)] 0.15 x 2.75 x 6900	=	24160 N
floor (1200 + 1200 + 3000) x (1.725 x 5.85 + 1.325 x 1.875)	=	67900 N
		180490 N

Total for nine stories: 180.5 x 9 = 1610 KN

Axial force in coupled wall M1

From one story:

Shear wall 0.630 x 2.75 x 25000	=	43310 N
autoclaved aerated concrete 0.3 x 2.75 x 2.15 x 6900	=	12240 N
live load + flooring + slab 0.8 x 1500 + 1200 + 3000 = 5400 Nm² $\times \frac{3.45 \times 2.70}{2} \times 3.65$	=	60600 N
beam 0.25 x 0.30 x 3.07 x 25000	=	5750 N
		121900 N

Total for nine stories: 121.9 x 9 = 1097 KN

Axial force in coupled wall M2

From one story:

shear wall 0.663 x 2.75 x 25000	=	45580 N
floor 5400x[(1.325 x 2.62) + 2.20 x (1.575 + 1.35)]	=	53490 N
		99070 N

Total for nine stories: 99 x 9 = 891 KN

3.4 Design and Reinforcement in Shear Walls DT1 and DT2

3.4.1 Material Properties

1. Concrete grade C16

$$f_{ck} = 16 MP_a;\ f_{cd} = \frac{f_{ck}}{\gamma} = \frac{16}{1.5} = 10.67\ MP_a \text{ and } \tau_{Rd} = 0.22\ MP_a$$

The design compressive strength for concrete in cast-in-situ walls is:

- for web region (with thickness $b_w < 300$ mm):
  $\overline{f}_{cd} = 0.75 \times 0.85\ f_{cd} = 0.75 \times 0.85 \times 10.67 = 6.8\ MP_a$
- for vertical boundary elements:
  $\overline{f}_{cd} = 0.75 \times 0.85 \times f_{cd} = 0.75 \times 0.85 \times 10.67 = 6.8\ MP_a$ for $b < 300$ mm
  and $\overline{f}_{cd} = 0.85 \times 0.85 \times f_{cd} = 0.85 \times 0.85 \times 10.67 = 7.7\ MP_a$ for $b \geq 300$ mm

Due to the unfavorable effect of vertical casting of concrete over heights of more than 1.50 m, supplementary reduction factors (0.75 and 0.85 for thickness <300 mm and ≥300 mm, respectively) have been used.

2. Steel

S 400 for longitudinal reinforcement in vertical boundary elements:

$$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{400}{1.15} = 348 \ MP_a$$

S 220 for stirrups and web reinforcement:

$$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{220}{1.15} = 191 \ MP_a$$

3.4.2 Design of Reinforcement in Cantilever Shear Wall DT1

The internal forces induced by the seismic loading in the lower cross section of a boundary shear wall are:

- the design axial force Nd = 1610 KN
- the desing shear force V_d = 239.1 KN, and
- the design bending moment Md = 3945 KN m

a) Design of the equivalent cross section (Fig. 5)

In order to take into account the unfavorable effect of eccentricities inherently resulting from the construction technique of the walls as well as of the nonhomogeneous structure of the concrete, a design thickness may be used by reducing the actual web thickness δ_w.

- design web width:

$$b_a = b_w - \delta$$

- for boundary shear walls:

$\delta = \frac{b}{10}$ or at least 30 mm. Hence, with $\delta = \frac{150}{10} = 15$ mm or less than 30 mm, hence, δ = 30 mm; consequently,

$$b_a = 150 - 30 = 120 \text{ mm}$$

$$A_{fr\ actual} = (300 - 150) \times 300 + 400 \times 150 = 45 \times 10^3 + 60 \times 10^3 = 0.105 \times 10^6 \text{ mm}^2$$

$$A_{f1} = \Sigma A_{f\ actual} \cdot \frac{f_{cd(f)}}{f_{cd(w)}} = 45 \times 10^3 \times \frac{7.7}{6.8} + 60 \times 10^3 \frac{6.8}{6.8} = 0.1109 \times 10^6 \text{ mm}^2$$

$$A_{f2} = A_{f2\ actual} = (1875 - 150) \times 150 = 0.2587 \times 10^6 \text{ mm}^2$$

$$\frac{A_{f1}}{A_w} = \frac{0.1109 \times 10^6}{120 \times 6300} = 0.145$$

$$\frac{A_{f2}}{A_w} = \frac{0.2587 \times 10^6}{120 \times 6300} = 0.342$$

b) Preliminary check of concrete cross section

For first-level-of-safety structures the concrete cross section must comply with equation:

$$\frac{N_d}{A_w f_{cd}} \leq 2\frac{A_f}{A_w} + 0.35 = \frac{2Af_1}{A_w} + 0.35, \text{ or}$$

$$\frac{1610.000}{120 \times 6300 \times 6.8} = 0.313 \leq 2 \times 0.145 + 0.35 = 0.64$$

This condition is aimed at limiting axial stress value in the wall section, thus ensuring a satisfactory curvature ductility factor of $\mu_\varphi = 6$.

The condition is more severe than for columns of moment-resisting frames, knowing that the seismic energy dissipation in shear walls is achieved mainly by local yielding of the base actions. By observing the condition, a mean displacement ductility factor of 3, is ensured or a mean curvature ductility (structural) factor of 6, corresponding to the shear walls with total height-to-section length ratios between 3 and 6.

Checking of dimensions for the vertical boundary elements:

$b_f > 250$ mm

$b_{f1} = 300 + 400 = 700 > 250$ mm

$b_{f2} = 1875$ mm > 250 mm

consequently, $b_f > 2$ as $b_w = 2 \times 150 = 300$ mm

In flexural capacity computation one must take into account the contribution of longitudinal web reinforcement. For that reason a preliminary design of vertical reinforcement is necessary.

c) Design of boundary element reinforcement

The minimum reinforcement ratio in vertical boundary elements of the lower third, steel grade S 400 and the first level of safety is 0.09% (Table 1).

Table 1

Position of floor with respect to total shear wall height	1st level of safety		2nd level of safety	
	S 220	S 400	S 220	S 400
within the lower third (zone A)	0.12	0.09	0.09	0.07
within the upper two thirds (zone B)	0.08	0.06	0.07	0.05

Therefore,

$$A_{s\,min} = \frac{\rho_{min}}{100} b_w h = \frac{0.09}{100} \times 150 \times 6300 = 850 \text{ mm}^2$$

$$\alpha_{min} = \frac{\rho_{min}}{100} \frac{f_{yd}}{f_{cd}} = \frac{0.09}{100} \times \frac{348}{6.8} = 0.0460$$

$$n = \frac{N_d}{b_a\, h\, \bar{f}_{cd}} = \frac{1610 \times 10^3}{120 \times 6300 \times 6.8} = 0.313$$

If primarily considering $\rho_w = 0.2\%$ (S 220 steel), the ultimate flexural strength can be computed using design charts given in Appendix 2.

For A_{f1} in compression zone, with $\frac{A_{f1}}{A_w} = 0.145$,

$$m_1 = 0.200$$

$$M_{1r} = m_1 . b_a . h^2 . \bar{f}_{cd} = 0.200 \times 120 \times 6300^2 \times 6.8 = 6.477 \times 10^3 \text{ KNm}$$

For A_{f2} in compression zone with $\frac{A_{f2}}{A_w} = 0.342$,

$$m_2 = 0.235$$

$$M_{2r} = m_2 . b_a . h^2 . \bar{f}_{cd} = 0.235 \times 120 \times 6300^2 \times 6.8 = 7.611 \times 10^3 \text{ KNm}$$

The reinforcement of the boundary element remains unchanged over the height of the wall. Though the reinforcement is kept constant, flexural strength varies over height due to the variation of the axial force.

$$n = 0;\ \rho_{min} = 0.06;\ \alpha_{min} = \frac{0.06}{1.00} \times \frac{348}{6.8} = 0.030$$

For $\rho_w = 0.2\%$ and $\frac{A_f}{A_w} = 0.145$, one obtains $m = 0.080$; hence,

$$M_r = m b_a \times h^2 \times \bar{f}_{cd} = 0.080 \times 120 \times 6300^2 \times 6.8 = 2591 \text{ KNm}$$

d) Shear force design. Calculation of the web reinforcement

1. Horizontal web reinforcement

The design shear force V_{du} is associated with the ultimate maximum flexural strength $M_{r\,max}$.

$$V_{du} = \gamma V_d,$$

$$\text{where, } \gamma = \frac{M_{r\,max}}{M_d} = \frac{7611}{3945} = 1.929;\ \text{hence,}$$

$$V_{du} = 1.929 \times 239.1 = 461.3 \text{ KN}$$

Assuming the failure crack angle is 45°, the design condition for web is:

$$V_{du} \leq V_{cd} + 0.8\, A_{sh}\, f_{yd} \frac{h}{h_{sx}}\ ;\ \text{hence,}$$

$$A_{sh} = \frac{V_{du} - V_{cd}}{0.8 f_{yd}} \cdot \frac{h_{sx}}{h}.$$

In the plastic zones(x), located at the shear wall base, the shear carried by the concrete is:

$$V_{cd} = 2 b_a\, h\, \tau_{Rd} = 2 \times 120 \times 6300 \times 0.22 = 332.64 \text{ KN}$$

(x) Beyond the plastic zones, as defined by the Rumanian Code, the concrete is considered less affected by load reversals. Therefore, the concrete is assumed capable of carrying a greater amount of shear, namely, $V_{cd} = 2.5\, b_a\, h\, \tau_{Rd}$

$$A_{sh} = \frac{461.3 \times 10^3 - 332.6 \times 10^3}{0.8 \times 191} \times \frac{2750}{6300} = 367.5 \text{ mm}^2\text{, or distributed}$$

over the story height 367/2.75 = 133.6 mm²/m

For the first level of safety, the minimum reinforcement ratio is given in Table 2 with reinforcement ratios for horizontal bars 0.25% in zone A and 0.20% in zone B.

Table 2

Level of safety	Minimum reinforcement ratio for	
	Horizontal bars	Vertical bars
1	0.25%	0.20%
2	0.20%	0.15%

Using Table 2, the minimum reinforcement ratio in zone A (lower third of shear wall height) will be:

$$A_{sh\ min} = \frac{0.25}{100} \times 150 \times 1000 = 375 \text{ mm}^2/\text{m} > 133.6 \text{ mm}^2/\text{m}.$$

Selecting 2 Φ 8/250 mm,

$$A_{sh\ actual} = \frac{1000}{250} \times 2 \times 50.2 = 401.6 \text{ mm}^2/\text{m} > 375 \text{ mm}^2/\text{m}.$$

2. Vertical web reinforcement

The area of vertical web reinforcement can be derived from the equilibrium condition written for a horizontal construction joint as,

$$V_{du} \leq 0.8\ A_{sv}\ f_{yd} + 0.2\ N_d\text{; hence,}$$

$$A_{sv} = \frac{V_{du} - 0.2N_d}{0.8\ f_{yd}} = \frac{461300 - 0.2 \times 1610000}{0.8 \times 191} = 911.6 \text{ mm}^2\text{, reflecting}$$

a reinforceing percentage for the web reinforcement of:

911.6 mm²/6.30 = 144.7 mm²/m.

The minimum vertical reinforcement ratio given in Table 2 for the first level of safety is:

$\rho_{v\ min}$ = 0.20%; hence, the minimum required is:

$$A_{sv\ min} = \frac{0.20}{100} \times 150 \times 1000 = 300 \text{ mm}^2/\text{m} > 144.7 \text{ mm}^2/\text{m}.$$

However, selecting 2 Φ 8/250 mm,

$$A_{sv} = \frac{1000}{250} \times 2 \times 50.2 = 401.6 \text{ mm}^2/\text{m}.$$

Over the heights of the first and last stories, the web reinforcement has to meet the minimum reinforcement requirements imposed by the existence of forces due to restrained deflections, i.e. shrinkage for the first story and restrained temperature expansion of the roof floor for the last story.

3.4.3 Design of Reinforcement in Shear Wall DT2

The design values of bending moments and forces have been computed in the first part of the design example.

3.4.3.1 Design of Reinforcement in Coupled Wall DT2.1

In the lowest cross section the internal forces are:

- design axial force N_d:

$$N_d = N \pm \Sigma V_{\text{coupled beam}}$$

$$N_{d\,max} = 1057 + 344 = 1401 \text{ KN}$$

$$N_{d\,min} = 1057 - 344 = 713 \text{ KN}$$

- design shear force V_d:

$$V_d = V_{dDT2} \frac{I_1}{I_1 + I_2} = 125.8 \frac{0.746}{0.746 + 0.225} = 96.65 \text{ KN}$$

- design bending moment M_d:

$$M_d = 690.7 \text{ KN m}$$

a) Design cross section of the coupled wall

- design web width b_a:

$b_a = b - \delta$; furthermore, $\delta = b/15$ or at least 20 mm; hence,

$\delta = 150/15 = 10$ mm; therefore, $\delta = 20$ mm and

$b_a = 150 - 20 = 130$ mm. The length is $h = 3400$ mm.

Further, $A_{f1\,real} = A_{f1'} = (950 - 150) \times 150 = 120 \times 10^3 \text{ mm}^2$ and $A_{f2} = 0$.

Hence, $\frac{A_{f1}}{A_w} = \frac{120000}{130 \times 3400} = 0.271$ and $\frac{A_{f2}}{A_w} = 0$.

b) Preliminary check of the concrete cross section

For the first level of safety the following condition must be fulfilled:

$$\frac{N_{d\,max}}{A_w\, f_{cd}} \leq \frac{2\, A_f}{A_w} + 0.35 = \frac{2\, A_{f1}}{A_w} + 0.35$$

$$\frac{1401000}{130 \times 3400 \times 6.8} = 0.466 < 2 \times 0.271 + 0.35 = 0.892\text{; furthermore,}$$

$$\frac{N_{d\,min}}{A_w\, f_{cd}} \leq \frac{2A_{f2}}{A_w} + 0.35$$

$$\frac{713000}{130 \times 3400 \times 6.8} = 0.237 < 2 \times 0 + 0.35 = 0.35$$

The relative axial force is related to the ratio A_f/A_w corresponding to the flange in the compressed zone.

c) Design of reinforcement in the boundary element

The longitudinal reinforcement can be computed using the design charts in Appendix 2, with a preliminary value of ρ_w = 0.2% (ratio of total area of vertical uniformly distributed reinforcement and total cross area of shear-wall web).

$$n_{max} = \frac{N_{d\ max}}{b_a\ h\ f_{cd}} = \frac{1401000}{130 \times 3400 \times 6.8} = 0.467$$

$$n_{min} = \frac{N_{d\ min}}{b_a\ h\ f_{cd}} = \frac{713000}{130 \times 3400 \times 6.8} = 0.237$$

$$m = \frac{M_d}{b_a\ h^2\ f_{cd}} = \frac{690.7 \times 10^6}{130 \times 3400^2 \times 6.8} = 0.0675$$

$$\frac{A_{f1}}{A_w} = 0.271;\ \frac{A_{f2}}{A_w} = 0.0$$

Considering the load combinations and the cross sections, the values of coefficient α are less than the minimum values given in the design charts, namely:

$$n = 0.467;\ m = 0.0675;\ \frac{A_f}{A_w} = 0.271 \Rightarrow \alpha \leq 0.01$$

$$n = 0.237;\ m = 0.0675;\ \frac{A_f}{A_w} = 0.0 \Rightarrow \alpha \leq 0.01$$

The minimum nominal reinforcement is given by ρ_{min} = 0.09.

$$A_{s\ min} = \frac{\rho_{min}}{100}\ b\ h = \frac{0.09}{100} \times 150 \times 3400 = 459\ \text{mm}^2$$

The bars selected are 6 Φ10 (for the free end); hence,

$$A_{s\ actual} = 6 \times 78.5 = 471\ \text{mm}^2$$

The actual flexural capacity is computed by aid of charts in Appendix 2, considering:

$$\alpha = \rho \frac{f_{yd}}{f_{cd}} = \frac{471}{130 \times 3400} \times \frac{348}{6.8} = 0.0545,$$

$$\rho_w = 0.20\%;\ \frac{A_f}{A_w} = 0;\ n = 0.237 \text{ gives } m = 0.160; \text{ hence,}$$

$$M_{r\ max} = m\ b_a\ h^2\ \overline{f}_{cd} = 0.160 \times 130 \times 3400^2 \times 6.8 = 1635\ \text{KN m}.$$

d) Shear force design. Calculation of the web reinforcement

The maximum design shear force V_{du} is:

$$V_{du} = \gamma\ V_d \text{ with } 3 \geq \gamma = \frac{M_{r\ max}}{M_d} \geq 1.50. \text{ Considering } \rho_w = 0.20\%;$$

$$\frac{A_f}{A_w} = 0.271;\ n = 0.467 \text{ and } \alpha = 0.0545 \text{ gives } m = 0.280.$$

Hence, $M_{r\ max} = 0.280 \times 130 \times 3400^2 \times 6.8 = 2861$ KN m and $\gamma = \frac{2861}{690.7} = 4.14$.
With the allowable maximum γ of 3, the design shear force becomes,

$V_{du} = 3 \times 96.65 = 289.95$ KN <477 KN $= 6\ b_a\ l_w\ \tau_{Rd}$, thus meeting the requirements.

1. Horizontal web reinforcement:

Whereas, $A_{sh} = \frac{V_{du} - V_{cd}}{0.8\ f_{yd}} \frac{h_{sx}}{h}$ and

$V_{cd} = 2\ b_a\ h\ \tau_{Rd} = 2 \times 130 \times 3400 \times 0.22 = 194480$ N, A_{sh} becomes:

$$A_{sh} = \frac{289950 - 194480}{0.8 \times 191} \times \frac{2750}{3400} = 505.4 \text{ mm}^2$$

Hence, the area of horizontal reinforcement per height unit is $\frac{505 \text{ mm}^2}{2.75 \text{ m}} =$ 183.8 mm^2/m.

The minimum horizontal reinforcement ratio required, as given in Table 2, is 0.25%. Consequently,

$A_{sh\ min} = \frac{0.25}{100} \times 150 \times 100 = 375 \text{ mm}^2/\text{m} > 183.8 \text{ mm}^2/\text{m}$. As a result, 2Φ8/250 mm are selected, with:

$$A_{sh\ actual} = \frac{1000}{250} \times 2 \times 50.2 = 401.6 \text{ mm}^2/\text{m}.$$

2. Vertical web reinforcement:

For the maximum value of shear force V_{du},

$$A_{sv} = \frac{V_{du} - 0.2\ N_d}{0.8\ f_{yd}} = \frac{289950 - 0.2 \times 1401000}{0.8 \times 191} = 63.8 \text{ mm}^2$$

For the minimum value of the shear force V_{du} associated with the minimum flexural strength $M_{r\ min}$,

$M_{r\ min} = 1635$ KN, $\gamma = \frac{1635}{690.7} = 2.36$. Consequently,

$V_{du} = 2.36 \times 96.65 = 228.78$ KN; hence,

$$A_{sv} = \frac{V_{du} - 0.2\ N_{d\ min}}{0.8\ f_{yd}} = \frac{228780 - 0.2 \times 71300}{0.8 \times 191} = 564 \text{ mm}^2$$

The required area of vertical reinforcement per length unit is $\frac{564 \text{ mm}^2}{3.40 \text{ m}} =$ 166 mm^2/m.

For the first level of safety, the minimum vertical reinforcement ratio, as given in Table 2, is 0.20%; consequently:

$$A_{sv\ min} = \frac{0.20}{100} \times 150 \times 100 = 300 \text{ mm}^2/\text{m} > 166 \text{ mm}^2/\text{m}.$$

The bars selected are 2Φ8/250 mm with

$$A_{sv\ actual} = \frac{1000}{250} \times 2 \times 50.2 = 401.6 \text{ mm}^2/\text{m} > 300 \text{ mm}^2/\text{m}.$$

3.4.3.2 Design of Reinforcement in Coupled Wall DT2.2

The internal forces at the base of the wall are:

$$N_d = N \pm \Sigma V_{coupling\ beams}$$

$$N_{d\ max} = 867 + 344 = 1211\ KN$$

$$N_{d\ min} = 867 - 344 = 513\ KN$$

$$V_d = V_{d\ DT2} \frac{I_2}{I_1 + I_2} = 125 \frac{0.225}{0.746 + 0.225} = 29.15\ KN \text{ and}$$

$$M_d = 208.1\ KN\ m.$$

a) The design cross section of the wall

The following pertinent data have to be considered:

$$b_a = b_w - \delta$$

$$\delta = \frac{b_w}{15} \geqq 20\ mm,\ \delta = \frac{150}{15} = 10\ mm \Rightarrow \delta = 20\ mm$$

$$b_a = 150 - 20 = 130\ mm$$

$$h = 1950\ mm$$

$$A_{f1} = 0$$

$$A_{f2\ actual} = A_{f1} = (1575 + 900 - 150) \times 150 = 0.348 \times 10^6\ mm^2$$

$$\frac{A_{f1}}{A_w} = 0$$

$$\frac{A_{f2}}{A_w} = \frac{0.348 \times 10^6}{130 \times 1950} = 1.37$$

b) Preliminary check of the concrete cross section

$$\frac{N_d}{A_w\ f_{cd}} \leqq 2\frac{A_f}{A_w} + 0.35$$

For $N_{d\ max}$, $\frac{1211000}{130 \times 1950 \times 6.8} = 0.702 < 2 \times 1.37 + 0.35 = 3.09$

For $N_{d\ min}$, $\frac{513000}{130 \times 1950 \times 6.8} = 0.297 < 0.35$

c) Design of reinforcement in the boundary elements

The longitudinal reinforcement is computed by use of the design charts in Appendix 2 with a preliminary value of $\rho_w = 0.20\%$.

For position of the flange in the compressed zone:

With $M_d = 208.1$ KNm, $N_d = 1211$, $\frac{A_f}{A_w} = 1.37$ and $\rho_w = 0.20\%$,

$$m = \frac{208.1 \times 10^6}{130 \times 1950 \times 6.8} = 0.0619; \quad n = 0.702$$

According to the design charts, α would be less than 0.01; hence, the design reinforcement will be increased to the minimum nominal value:

$$\rho = \alpha \frac{\bar{f}_{cd}}{f_{yd}} \cdot 100 = 0.01 \times \frac{6.8}{348} \times 100 = 0.019\% < 0.09\% = \rho_{min}$$

For position of the flange in the tension zone:

With $M_d = 208.1$ KN, $N_d = 513$ KN, $\frac{A_f}{A_w} = 0$, $\rho_w = 0.20\%$, $m = 0.0619$ and $n = 0.297$, the design charts give α as less than 0.01, i.e.

$$\rho < \rho_{min} = 0.09\%$$

The reinforcement in the wall is computed from

$$A_s = \frac{\rho_{min}}{100} \times 150 \times 1950 = 263.25 \text{ mm}^2$$

The actual reinforcement consists of 6Φ10 in the end without flange and 6Φ10 + 6Φ8 in the flanged end of the wall. The corresponding reinforcement areas and ratios are:

$$A_{s1} = 6 \times 78.5 = 471.0 \text{ mm}^2; \quad \rho_1 = \frac{471}{130 \times 1950} \times 100 = 0.186\%$$

$$A_{s2} = 4 \times 78.5 + 6 \times 50.2 = 615.2 \text{ mm}^2; \quad \rho_2 = \frac{615}{130 \times 1950} \times 100 =$$
$$= 0.242\%.$$

d) Shear force design. Computation of web reinforcement.

The design shear force V_{du} is:

$$V_{du} = \gamma \, V_d \text{ with } 3 \geq \gamma = \frac{M_r}{M_d} \geq 1.5$$

$M_{r\,max}$ are M_{r1} and M_{r2} with the values of M_{r1} and M_{r2} computed by means of the design charts; hence,

for $\frac{A_{f1}}{A_w} = 0$, $n = 0.297$, $\rho_w = 0.2$ and $\alpha = \frac{\rho_1}{100} \frac{f_{yd}}{f_{cd}} = \frac{0.242}{100} \times \frac{348}{6.8} = 0.123$,

m_1 from the charts is $m_1 = 0.216$. Similarly,

for $\frac{A_f}{A_w} = 1.37$, $n = 0.702$, $\rho_w = 0.20$ and $\alpha = \frac{0.186}{100} \times \frac{348}{5.1} = 0.095$,

m_2 from the charts becomes $m_2 = 0.470$.

Thus, the computation of the design shear force gives M_r:

$$M_r = 0.470 \times 130 \times 1950^2 \times 6.8 = 1580 \text{ KN m.}$$

With, $\gamma = \frac{1580}{208.1} = 7.60$, or as it is limited to $\gamma = 3.0$, V_{du} becomes,

$V_{du} = 3.0 \times 29.15 = 87.45 \text{ KN} < 273.8 \text{ KN} = 6\, b_a\, h\, \overline{f}_{cd}$ thus meeting the condition.

1. Horizontal web reinforcement

whereas, $A_{sh} = \frac{V_{du} - V_{cd}}{0.8\, f_{yd}} \cdot \frac{h_{sx}}{h}$ and

$V_{cd} = 2\, b_a\, h\, \tau_{Rd} = 2 \times 130 \times 1950 \times 0.22 = 111540$ N, V_{cd} is larger than $V_{du} = 87.45$ KN. This would give A_{sh} less than 0.

Consequently, the required area of horizontal reinforcement per unit height has to meet the minimum values given in Table 2 or:

$$A_{sh\ min} = \frac{\rho_{min}}{100}\, b_w\, h = \frac{0.25}{100} \times 150 \times 1000 = 375 \text{ mm}^2\text{/m.}$$

The reinforcement selected is 2Φ8/250 mm, giving:

$$A_{sh\ actual} = \frac{1000}{250} \times 2 \times 50.2 = 401.6 \text{ mm}^2\text{/m} > 375 \text{ mm}^2\text{/m.}$$

2. Vertical web reinforcement

whereas, $A_{sv} = \frac{V_{du} - 0.2\, N_d}{0.8\, f_{yd}} = \frac{87450 - 0.2 \times 513000}{0.8 \times 191}$ is less than 0, the vertical web reinforcement is governed also by the minimum reinforcement ratio defined in Table 2, or, for the first level of safety, $\rho_{v\ min} = 0.20\%$.

Thus, $A_{sv\ min} = \frac{0.20}{100} \times 150 \times 1000 = 300 \text{ mm}^2\text{/m}$

The reinforcing bars selected are 2Φ8/250 mm, resulting in:

$$A_{sv\ actual} = \frac{1000}{250} \times 2 \times 50.2 = 401.6 \text{ mm}^2\text{/m} > 300 \text{ mm}^2\text{/m.}$$

3.4.3.3 Design of a Coupling Beam

Due to the fact that the internal forces induced in the coupling beams by the horizontal loading do not vary significantly over the building height, the reinforcement of these elements is kept constant.

The design bending moment is:

$M_d = 85.7 \times \frac{610}{2145} = 24.37$ KN and the design shear force is

$V_d = 48.73$ KN

a) The design dimensions of the coupling beams are:

$l_d = 900$ mm, $h_b = 600$ mm, and $b_w = 150$ mm.

b) Design of the longitudinal reinforcement

$$A_s = \frac{M_d}{(d-c)\ f_{yd}} = \frac{24.37 \times 10^6}{(575-25) \times 348} = 127.3 \text{ mm}^2$$

The maximum reinforcement ratio allowed is given by the relation:

$$\rho_{max} = 480 \frac{\tau_{Rd}}{f_{yd}} \frac{l_d}{h_b} = 480 \times \frac{0.22}{348} \times \frac{900}{600} = 0.454\%; \text{ hence,}$$

$$A_{s\ max} = \frac{\rho_{max}}{100} b_w\ h_b = \frac{0.454}{100} \times 150 \times 600 = 409 \text{ mm}^2$$

The reinforcement selected is 2Φ12 or A_s = 226.2 mm²

c) Design of the stirrups

The design shear force associated with M_{du} is:

$$V_{du} = 2 \times 0.85\ A_s\ f_{yd} \frac{h_b}{l_d} = 2 \times 0.85 \times 226.2 \times 348 \times \frac{600}{900} = 89.2 \text{ KN}$$

The value of V_{du} must be compliant with condition:

$$V_{du} \leqq 15\ b_w\ h_b\ \tau_{Rd} = 15 \times 150 \times 600 \times 0.22 = 297 \text{ KN} > 89.2 \text{ KN}$$

The transverse steel area will be:

$$A_{sw} = \frac{V_{du}}{n\ 0.8\ f_{yd} \frac{h_b}{S_h}} = \frac{89.200}{2 \times 0.8 \times 191 \times 600/100} = 45.8 \text{ mm}^2 < 78.5 \text{ mm}^2$$

The condition is similar to the condition applying to wall web and intends to reduce the sensibility to brittle failure under shear action.

The design equations for the transverse reinforcement is derived assuming that the entire shear force is taken by the hoops.

The angle of inclination of the critical shear crack is assumed to be 45°. Considering the advanced degree of degradation occuring during severe seismic actions, the contribution of concrete in carrying shear is neglected.

For stirrups Φ10: A_{sw} = 78.5 mm²> 45.8 mm²

d) Design of intermediate longitudinal bars

The minimum reinforcement of the lateral sides will be in accordance with:

ρ_{min} = 0.20% for the first level of safety

$$A_{sl} = \frac{0.20}{100} b_w\ h_b = \frac{0.20}{100} \times 150 \times 550 = 165 \text{ mm}^2$$

The reinforcing bars are 4Φ8, with $A_{sl\ actual}$ = 4 x 50.2 = 200.8 mm²> 165 mm²

The detailing of reinforcement is shown in Fig. 13.

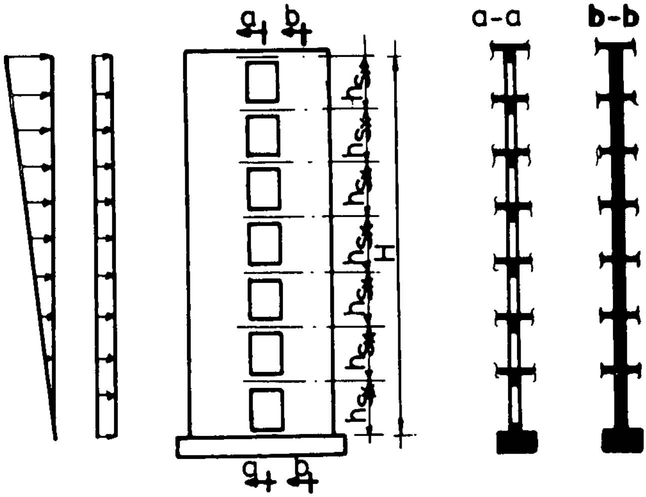


Fig. 1

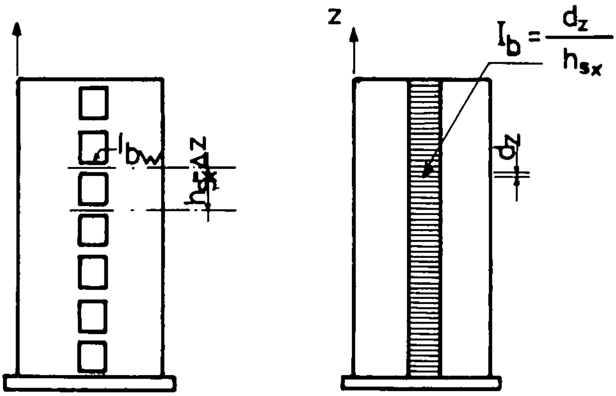


Fig. 2

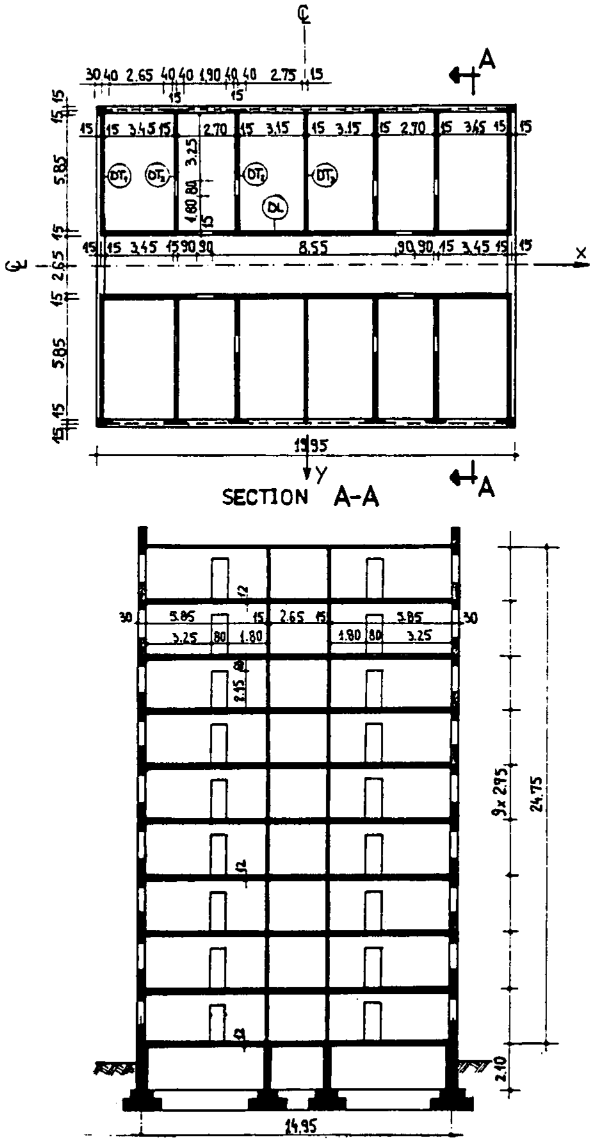


Fig. 3 - Structural layout and vertical section

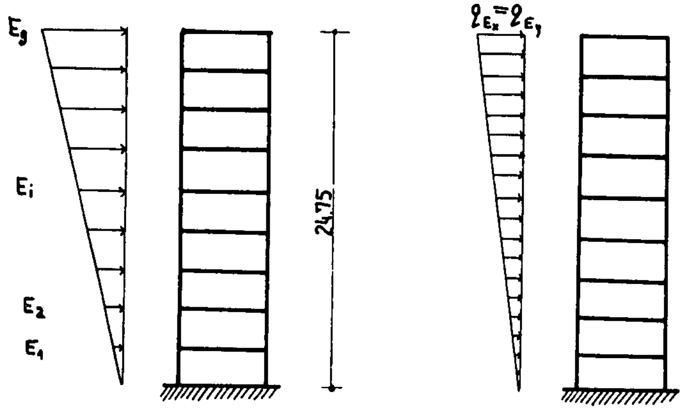


Fig. 4 - Distribution of seismic force

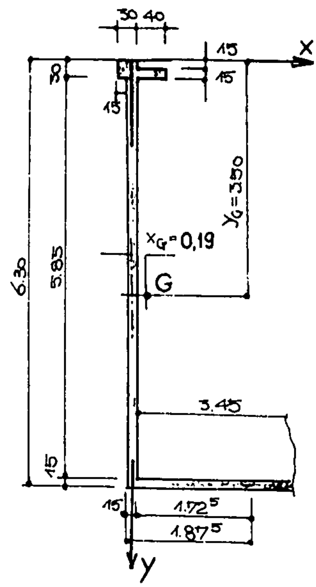


Fig. 5 - Shear wall DT1

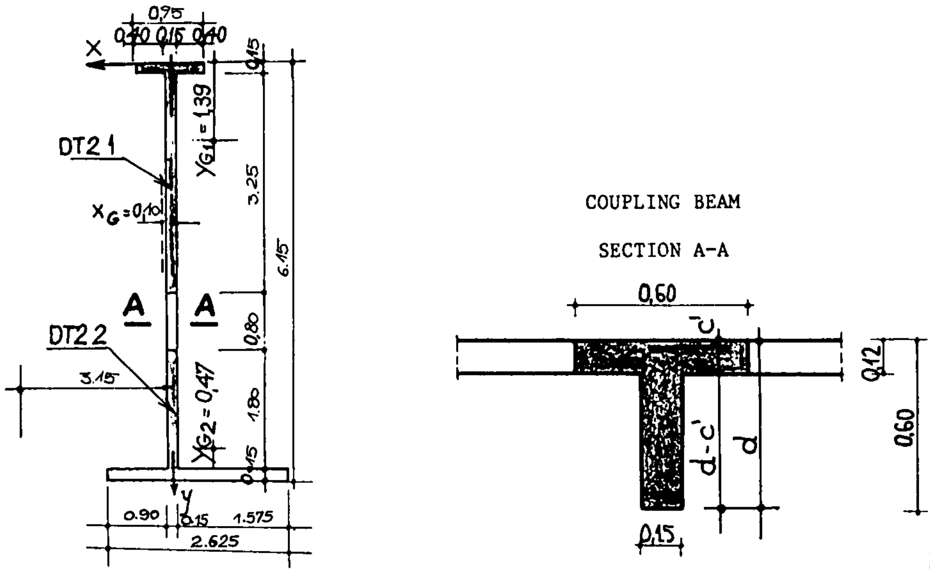


Fig. 6 - Shear wall DT2

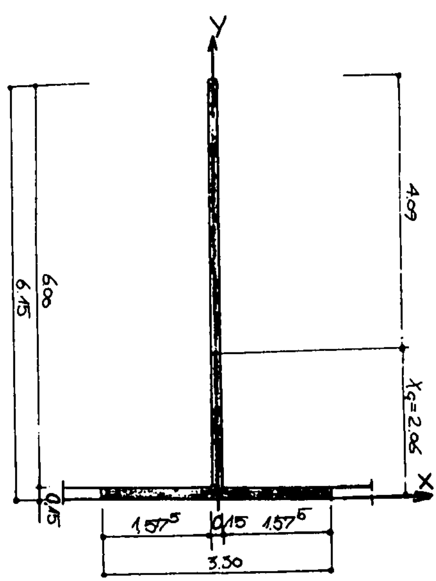
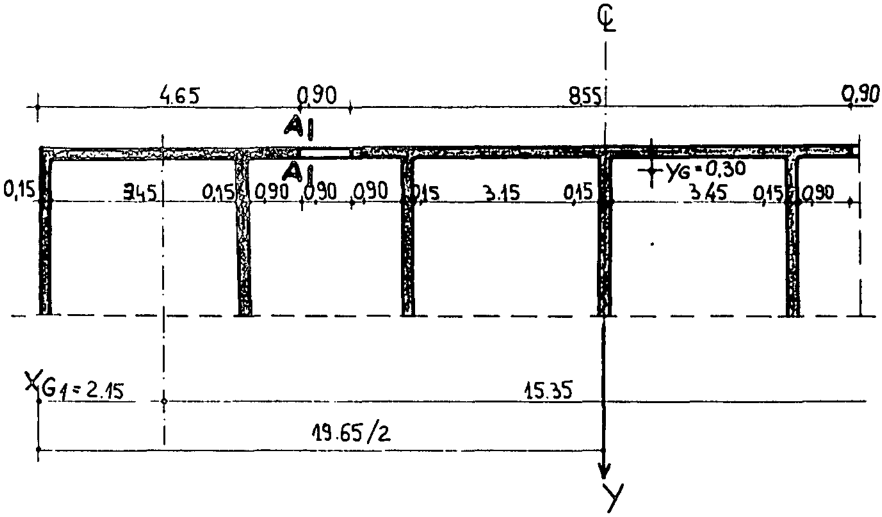
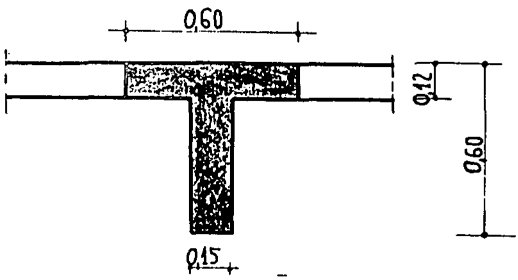


Fig. 7 - Shear wall DT3



a. longitudinal shear wall, cross section



b. coupling beam cross section A-A

Fig. 8 Longitudinal shear wall DL1

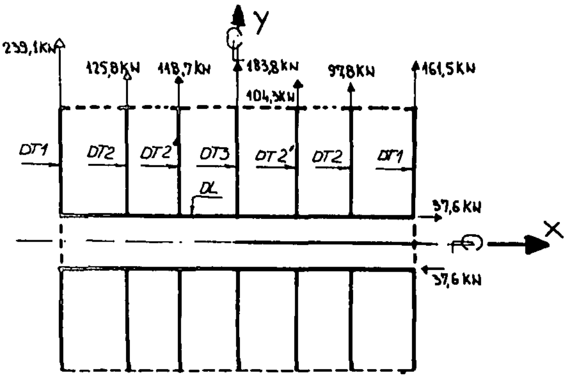


Fig. 9 - Distribution of horizontal loading

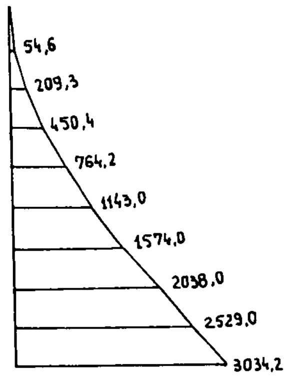


Fig. 10 - Bending moments diagram

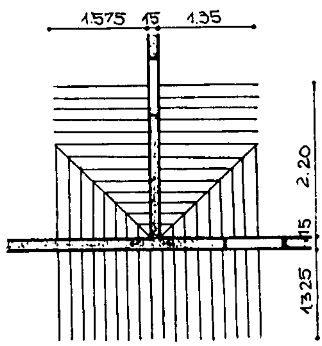


Fig. 11 - Tributary area for wall M2

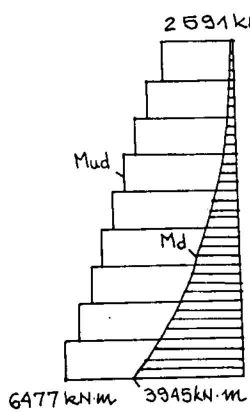


Fig. 12 - Ultimate moments diagram

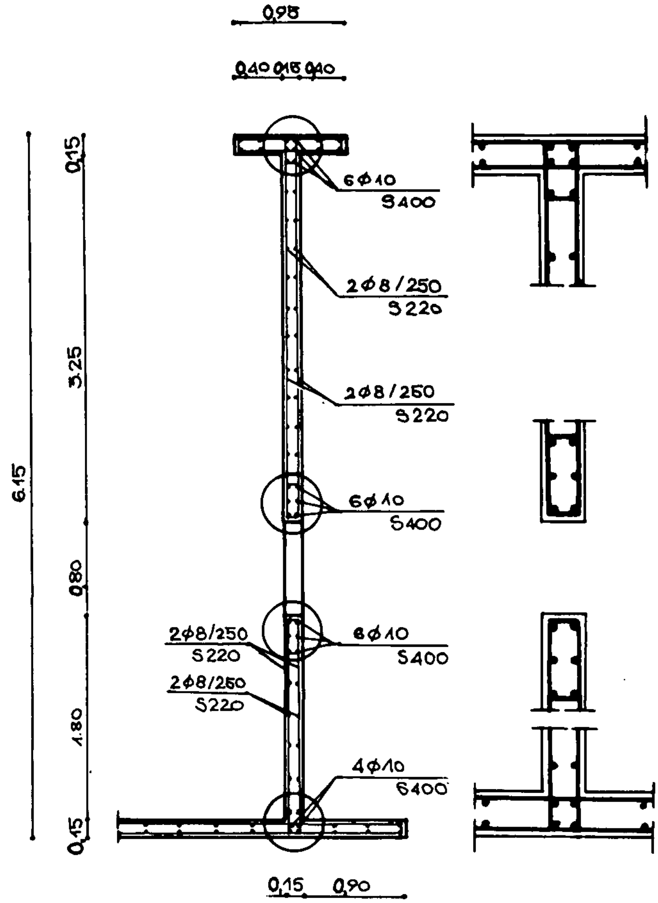


Fig. 13 - Detailing of reinforcement,
Shear wall DT2

A P P E N D I X A1 - EQUIVALENT CONTINUOUS STRUCTURE METHOD

A1.1 Basic Assumptions

A1.1.1 Appendix 1 presents a simplified method for shear wall analysis under lateral forces, either wind or seismic action.

The method permits calculation of internal forces in shear wall structures of maximum of ten floors. The structure must be "monotonous" over its height, thereby meaning:

- story height is the same for all floors;
- wall thickness is constant over the building height;
- openings in wall are of the same dimensions and superposed in a vertical regular pattern;
- coupling beams keep the same cross section at each floor.

The following assumptions are also made:

- lateral forces distribution over the building height is considered to be uniform for wind loading and triangular for seismic loading (Fig. 1);
- deformed axis shape of all walls in the structure are alike; in this case, for distributing lateral loads among the shear walls of the structure, the equality of lateral displacement may be imposed to a single level.

The basic principle of the method is to replace the actual coupling beams on the coupled shear wall by an infinite number of infinitesimal coupling beams, uniformly distributed over the height (Fig. 2). An equivalent stiffness has to be kept.

This enables to reduce the algebraic system of linear equations to a single differential equation with constant coefficients. This way, the analysis can be organized by using graphs of the internal forces functions. The present section of the manual reproduces design charts from "Analyse statique et dynamique des contreventements par consoles elementaires" [2], published in "Annales de l'Institut technique du bâtiment et des travaux publics", Feb. 1972.

A1.1.2 The method presented herein is recommended for design of building with at least 6 floors. For lesser numbers of floors, it leads to certain differences as compared to more exact methods.

This method can be used in the analysis of buildings with more than seven floors if taking into account that only the first mode of vibration is considered satisfactory.

A1.1.3 Due to the limited character of this appendix, its content is limited to:

- definitions, notations and equations which permit the use of design charts;
- design charts for wall stiffness and internal forces calculation;
- design examples.

A1.2 Internal Forces Computation in Shear Wall Structures

Taking into account the assumptions contained in section A1.1, the computation of the internal forces for shear wall structures subjected to lateral actions has to take the following steps:

A1.2.1 Calculation of equivalent moment of inertia for all shear walls.

By definition, the equivalent moment of inertia of either cantilever or coupled shear wall is the moment of inertia of a cantilever, which under the action of the same loading experiences the same flexural horizontal displacement at a determined level, as the total displacement (i.e. displacement produced by bending moments, shear forces and, for coupled shear walls, axial forces) of the actual shear wall.

If we impose the displacement equality condition at the top level of the structure, we obtain an error, always in the same direction (Fig. A1.1c).

It would be advisable to impose the displacement equality condition at a level where the error would be at minimum (Fig. A1.1d). Taking into account that in most cases this level is about 0.8 H, it is recommended to calculate the equivalent moment of inertia at the floor level nearest to 0.8 H. Equivalent moments of inertia must be calculated both for uniform and for triangular loading.

A1.2.2 Wind forces have code-specified values. Their resultant must be distributed to each shear wall proportionately to its stiffness (equivalent moment of inertia). Wind loading is considered uniformly distributed over the (vertical) shear wall height.

A1.2.3 Seismic forces are obtained using aseismic code provisions.

In order to determine the dynamic coefficient and the equivalence coefficient (necessary for the seismic coefficient), we must compute the story drifts.

Calculation of story drifts is made for equivalent shear wall with a moment of inertia equal to the sum of the equivalent moment of inertia for all shear walls of the structure.

Each shear wall will carry a part of the total seismic load proportionately to its stiffness (equivalent moment of inertia) including the general torsion moment.

Seismic loading is considered triangularly distributed over the shear wall height.

A1.3 Cantilever Shear Walls. Calculation of Displacements and Internal Forces

A1.3.1 Horizontal displacements

For cantilever shear walls loaded with lateral forces, the displacement at a relative height ξ (Fig. A1.2) produced by bending moment and shear force is:

- For uniformly distributed loading:

$$y_v(\xi) = \frac{V_{ov} H^3}{24 E_c I} \xi^2 (\xi^2 - 4\xi + 6) + \frac{V_{ov} H}{G A_{wv}} \xi \left(1 - \frac{\xi}{2}\right) =$$

$$= \frac{V_{ov} H^3}{E_c I} Y_v^M(\xi) + \frac{V_{ov} H}{G A_{wv}} Y_v^V(\xi) \qquad (1)$$

The values for function $Y_v^M(\xi)$ and $Y_v^V(\xi)$ for buildings with 7 to 11 stories are given in Tables A1 and A3, respectively.

- For triangularly distributed loading:

$$y_s(\xi) = \frac{V_{os} H^3}{60 E_c I} \xi^2 (\xi^2 - 10\xi + 20) + \frac{V_{os} H}{G A_{wv}} \xi \left(1 - \frac{\xi^2}{3}\right) =$$

$$= \frac{V_{os} H^3}{E_c I} Y_s^M(\xi) + \frac{V_{os} H}{G A_{wv}} Y_s^V(\xi) \qquad (2)$$

The values for functions $Y_s^M(\xi)$ and $Y_s^V(\xi)$ for buildings with 7 to 11 stories are given in Tables A2 and A4, respectively.

The shear modulus of elasticity for concrete is $G = 0.4\ E$.

In a shear wall structure, stiffness of a cantilever shear wall is characterized by the equivalent moment of inertia. By definition the equivalent moment of inertia is the moment of inertia of a cantilever which for the same loading gives, at a specific level, the same flexural displacement as the total displacement (from flexure and shearing) of the actual shear wall. It may be calculated as follows:

- For wind loading:

$$\bar{I}_{ev} = \frac{I}{1 + \dfrac{Y_v^V(\xi)}{0.4\ Y_v^M(\xi)} \cdot \dfrac{I}{A_{wv} H^2}} \qquad (3)$$

- For seismic loading:

$$\bar{I}_{es} = \frac{I}{1 + \dfrac{Y_s^V(\xi)}{0.4\ Y_s^M(\xi)} \cdot \dfrac{I}{A_{wv} H^2}} \qquad (4)$$

$Y_v^M(\xi)$ and $Y_v^V(\xi)$ respectively $Y_s^M(\xi)$ and $Y_s^V(\xi)$ must be computed at the same level where the displacement equality is imposed. When this level is the nearest floor to a $0.8\ H_w$ level, the values for the displacement functions noted $\bar{Y}_v^M$, $\bar{Y}_s^M$, $\bar{Y}_v^V$, and $\bar{Y}_s^V$ and coefficients β_v and β_s are in Tab.A5.

$$\beta_v = \frac{\bar{Y}_v^V}{0.4\ \bar{Y}_v^M} \qquad \beta_s = \frac{\bar{Y}_s^V}{0.4\ \bar{Y}_s^M} \qquad (5)$$

A1.3.2 Calculation of bending moments and shear forces

For uniformly distributed loading the bending moment and the shear force in the horizontal section at a relative height, are:

$$M_v\ (\xi) = V_{ov}\ H \frac{(1-\xi)^2}{2} = V_{ov}\ H\ K_v^M\ (\xi) \tag{6}$$

$$V_v\ (\xi) = V_{ov}\ (1-\xi) = V_{ov}\ K_v^V\ (\xi) \tag{7}$$

For triangularly distributed loading, the bending moment and the shear force are:

$$M_s\ (\xi) = V_{os}\ H \frac{2 - 3\xi + \xi^2}{3} = V_{os}\ H\ K_s^M\ (\xi) \tag{8}$$

$$V_s\ (\xi) = V_{os}\ (1-\xi^2) = V_{os}\ K_s^V\ (\xi) \tag{9}$$

The values for K_V^M, K_S^M, K_v^V, and K_s^V for 7-to-11-floor buildings are given in Tables A6, A7, A8 and A9.

A1.4 Coupled Shear Walls. Calculation of Displacements. Equivalent Moments of Inertia and Internal Forces (Fig. A1.3)

A1.4.1 Monolithic behavior index

The influence of openings on overall behavior of shear walls under horizontal loading actions depends on the value of the monolithic behavior index defined by equation (10):

$$\alpha = \sqrt{12\ \gamma \frac{E_b}{E_w} \cdot \frac{1}{h_{sx} \sum_1^z I_j} \cdot \sum_1^{z-1} \frac{\mu\ I_{bj}\ L_j^2}{l_{dj}^3}}\ H \tag{10}$$

Where γ = factor accounting for axial deformability influence (eq.11):

$$\gamma = 1 + \frac{\sum_1^z I_j}{\left(\sum_1^{z-1} L_j\right)^2} \cdot \left(\frac{1}{A_1} + \frac{1}{A_z}\right) \tag{11}$$

In the case of shear walls with a single row of openings, equations (10) and (11) become:

- for nonsymmetrical disposition of openings:

$$\alpha = \sqrt{12\ \gamma \frac{E_b}{E_w}\ \frac{I}{I_1 + I_2}\ \frac{\mu\ I_b\ L^2}{h_{sx}\ l_d^3}}\ H \tag{10'}$$

where $$\gamma = 1 + \frac{I_1 + I_2}{L^2}\ \left(\frac{1}{A_1} + \frac{1}{A_2}\right) \tag{11'}$$

- for symmetrical disposition of openings:

$$\alpha = \sqrt{6\ \gamma \frac{E_b}{E_w} \cdot \frac{I_b}{I} \cdot \frac{\mu\ L^2}{h_{sx}\ l_d^3}}\ H \tag{10''}$$

where $\gamma = 1 + \frac{4\ I}{A\ L^2}$ (11'')

Note: In eq. (10), (10') and (10''), if one takes into account the shear deformations in beams, an equivalent moment of inertia $I_{be,j}$ should be used (eq. 12):

$$I_{be,j} = \frac{I_{bj}}{1 + \frac{30\ I_{bj}}{A_{b,v}\ l_{dj}^2}} \tag{12}$$

A1.4.2 Calculation of Displacements

Shear wall displacement at a level of relative height (ξ) produced by bending moments and axial forces may be calculated as follows:

a) for wind loading

$$y_v\ (\xi) = \frac{V_{ov}\ H^3}{\alpha^2\ \gamma\ E_w \sum_1^z I_j} \left[\psi_v\ (\alpha,o) - \psi_v\ (\alpha,\xi)\right] +$$

$$+ \frac{\gamma - 1}{\gamma} \frac{V_{ov}\ H^3}{E_w \sum_1^z I_j} \cdot Y_v^M\ (\xi) \tag{13}$$

or,

$$y_v\ (\xi) = \frac{V_{ov}\ H^3}{\alpha^2\ (\gamma-1)\ E_w\ I_o} \left[\psi_v\ (\alpha,o) - \psi_v\ (\alpha,\xi)\right] +$$

$$+ \frac{V_{ov}\ H^3}{E_w\ I_o}\ Y_v^M\ (\xi) \tag{13'}$$

where $I_o = \frac{\gamma}{\gamma - 1} \sum_1^z I_j$ (14)

and represents the moment of inertia of the shear wall as a whole and is considered as non-deformable.

$$\psi_v\ (\alpha,\xi) = \frac{(1-\xi)^2}{2} + \frac{1}{\alpha^2} - \frac{sh\ \alpha(1-\xi)}{\alpha\ ch\ \alpha} - \frac{ch\ \alpha\xi}{\alpha^2\ ch\ \alpha} \tag{15}$$

Graphs of the function $\psi(v,\xi)$ are plotted in chart A2.2.1.

b) for seismic loading:

$$y_s\ (\xi) = \frac{V_{os}\ H^3}{\alpha^2\ \gamma\ E_w \sum\limits_1^z I_j} \left[\psi_s\ (\alpha,o) - \psi_s\ (\alpha,\xi)\right] +$$

$$+ \frac{V_{ov}\ H^3}{E_w \sum\limits_1^z I_j}\ \frac{\gamma - 1}{\gamma} Y_s^M\ (\xi) \tag{16}$$

or

$$y_s(\xi) = \frac{V_{os}\ H^3}{\alpha(\ \gamma - 1)\ E_w\ I_b} \left[\psi_s\ (\alpha,o) - \psi_s\ (\alpha,\xi)\right] +$$

$$+ \frac{\gamma - 1}{\gamma}\ \frac{V_{os}\ H^3}{E_w\ I_b}\ Y_s^M\ (\xi) \tag{16'}$$

where

$$\psi(\alpha,\xi) = \frac{1}{3}(\ 2 - 3\xi\ + \xi^2) + \frac{2}{\alpha^2}\ (\ \xi - \frac{ch\alpha\xi}{ch\alpha}) -$$

$$- \frac{\alpha^2 - 2}{\alpha^3} \cdot \frac{sh\alpha\ (1 - \xi)}{ch\ \alpha} \tag{17}$$

Graphs of the functions $\psi_s\ (\alpha,\xi)$ are plotted in chart A2.2.2.

For the calculation of shear displacement, the same equations as for cantilever walls can be used. In these equations the effective area in shearing action should be $\sum\limits_1^z A_{wvj}$.

A1.4.3 Equivalent moments of inertia

The equivalent moments of inertia of coupled shear walls can be obtained by equalizing the total displacement of the coupled shear wall (i.e. displacement induced by bending moments and axial force) with the flexural displacement of a cantilever shear wall, produced by the same loading at the floor located nearest to the level of 0.8 H (Fig. A1.1).

- for uniform (wind) loading:

$$I_{ev} = \frac{\sum\limits_1^z I_j}{\dfrac{\gamma - 1}{\gamma} + \dfrac{\psi_v\ (\alpha,o) - \psi_v\ (\alpha,\xi)}{\gamma\ \alpha^2\ \overline{Y}_v^M}} \tag{18}$$

or

$$I_{ev} = \frac{I_b}{1 + \dfrac{\psi_v\ (\alpha,o) - \psi_v\ (\alpha,\xi)}{(\gamma - 1)\alpha^2\ \overline{Y}_v^M}} \tag{18'}$$

- for triangular (seismic) loading:

$$I_{es} = \frac{\sum_1^z I_j}{\frac{\gamma - 1}{\gamma} + \frac{\psi_s(\alpha,o) - \psi_s(\alpha,\xi)}{\gamma\ \alpha^2\ \overline{Y}_s^M}} \tag{19}$$

or,
$$I_{es} = \frac{I_b}{1 + \frac{\psi_s(\alpha,o) - \psi_s(\alpha,\xi)}{(\gamma - 1)\alpha^2\ \overline{Y}_s^M}} \tag{19'}$$

Y_v^M and Y_s^M are the values of $Y_v^M(\xi)$ and $Y_s^M(\xi)$ computed at the floor level as mentioned above; their values are given in Table A5.

If the effect of axial deformability of cantilever is neglected ($\gamma = 1$), eq. (18) and (19) become:

$$I_{ev} = \frac{\sum_1^z I_j}{\psi_v(\alpha,o) - \psi_v(\alpha,\xi)}\ \alpha^2\ \overline{Y}_v^M \quad \text{and respectively,} \tag{18''}$$

$$I_{es} = \frac{\sum_1^z I_j}{\psi_s(\alpha,o) - \psi_s(\alpha,\xi)}\ \alpha^2\ \overline{Y}_s^M \tag{19''}$$

In determining the spatial distribution of lateral loads among structural elements, equivalent values $\overline{I}_{ev}$ or $\overline{I}_{es}$ should be used for the moments of inertia. They are obtained by equalizing the displacement of the actual shear wall from the effect of all internal forces (bending moments, axial forces and shear forces) and the flexural displacement of an equivalent cantilever. The values of $\overline{I}_{ev}$ and $\overline{I}_{es}$ are given in eq. (20) and (21) as follows:

$$\overline{I}_{ev} = \frac{I_{ev}}{1 + \beta_v \frac{I_{ev}}{(\sum_1^z A_{wvj})\ H^2}} \tag{20}$$

$$\overline{I}_{es} = \frac{I_{es}}{1 + \beta_v \frac{I_{es}}{(\sum^z A_{wvj})\ H^2}} \tag{21}$$

The coefficients $\overline{Y}$ and β are defined at A1.3.1 and their values can be found in Table A5.

A1.4.4 Calculation of internal forces

Shear forces in coupling beams and bending moments in walls are obtained from the following equations:

a) uniform lateral loading

The shear force in coupling beams of the first row of openings, respectively of the second row etc. at the level with relative height ξ are:

$$
\begin{aligned}
V_{1v}(\xi) &= F_{ov1}\ \varphi_v(\alpha,\xi)\\
V_{2v}(\xi) &= F_{ov2}\ \varphi_v(\alpha,\xi)\\
V_{z-1}(\xi) &= F_{ov_{z-1}}\ \varphi_v(\alpha,\xi)
\end{aligned}
\tag{22}
$$

where F_{ov1}, F_{ov2} ... $F_{ov_{z-1}}$ are the resultant shear forces transmitted by the coupling beam of the first floor, considering the overall coupled shear-wall section as non-deformable.

$$
\begin{aligned}
F_{ov1} &= \frac{V_{ov}\,h_{sx}}{\gamma}\ \frac{I_{b1}\,L_1}{l^3_{d_1}\sum\limits_1^{z-1}\dfrac{I_{bj}\,L_j^2}{l^3_{dj}}}\\
F_{ov2} &= \frac{V_{ov}\,h_{sx}}{\gamma}\ \frac{I_{b2}\,L_2}{l^3_{d_2}\sum\limits_1^{z-1}\dfrac{I_{bj}\,L_j^2}{l^3_{dj}}}\\
F_{ov_{(z-1)}} &= \frac{V_{ov}\,h_{sx}}{\gamma}\ \frac{I_{b\,(z-1)}\,L_{(z-1)}}{l^3_{d\,(z-1)}\sum\limits_1^{z-1}\dfrac{I_{bj}\,L_j}{l^3_{dj}}}
\end{aligned}
\tag{23}
$$

and
$$\varphi_v(\alpha,\xi) = 1 - \frac{\alpha\ \mathrm{ch}\ \alpha(1-\xi) - \mathrm{sh}\ (\alpha,\xi)}{\alpha\ \mathrm{ch}\ \alpha} \tag{24}$$

Graphs of functions $\varphi_v(\alpha,\xi)$ are plotted in chart A2.2.3.

Flexural moments in cantilever are:

$$
\begin{aligned}
M_1(\xi) &= \frac{I_1}{\sum\limits_1^{z} I_j}\ V_{ov}\ H\left[K_v^M(\xi) - \frac{1}{\gamma}\psi_v(\alpha,\xi)\right]\\
M_2(\xi) &= \frac{I_2}{\sum\limits_1^{z} I_j}\ V_{ov}\ H\left[K_v^M(\xi) - \frac{1}{\gamma}\psi_v(\alpha,\xi)\right]\\
M_z(\xi) &= \frac{I_z}{\sum\limits_1^{z} I_j}\ V_{ov}\ H\left[K_v^M(\xi) - \frac{1}{\gamma}\psi_v(\alpha,\xi)\right]
\end{aligned}
\tag{25}
$$

In the case of a single row of openings, the equation for shear forces in coupling beams becomes:

$$V_b\ (\xi) = F_{ov}\ \varphi_v\ (\alpha,\xi) \tag{22'}$$

where $F_{ov} = V_{ov} \dfrac{h_{sx}}{\gamma\ L}$

b) In the case of triangular lateral loading, the equations are of the same type. They can be otained by replacing in eq. (22), (23), (24) and (25) V_{ov} , F_{ov}, $\varphi_v\ (\alpha,\xi)$, $K_v^M\ (\xi)$ and $\psi_v\ (\alpha,\xi)$ with V_{os}, F_{os}, $\varphi_s\ (\alpha,\xi)$ $K_s^M\ (\xi)$, and $\psi_s\ (\alpha,\xi)$,

where $$\varphi_s\ (\alpha,\xi) = 1 - \alpha^2 - \frac{2}{\alpha^2} - \frac{(\alpha^2 - 2)\ \mathrm{ch}\ [\alpha(1 - \xi)] + 2\alpha\ \ \mathrm{sh}\ \ \alpha}{\alpha^2\ \mathrm{ch}\ \ \alpha} \tag{26}$$

A1.5 Supplementary Notations for Appendix A1

i	index of current floor; $i = 1,n$; n = total number of floors
I	moment of inertia of shear wall cross section
I_b	moment of inertia of coupling beam cross section
I_{be}	equivalent moment of inertia of coupling beam cross section
I_{ev}	equivalent moment of inertia of either cantilever or coupled shear wall cross section for wind design
I_{es}	the same, for seismic design
$\bar{I}_{ev}$	equivalent moment of inertia of coupled shear wall cross section taking into account the effect of bending moment, axial forces and shear forces for wind design
$\bar{I}_{es}$	the same, for seismic design
j	index of coupled walls and coupling beam of coupled shear walls; $j = 1,z$ for the coupled walls and $j = 1,z-1$ for coupling beams
M_v	bending moment in shear wall cross section induced by wind forces
M_s	the same, for seismic forces
q_v	wind lateral load
q_s	seismic lateral load
V_{ov}	$q_v\, l_w$ = resultant of wind lateral loading over shear wall height
V_{os}	$1/2\, q_s\, l_w$ = resultant of seismic lateral loading over shear wall height
V_v	shear force in a shear-wall cross section from wind loading
V_s	the same, for seismic loading
$K_v^M(\xi)$	bending moment function in cantilever shear wall due to wind forces
$K_s^M(\xi)$	the same, for seismic forces
$K_v^V(\xi)$	shear force function in cantilever shear walls due to wind forces
$K_s^V(\xi)$	the same, due to seismic forces
y_v	horizontal displacement of shear wall caused by wind forces
y_s	the same, for seismic forces
$Y_v^M(\xi)$	horizontal displacements function of shear wall due to wind bending moments
$Y_s^M(\xi)$	the same, for seismic forces
$Y_v^V(\xi)$	horizontal displacements function of shear wall due to wind shear forces
$Y_s^V(\xi)$	the same, for seismic forces
α	monolithic behavior index

γ	factor accounting for deformation influence
ξ	relative height of current floor
$\psi_v(\alpha,\xi)$	axial forces function in coupled shear walls due to uniform lateral loading (wind)
$\psi_s(\alpha,\xi)$	the same, due to triangular lateral loading (earthquake)
$\varphi_v(\alpha,\xi)$	shear forces function in coupled shear walls due to uniform lateral loading (wind)
$\varphi_s(\alpha,\xi)$	the same, for triangular lateral forces (earthquake)

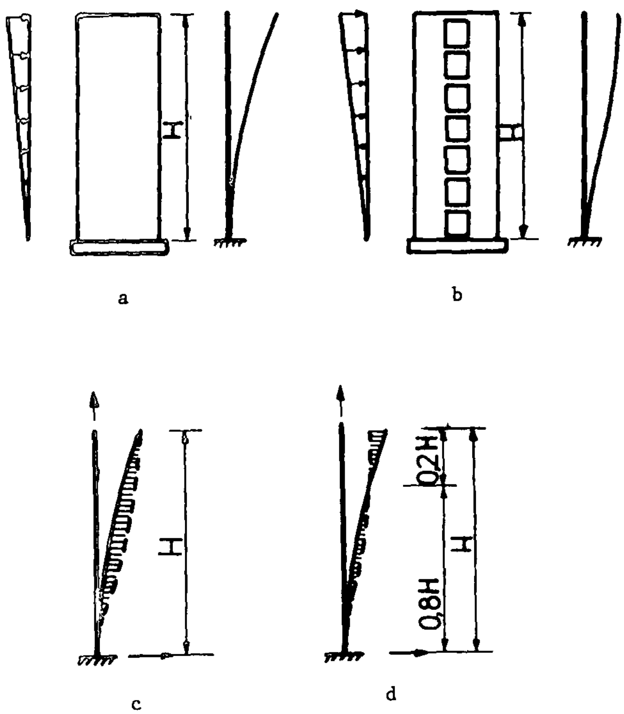


Fig. A1.1

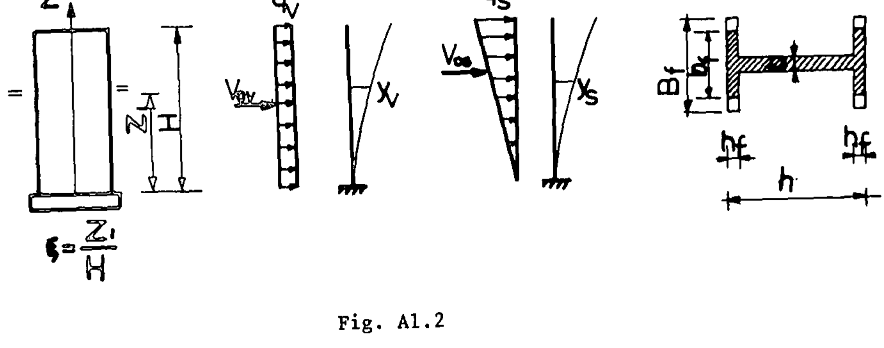
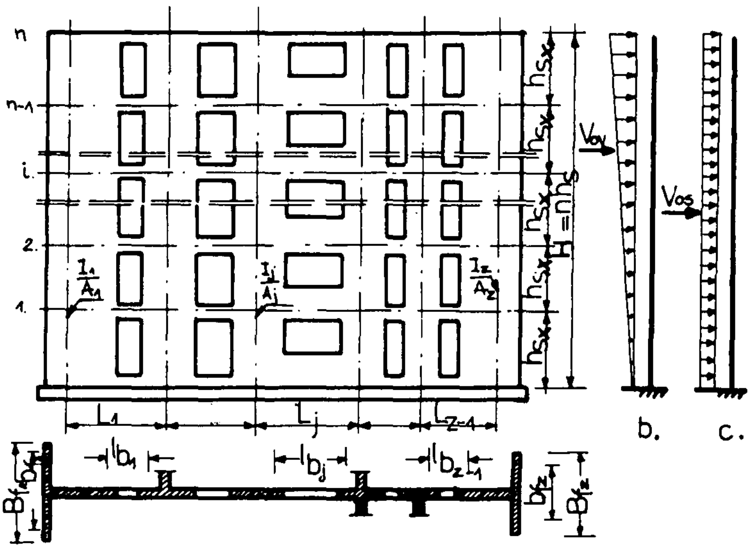
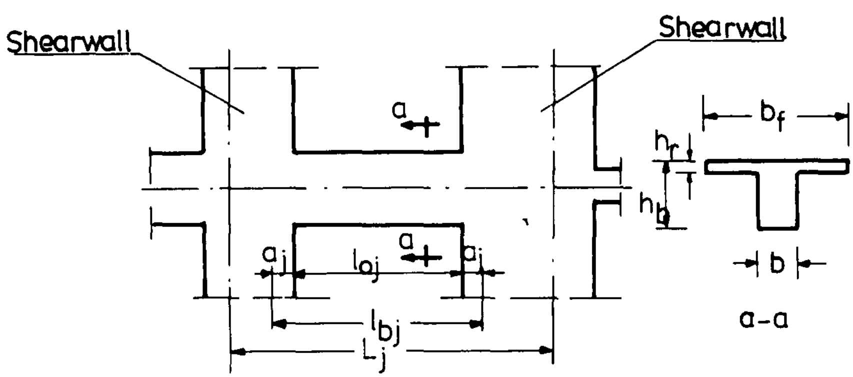


Fig. A1.2



n
n-1
i
2
1
hsx
H = nhs
Voy
Vos
L1
Lj
Lz-1
lb1
lbj
lbz
b.
c.



Shearwall
Shearwall
a
a
aj
loj
aj
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Lj
bf
hr
hb
b
a-a

Fig. A1.3

APPENDIX A2

A2.1 Values for Functions Y, K and β Necessary for Calculations of Displacements and Internal Forces for 7-11 Story Buildings

Table A1 - Function Y_v^M

Story	Total number of floors				
	7	8	9	10	11
11	-	-	-	-	0.1250
10	-	-	-	0.1250	0.1098
9	-	-	0.1250	0.1083	0.0947
8	-	0.1250	0.1065	0.0918	0.0798
7	0.1250	0.1042	0.0880	0.0753	0.0651
6	0.1012	0.0835	0.0700	0.0594	0.0510
5	0.0777	0.0634	0.0526	0.0443	0.0378
4	0.0549	0.0443	0.0364	0.0304	0.0258
3	0.0342	0.0272	0.0221	0.0183	0.0154
2	0.0168	0.0132	0.0106	0.0087	0.0073
1	0.0046	0.0036	0.0029	0.0023	0.0019

Table A2 - Function Y_s^M

Story	Total number of floors				
	7	8	9	10	11
11	-	-	-	-	0.1833
10	-	-	-	0.1833	0.1606
9	-	-	0.1833	0.1583	0.1380
8	-	0.1833	0.1556	0.1335	0.1156
7	0.1833	0.1521	0.1280	0.1090	0.0938
6	0.1477	0.1211	0.1010	0.0853	0.0729
5	0.1124	0.0921	0.0752	0.0630	0.0535
4	0.0788	0.0630	0.0515	0.0428	0.0362
3	0.0483	0.0382	0.0309	0.0255	0.0214
2	0.0234	0.0182	0.0146	0.0120	0.0100
1	0.0063	0.0049	0.0039	0.0032	0.0026

Table A3 - Function Y_v^V

Story	Total number of floors				
	7	8	9	10	11
11	-	-	-	-	0.500
10	-	-	-	0.500	0.496
9	-	-	0.500	0.495	0.483
8	-	0.500	0.494	0.480	0.463
7	0.500	0.492	0.475	0.455	0.434
6	0.490	0.469	0.444	0.420	0.397
5	0.459	0.430	0.401	0.375	0.351
4	0.408	0.375	0.346	0.320	0.298
3	0.337	0.305	0.278	0.255	0.236
2	0.245	0.219	0.198	0.180	0.166
1	0.133	0.117	0.105	0.095	0.087

Table A4 - Function Y_s^V

Story	Total number of floors				
	7	8	9	10	11
11	-	-	-	-	0.667
10	-	-	-	0.667	0.659
9	-	-	0.667	0.657	0.636
8	-	0.667	0.655	0.629	0.599
7	0.667	0.652	0.629	0.586	0.550
6	0.647	0.609	0.570	0.528	0.491
5	0.593	0.543	0.498	0.458	0.423
4	0.509	0.458	0.415	0.379	0.348
3	0.402	0.357	0.321	0.291	0.266
2	0.278	0.245	0.219	0.197	0.180
1	0.142	0.124	0.111	0.100	0.091

Table A5 - Functions $\bar{Y}_v^M$, $\bar{Y}_v^V$, $\bar{Y}_s^M$, $\bar{Y}_s^V$, β_s , β_v

Coefficient	Total number of floors				
	7	8	9	10	11
$\frac{\text{Floor no.}}{\text{no. floors}}$	$\frac{6}{7}$=0.858	$\frac{6}{8}$=0.750	$\frac{7}{9}$=0.778	$\frac{8}{10}$=0.800	$\frac{9}{11}$=0.818
$\bar{Y}_v^M$	0.1012	0.0835	0.0880	0.0918	0.0947
$\bar{Y}_v^V$	0.490	0.469	0.475	0.480	0.483
β_v	11.40	13.22	12.70	12.30	12.00
$\bar{Y}_s^M$	0.1477	0.1211	0.1280	0.1336	0.1380
$\bar{Y}_s^V$	0.648	0.609	0.621	0.629	0.636
β_s	11.65	12.55	12.13	11.77	11.50

Table A6 - Function K_v^M

Story	Total number of floors				
	7	8	9	10	11
11	-	-	-	-	0.000
10	-	-	-	0.000	0.004
9	-	-	0.000	0.005	0.017
8	-	0.000	0.006	0.020	0.037
7	0.000	0.008	0.025	0.045	0.066
6	0.010	0.031	0.056	0.080	0.103
5	0.041	0.070	0.099	0.125	0.149
4	0.092	0.125	0.154	0.180	0.202
3	0.163	0.195	0.222	0.245	0.264
2	0.255	0.281	0.302	0.320	0.335
1	0.367	0.383	0.395	0.405	0.413
0	0.500	0.500	0.500	0.500	0.500

Table A7 - Function K_s^M

Story	Total number of floors				
	7	8	9	10	11
11	-	-	-	-	0.000
10	-	-	-	0.000	0.004
9	-	-	0.000	0.010	0.031
8	-	0.000	0.012	0.037	0.068
7	0.000	0.015	0.046	0.081	0.116
6	0.019	0.057	0.099	0.139	0.175
5	0.074	0.123	0.168	0.208	0.243
4	0.157	0.208	0.251	0.288	0.319
3	0.264	0.309	0.346	0.376	0.401
2	0.389	0.422	0.448	0.469	0.487
1	0.525	0.542	0.556	0.567	0.576
0	0.667	0.667	0.667	0.667	0.667

Table A8 - Function K_v^V

Story	Total number of floors				
	7	8	9	10	11
11	-	-	-	-	0.000
10	-	-	-	0.000	0.091
9	-	-	0.000	0.100	0.182
8	-	0.000	0.111	0.200	0.273
7	0.000	0.185	0.222	0.300	0.364
6	0.143	0.250	0.333	0.400	0.455
5	0.286	0.375	0.444	0.500	0.545
4	0.428	0.500	0.555	0.600	0.636
3	0.571	0.625	0.666	0.700	0.727
2	0.714	0.750	0.777	0.800	0.818
1	0.857	0.875	0.888	0.900	0.909
0	1.000	1.000	1.000	1.000	1.000

Table A9 - Function K_s^V

Story	Total number of floors				
	7	8	9	10	11
11	-	-	-	-	0.000
10	-	-	-	0.000	0.173
9	-	-	0.000	0.190	0.331
8	-	0.000	0.210	0.360	0.471
7	0.000	0.234	0.395	0.510	0.595
6	0.265	0.438	0.555	0.640	0.702
5	0.490	0.609	0.691	0.750	0.793
4	0.673	0.750	0.802	0.840	0.868
3	0.816	0.859	0.889	0.910	0.923
2	0.918	0.937	0.951	0.960	0.967
1	0.980	0.984	0.988	0.990	0.992
0	1.000	1.000	1.000	1.000	1.000

A2.2 Design Charts for Functions ψ and φ

Chart A2.2.1

$\psi_v(\alpha, \xi)$

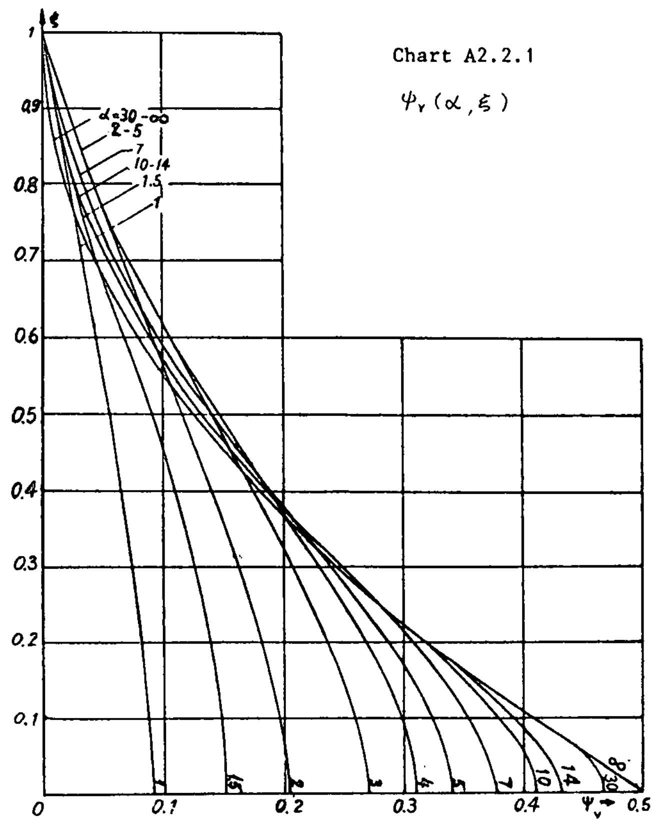


Chart A2.2.2

$\psi_3(\alpha, \xi)$

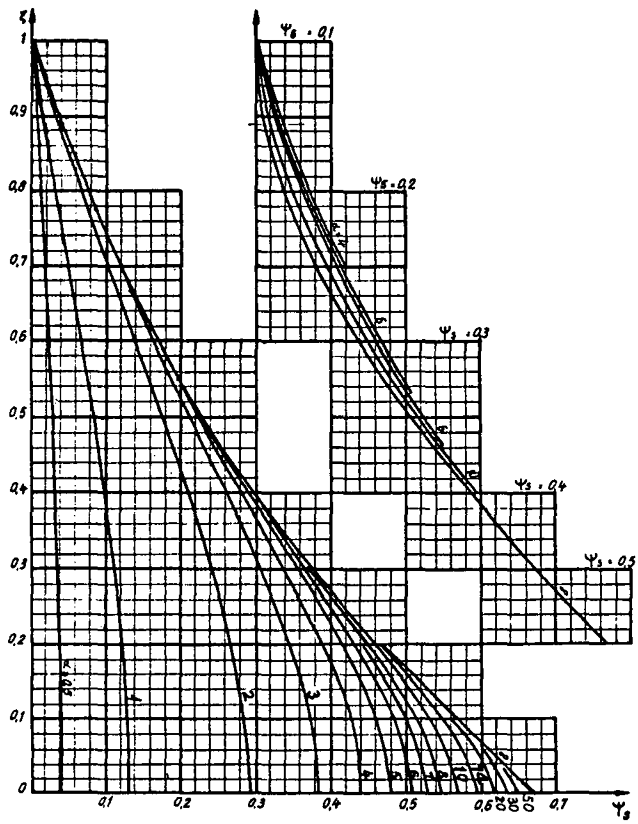


Chart A2.2.3

$\Phi_v(\alpha, \xi)$

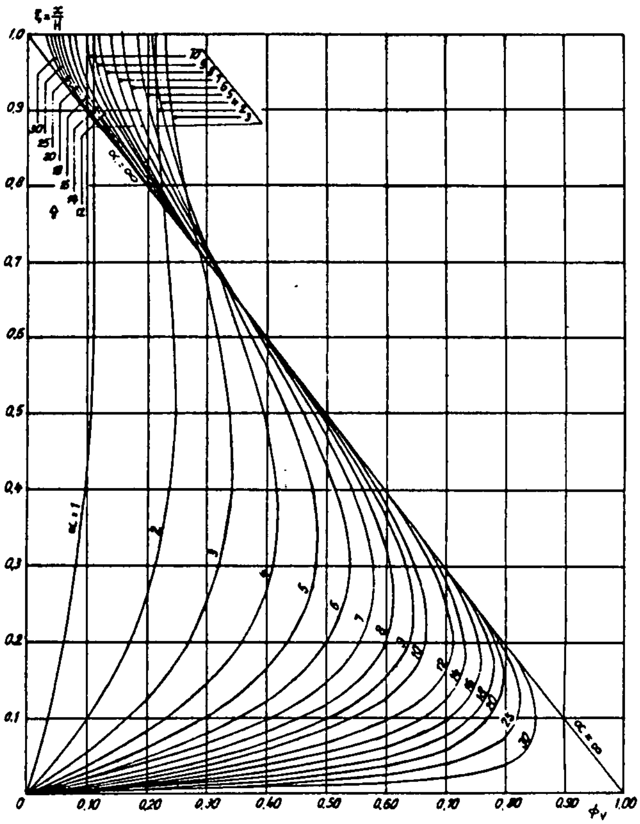
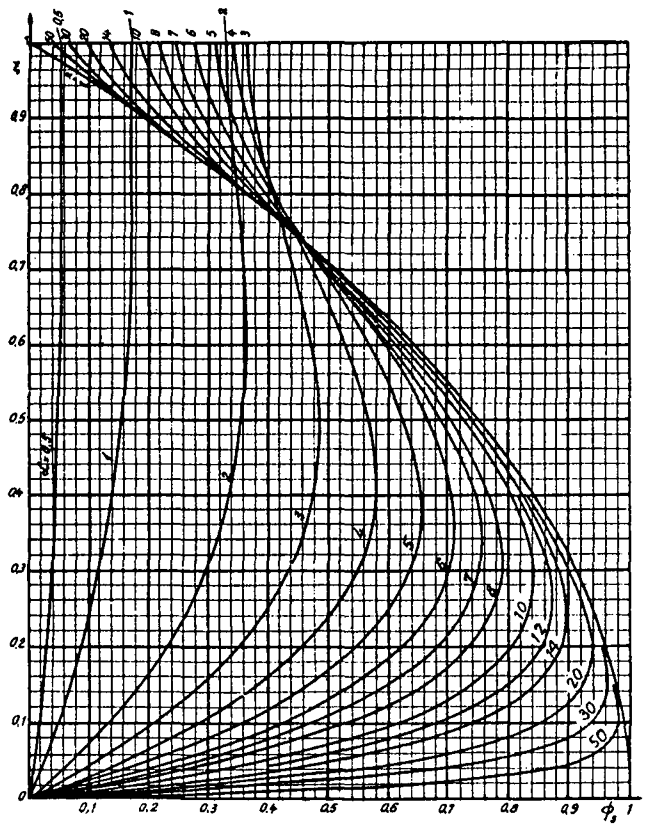
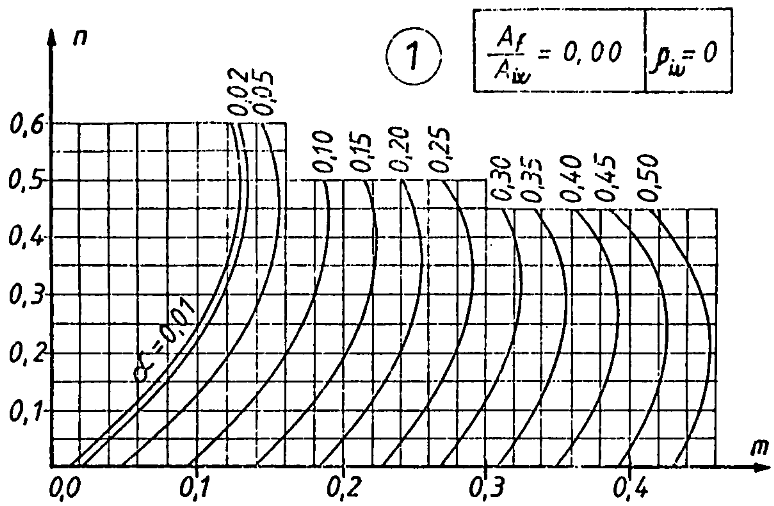


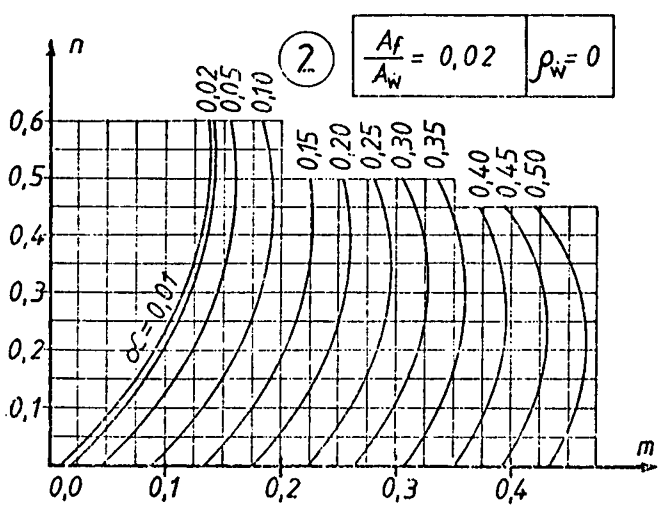
Chart A2.2.4

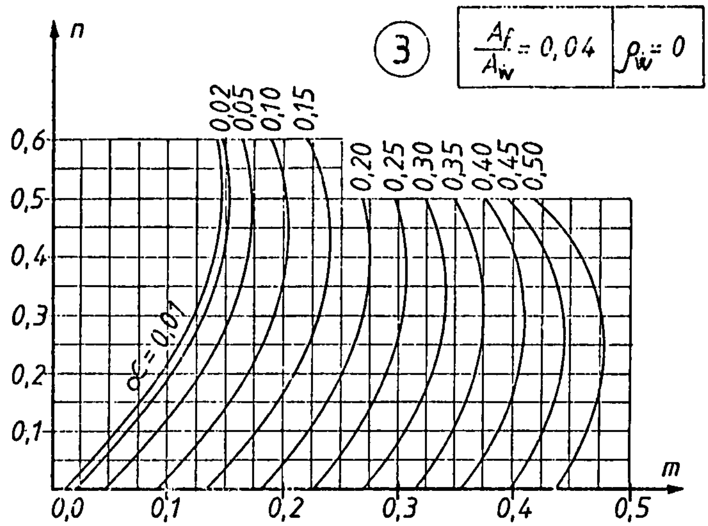
$\Phi_v(\alpha, \xi)$

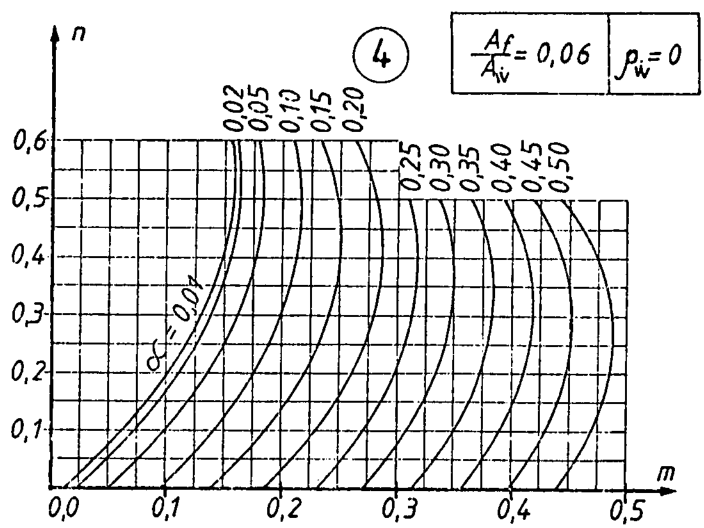


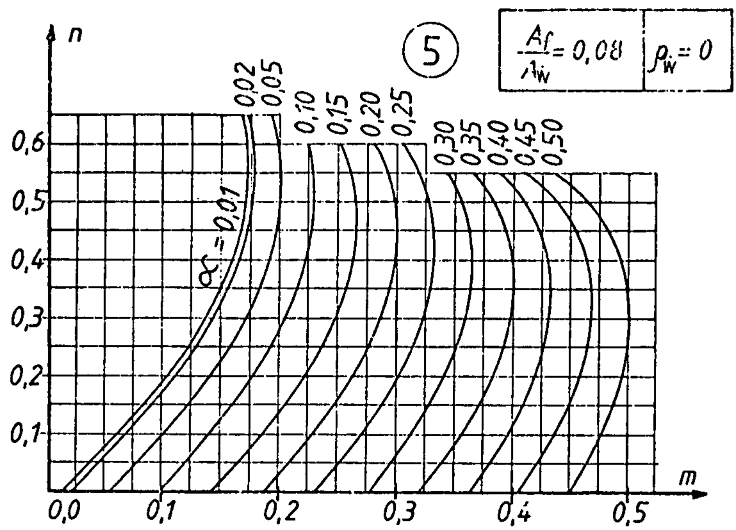
A2.3 Design Charts for Proportioning of Wall Sections

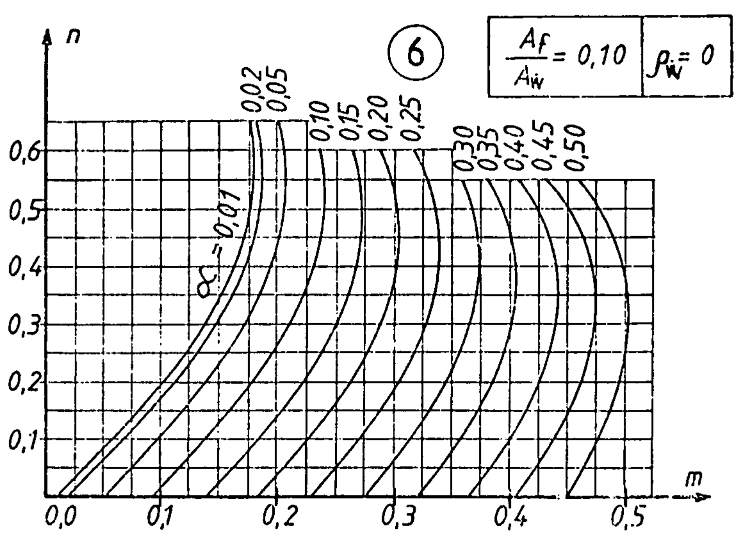


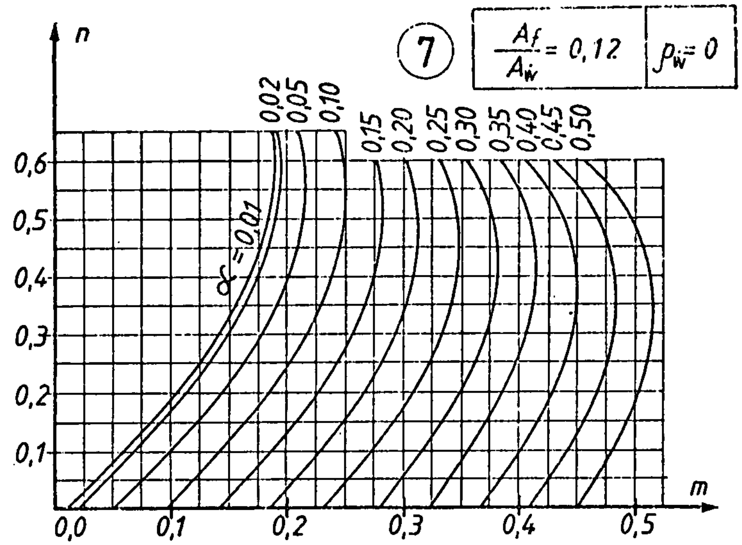


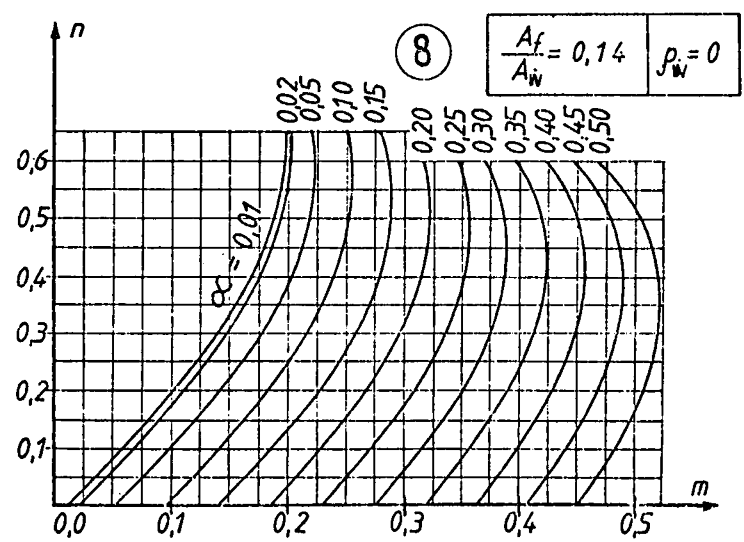

3
$\frac{A_f}{A_w} = 0,04$
$\rho_w = 0$
n
m
0,6
0,5
0,4
0,3
0,2
0,1
0,0
0,1
0,2
0,3
0,4
0,5
α = 0,01
0,02
0,05
0,10
0,15
0,20
0,25
0,30
0,35
0,40
0,45
0,50

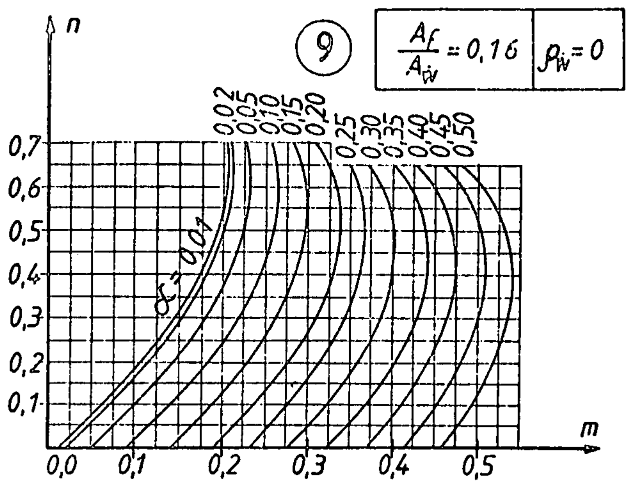

4
$\frac{A_f}{A_w} = 0,06$
$\rho_w = 0$
n
m
0,6
0,5
0,4
0,3
0,2
0,1
0,0
0,1
0,2
0,3
0,4
0,5
α = 0,01
0,02
0,05
0,10
0,15
0,20
0,25
0,30
0,35
0,40
0,45
0,50

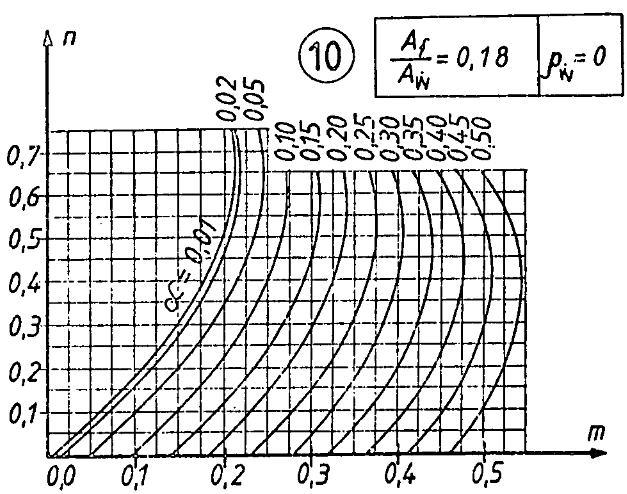

n
5
$\frac{A_f}{A_w} = 0,08$
$\rho_w = 0$
0,02
0,05
0,10
0,15
0,20
0,25
0,30
0,35
0,40
0,45
0,50
$\alpha = 0,01$
0,6
0,5
0,4
0,3
0,2
0,1
0,0
0,1
0,2
0,3
0,4
0,5
m

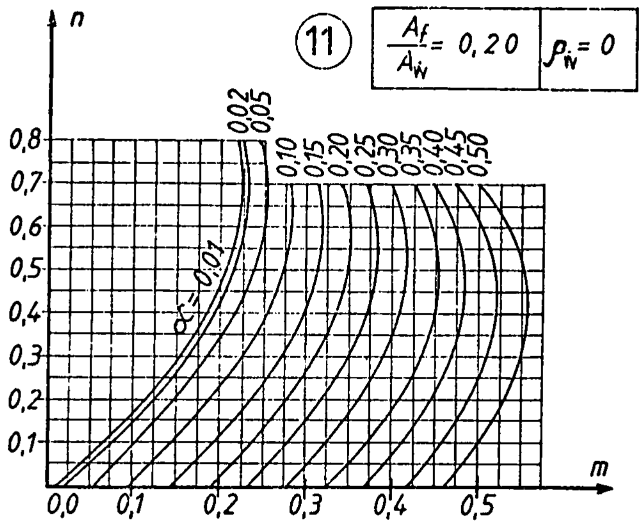

n
6
$\frac{A_f}{A_w} = 0,10$
$\rho_w = 0$
0,02
0,05
0,10
0,15
0,20
0,25
0,30
0,35
0,40
0,45
0,50
$\alpha = 0,01$
0,6
0,5
0,4
0,3
0,2
0,1
0,0
0,1
0,2
0,3
0,4
0,5
m

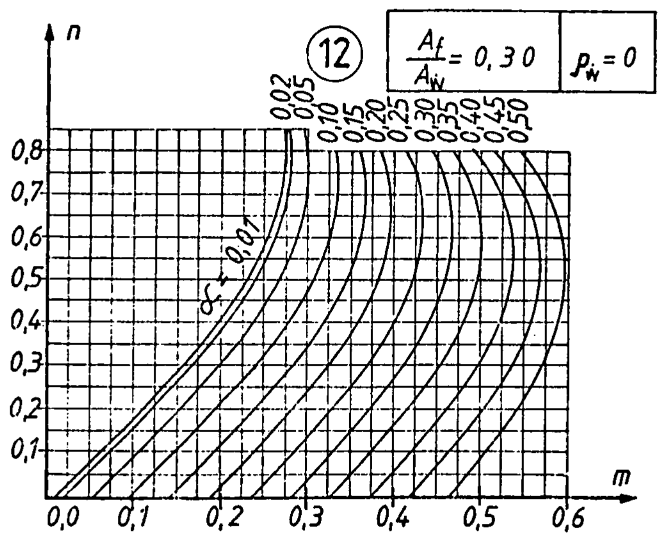

7
$\frac{A_f}{A_w}$ = 0,12
ρ_w = 0
n
m
0,0
0,1
0,2
0,3
0,4
0,5
0,1
0,2
0,3
0,4
0,5
0,6
α = 0,01
0,02
0,05
0,10
0,15
0,20
0,25
0,30
0,35
0,40
0,45
0,50

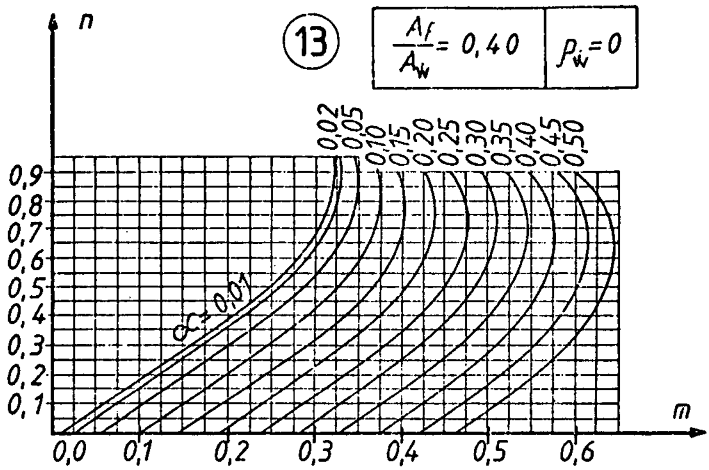

8
$\frac{A_f}{A_w}$ = 0,14
ρ_w = 0
n
m
0,0
0,1
0,2
0,3
0,4
0,5
0,1
0,2
0,3
0,4
0,5
0,6
α = 0,01
0,02
0,05
0,10
0,15
0,20
0,25
0,30
0,35
0,40
0,45
0,50

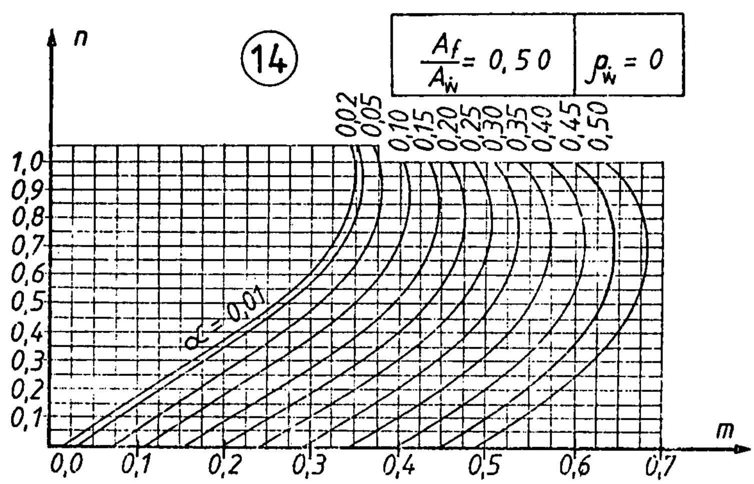

n
9
$\frac{A_f}{A_w} = 0,16$
$\rho_w = 0$
0,02 0,05 0,10 0,15 0,20
0,25 0,30 0,35 0,40 0,45 0,50
α = 0,01
0,7 0,6 0,5 0,4 0,3 0,2 0,1
0,0 0,1 0,2 0,3 0,4 0,5
m

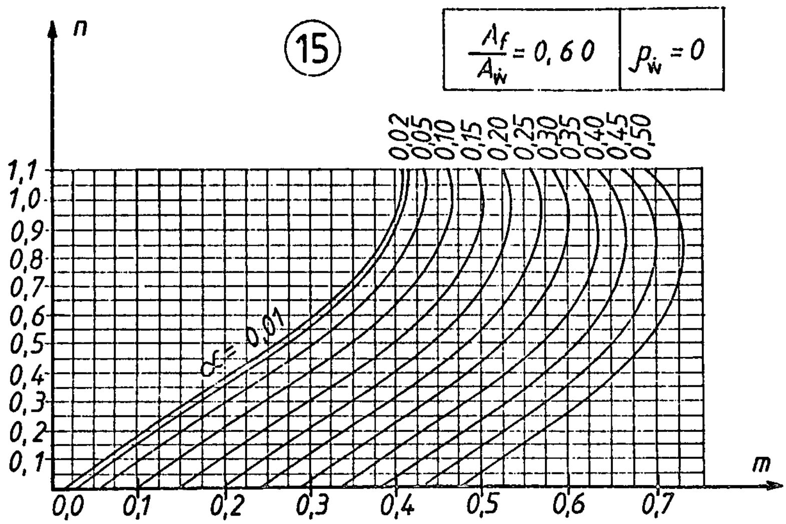

n
10
$\frac{A_f}{A_w} = 0,18$
$\rho_w = 0$
0,02 0,05
0,10 0,15 0,20 0,25 0,30 0,35 0,40 0,45 0,50
α = 0,01
0,7 0,6 0,5 0,4 0,3 0,2 0,1
0,0 0,1 0,2 0,3 0,4 0,5
m


n
11
$\frac{A_f}{A_w}$ = 0,20
ρ_w = 0
0,02
0,05
0,10
0,15
0,20
0,25
0,30
0,35
0,40
0,45
0,50
α = 0,01
0,8
0,7
0,6
0,5
0,4
0,3
0,2
0,1
0,0
0,1
0,2
0,3
0,4
0,5
m

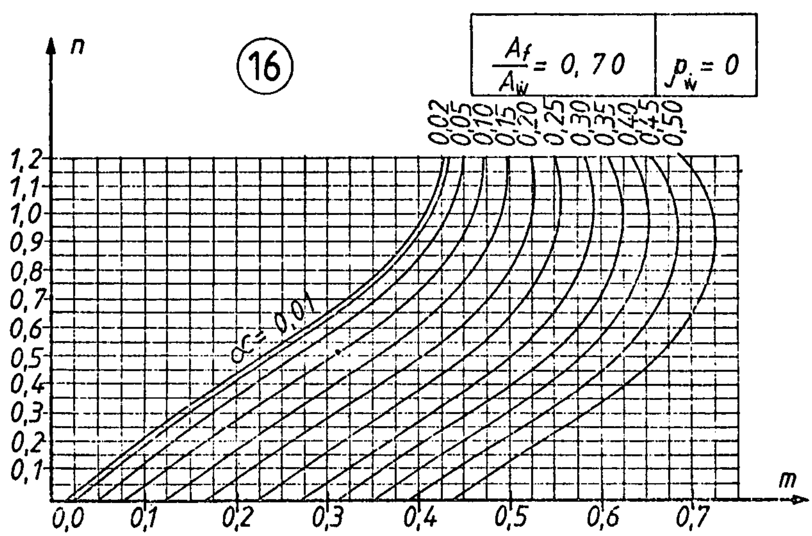

n
12
$\frac{A_f}{A_w}$ = 0,30
ρ_w = 0
0,02
0,05
0,10
0,15
0,20
0,25
0,30
0,35
0,40
0,45
0,50
α = 0,01
0,8
0,7
0,6
0,5
0,4
0,3
0,2
0,1
0,0
0,1
0,2
0,3
0,4
0,5
0,6
m


13
$\frac{A_f}{A_w} = 0,40$
$\rho_w = 0$
n
m
$\alpha = 0,01$
0,02 0,05 0,10 0,15 0,20 0,25 0,30 0,35 0,40 0,45 0,50
0,9 0,8 0,7 0,6 0,5 0,4 0,3 0,2 0,1
0,0 0,1 0,2 0,3 0,4 0,5 0,6

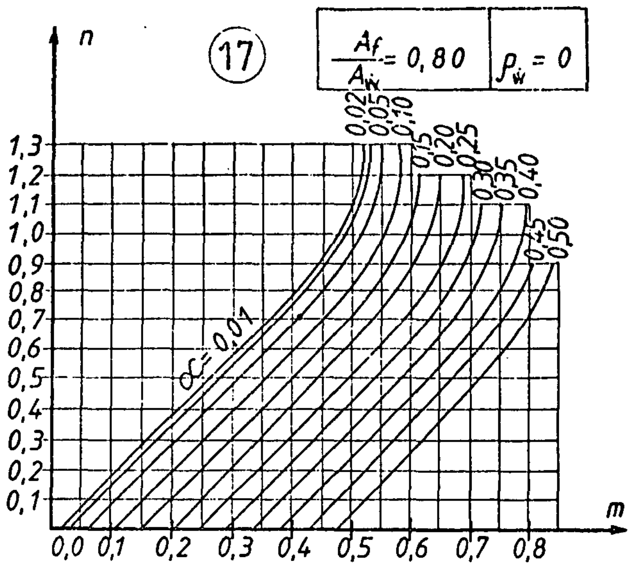

14
$\frac{A_f}{A_w} = 0,50$
$\rho_w = 0$
n
m
$\alpha = 0,01$
0,02 0,05 0,10 0,15 0,20 0,25 0,30 0,35 0,40 0,45 0,50
1,0 0,9 0,8 0,7 0,6 0,5 0,4 0,3 0,2 0,1
0,0 0,1 0,2 0,3 0,4 0,5 0,6 0,7

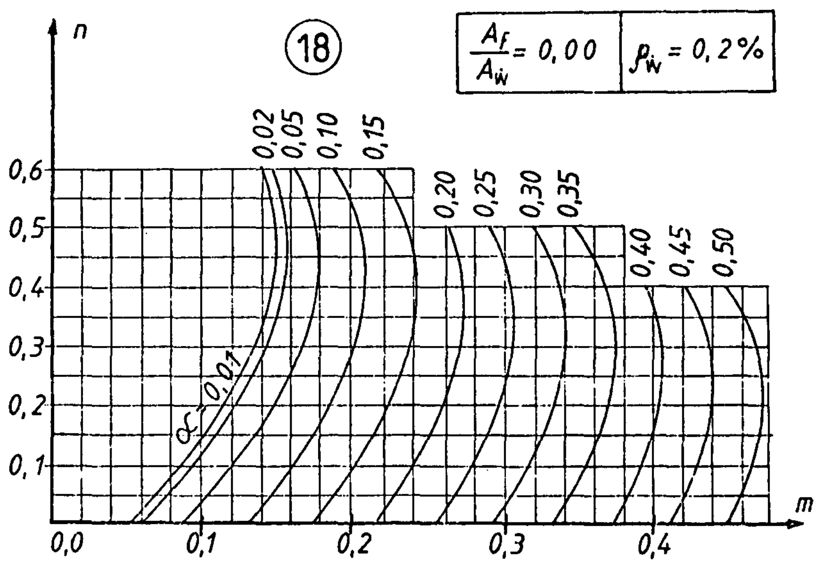


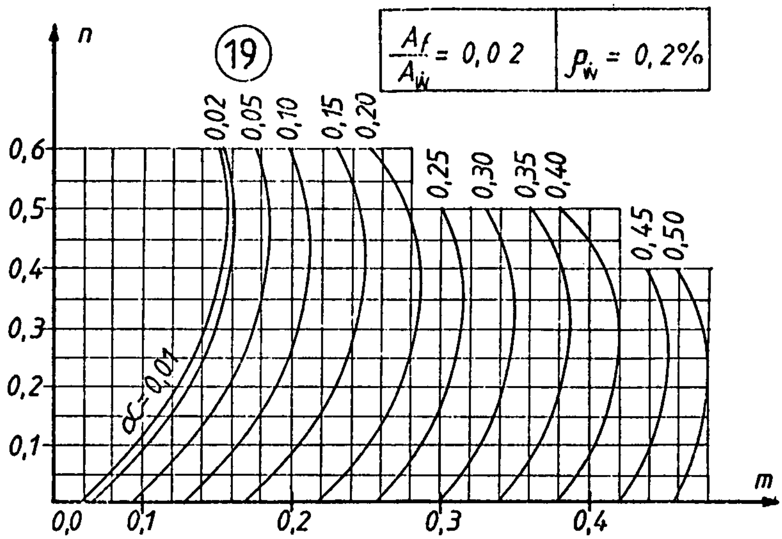
n
15
$\frac{A_f}{A_w} = 0,60$
$\rho_w = 0$
0,02 0,05 0,10 0,15 0,20 0,25 0,30 0,35 0,40 0,45 0,50
$\alpha = 0,01$
1,1
1,0
0,9
0,8
0,7
0,6
0,5
0,4
0,3
0,2
0,1
m
0,0 0,1 0,2 0,3 0,4 0,5 0,6 0,7



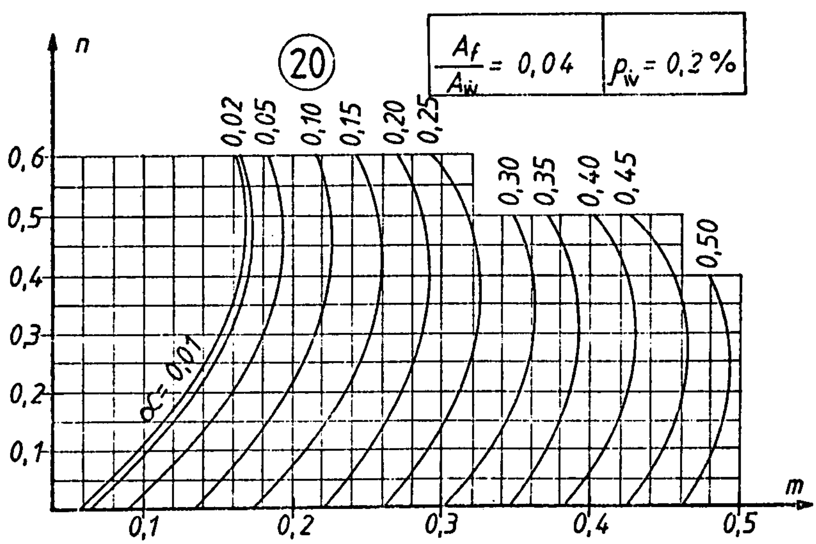
n
16
$\frac{A_f}{A_w} = 0,70$
$\rho_w = 0$
0,02 0,05 0,10 0,15 0,20 0,25 0,30 0,35 0,40 0,45 0,50
$\alpha = 0,01$
1,2
1,1
1,0
0,9
0,8
0,7
0,6
0,5
0,4
0,3
0,2
0,1
m
0,0 0,1 0,2 0,3 0,4 0,5 0,6 0,7


17
$\frac{A_f}{A_w}$ = 0,80
ρ_w = 0
n
m
α = 0,01
0,02 0,05 0,10 0,15 0,20 0,25 0,30 0,35 0,40 0,45 0,50
1,3 1,2 1,1 1,0 0,9 0,8 0,7 0,6 0,5 0,4 0,3 0,2 0,1
0,0 0,1 0,2 0,3 0,4 0,5 0,6 0,7 0,8

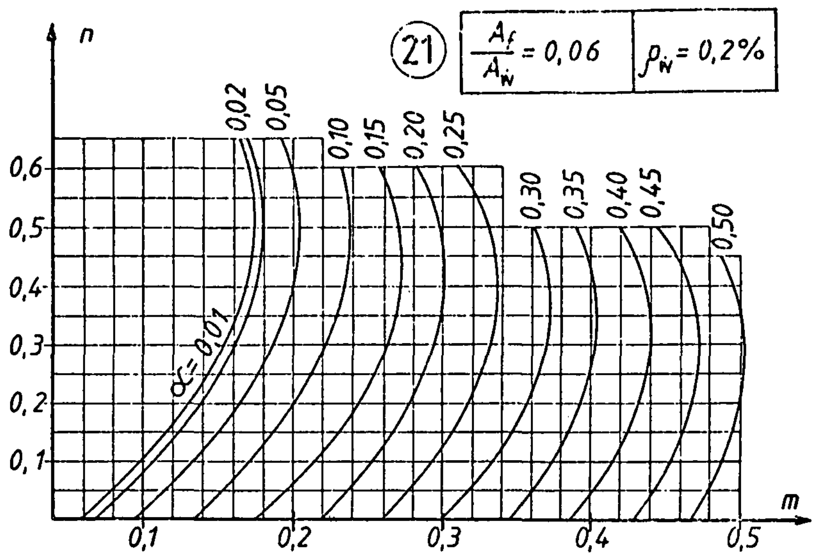

18
$\frac{A_f}{A_w}$ = 0,00
ρ_w = 0,2%
n
m
α = 0,01
0,02 0,05 0,10 0,15 0,20 0,25 0,30 0,35 0,40 0,45 0,50
0,6 0,5 0,4 0,3 0,2 0,1
0,0 0,1 0,2 0,3 0,4

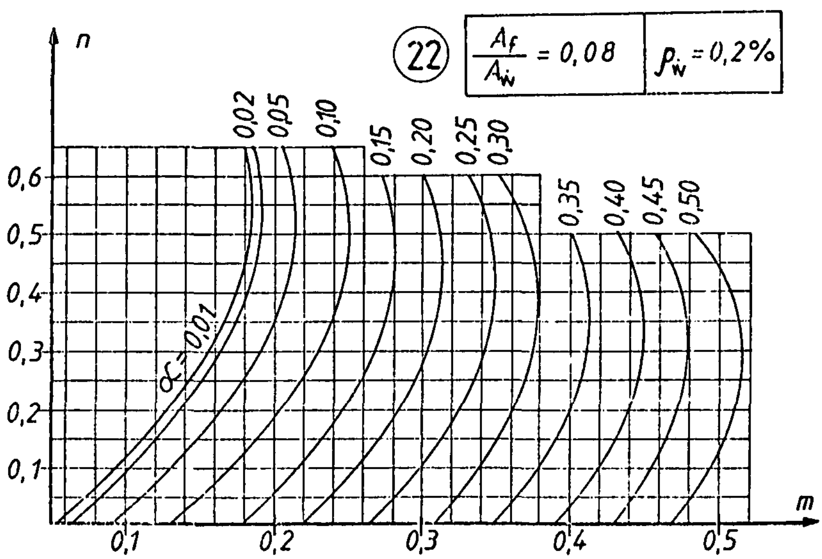


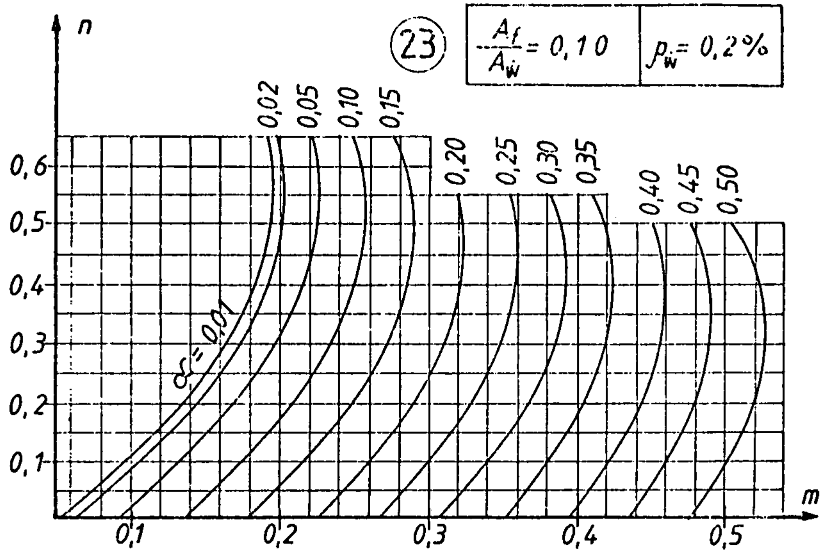
19
$\frac{A_f}{A_w} = 0,02$
$\rho_w = 0,2\%$
n
m
0,02 0,05 0,10 0,15 0,20 0,25 0,30 0,35 0,40 0,45 0,50
$\alpha = 0,01$
0,6 0,5 0,4 0,3 0,2 0,1
0,0 0,1 0,2 0,3 0,4



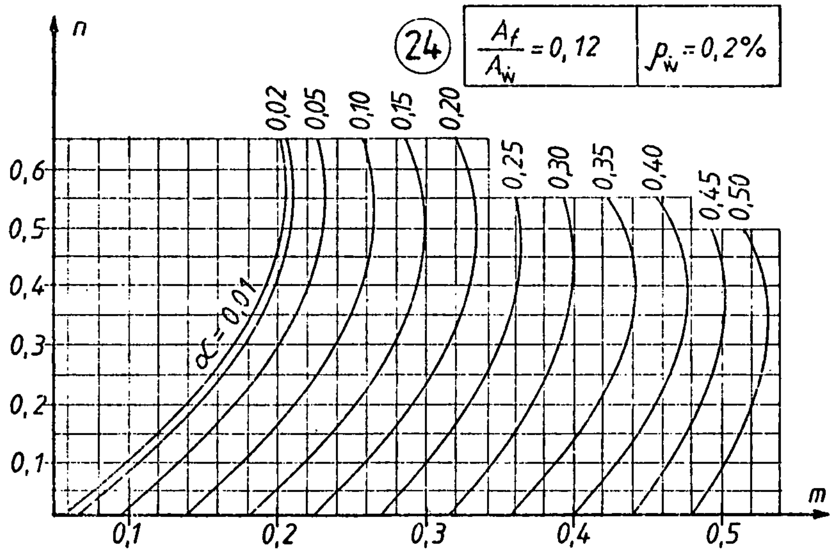
20
$\frac{A_f}{A_w} = 0,04$
$\rho_w = 0,2\%$
n
m
0,02 0,05 0,10 0,15 0,20 0,25 0,30 0,35 0,40 0,45 0,50
$\alpha = 0,01$
0,6 0,5 0,4 0,3 0,2 0,1
0,1 0,2 0,3 0,4 0,5


21
$\frac{A_f}{A_w} = 0,06$
$\rho_w = 0,2\%$
n
m
α = 0,01
0,02
0,05
0,10
0,15
0,20
0,25
0,30
0,35
0,40
0,45
0,50
0,6
0,5
0,4
0,3
0,2
0,1
0,1
0,2
0,3
0,4
0,5

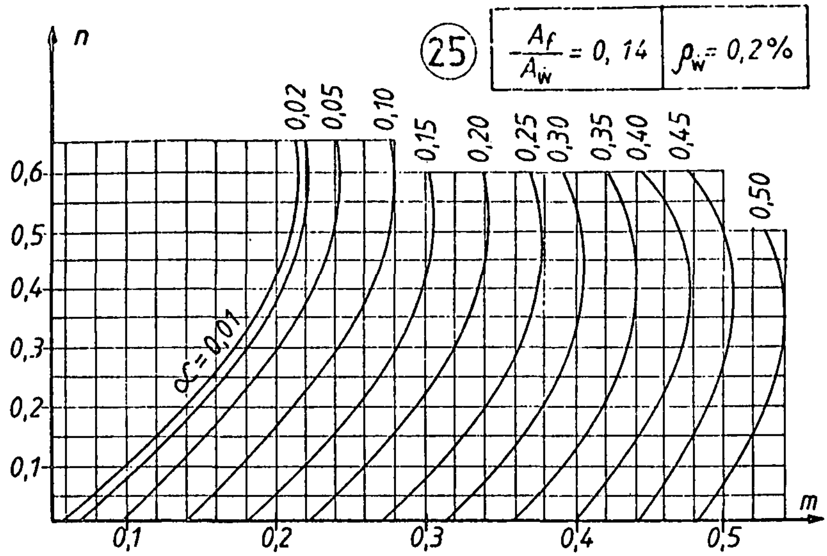

22
$\frac{A_f}{A_w} = 0,08$
$\rho_w = 0,2\%$
n
m
α = 0,01
0,02
0,05
0,10
0,15
0,20
0,25
0,30
0,35
0,40
0,45
0,50
0,6
0,5
0,4
0,3
0,2
0,1
0,1
0,2
0,3
0,4
0,5

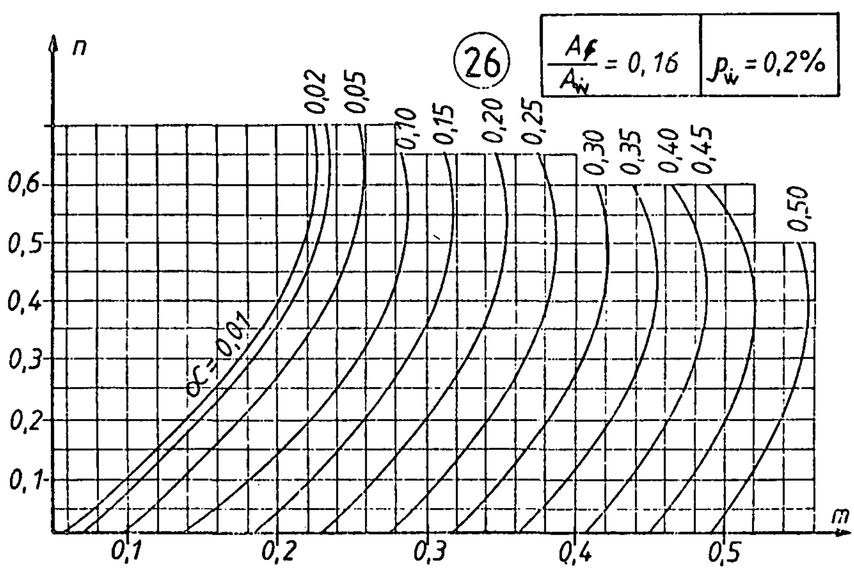


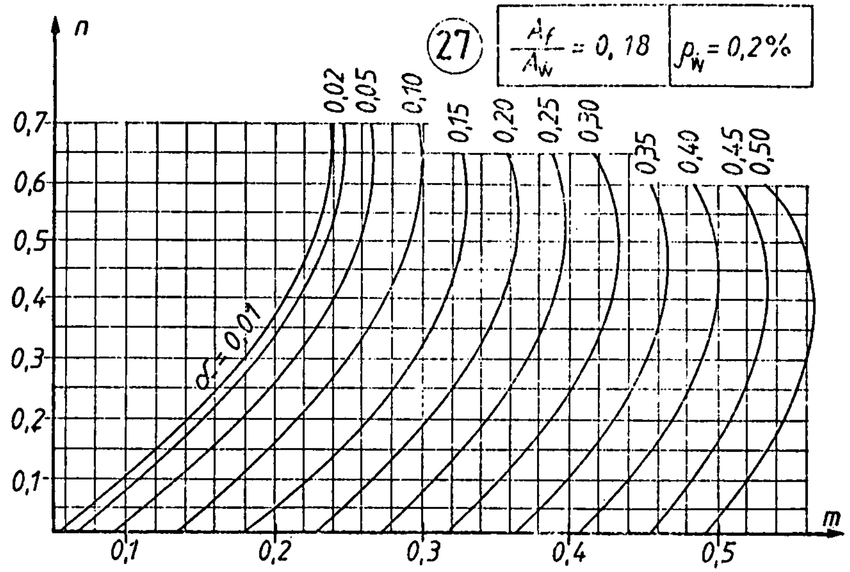
n
23
$\frac{A_f}{A_w}$ = 0,10
ρ_w = 0,2%
0,02
0,05
0,10
0,15
0,20
0,25
0,30
0,35
0,40
0,45
0,50
α = 0,01
0,6
0,5
0,4
0,3
0,2
0,1
m
0,1
0,2
0,3
0,4
0,5



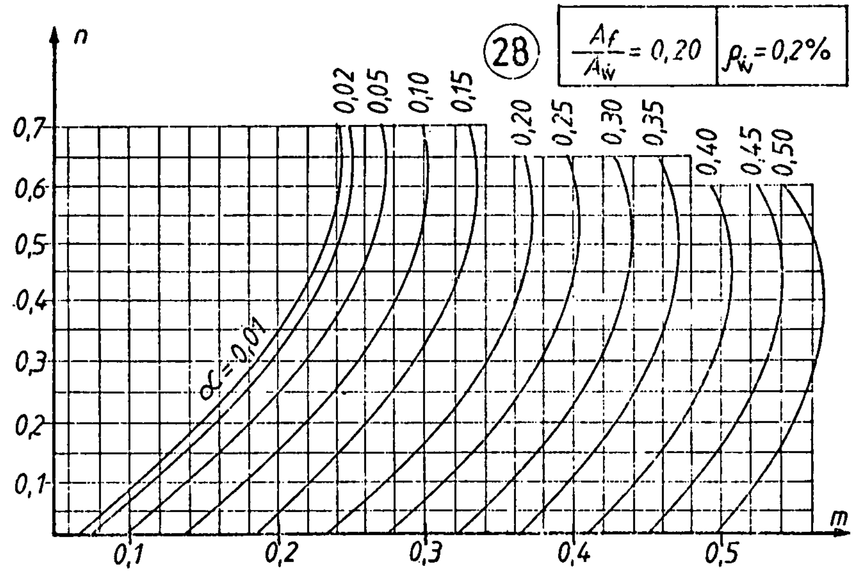
n
24
$\frac{A_f}{A_w}$ = 0,12
ρ_w = 0,2%
0,02
0,05
0,10
0,15
0,20
0,25
0,30
0,35
0,40
0,45
0,50
α = 0,01
0,6
0,5
0,4
0,3
0,2
0,1
m
0,1
0,2
0,3
0,4
0,5


25
$\frac{A_f}{A_w} = 0,14$
$\rho_w = 0,2\%$
n
m
α = 0,01
0,02
0,05
0,10
0,15
0,20
0,25
0,30
0,35
0,40
0,45
0,50
0,1
0,2
0,3
0,4
0,5
0,6

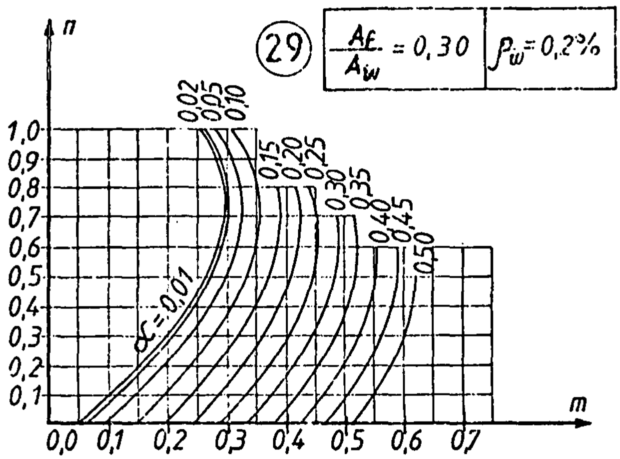

26
$\frac{A_f}{A_w} = 0,16$
$\rho_w = 0,2\%$
n
m
α = 0,01
0,02
0,05
0,10
0,15
0,20
0,25
0,30
0,35
0,40
0,45
0,50
0,1
0,2
0,3
0,4
0,5
0,6

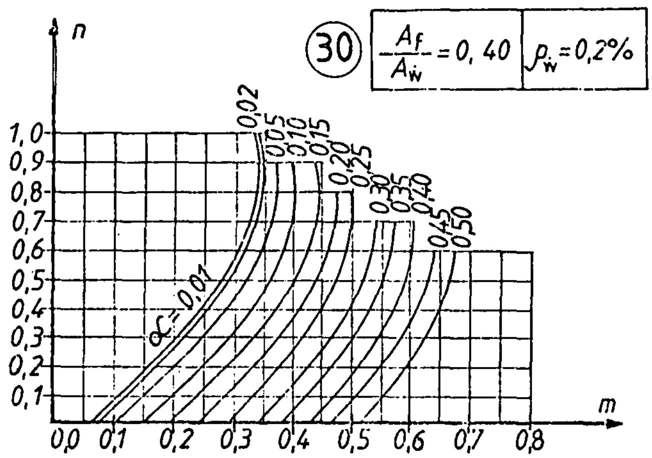


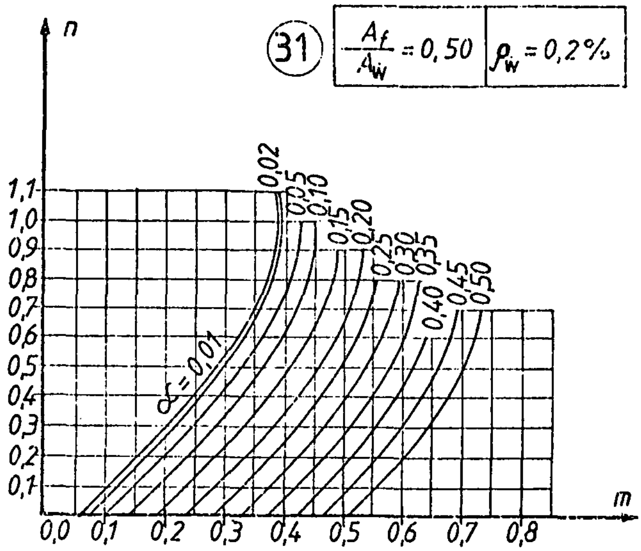
27
Af/Aw = 0,18
ρw = 0,2%
n
0,7
0,6
0,5
0,4
0,3
0,2
0,1
α = 0,01
0,02
0,05
0,10
0,15
0,20
0,25
0,30
0,35
0,40
0,45
0,50
m
0,1
0,2
0,3
0,4
0,5

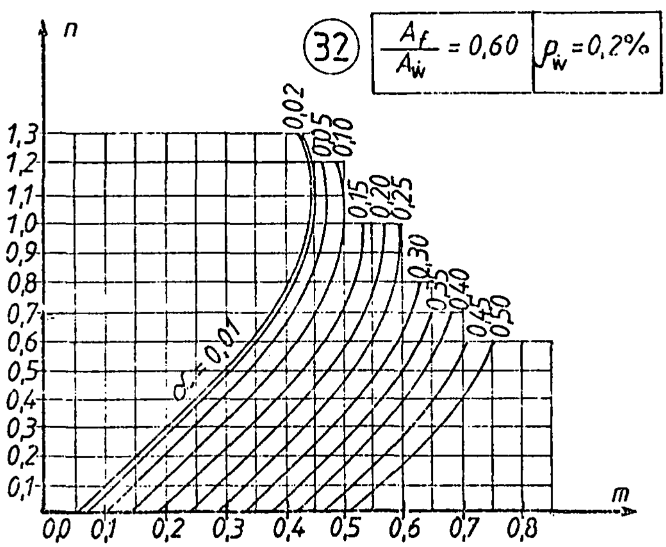


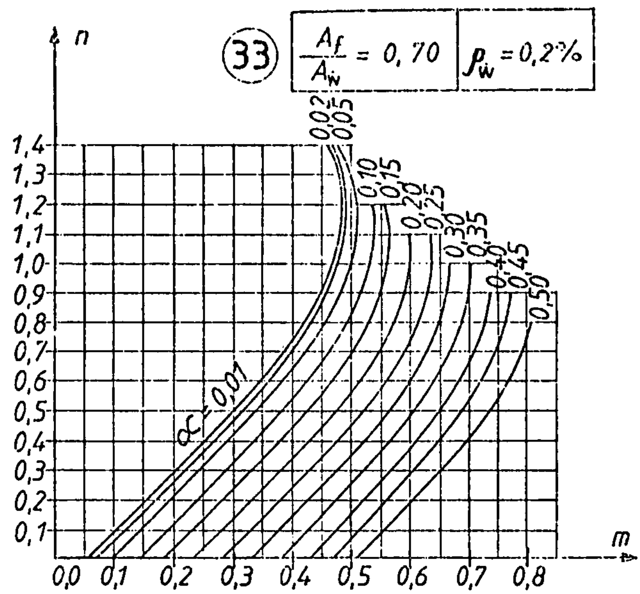
28
Af/Aw = 0,20
ρw = 0,2%
n
0,7
0,6
0,5
0,4
0,3
0,2
0,1
α = 0,01
0,02
0,05
0,10
0,15
0,20
0,25
0,30
0,35
0,40
0,45
0,50
m
0,1
0,2
0,3
0,4
0,5

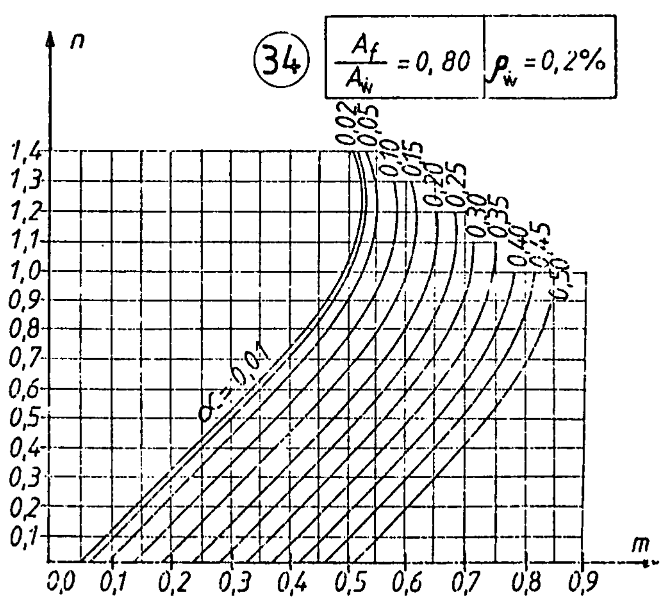

29
$\frac{A_f}{A_w} = 0,30$
$\rho_w = 0,2\%$
n
m
$\alpha = 0,01$
0,02
0,05
0,10
0,15
0,20
0,25
0,30
0,35
0,40
0,45
0,50
1,0
0,9
0,8
0,7
0,6
0,5
0,4
0,3
0,2
0,1
0,0
0,1
0,2
0,3
0,4
0,5
0,6
0,7


30
$\frac{A_f}{A_w} = 0,40$
$\rho_w = 0,2\%$
n
m
$\alpha = 0,01$
0,02
0,05
0,10
0,15
0,20
0,25
0,30
0,35
0,40
0,45
0,50
1,0
0,9
0,8
0,7
0,6
0,5
0,4
0,3
0,2
0,1
0,0
0,1
0,2
0,3
0,4
0,5
0,6
0,7
0,8


31
$\frac{A_f}{A_w} = 0,50$
$\rho_w = 0,2\%$
n
m
$\alpha = 0,01$
0,02
0,05
0,10
0,15
0,20
0,25
0,30
0,35
0,40
0,45
0,50
1,1
1,0
0,9
0,8
0,7
0,6
0,5
0,4
0,3
0,2
0,1
0,0
0,1
0,2
0,3
0,4
0,5
0,6
0,7
0,8


32
$\frac{A_f}{A_w} = 0,60$
$\rho_w = 0,2\%$
n
m
$\alpha = 0,01$
0,02
0,05
0,10
0,15
0,20
0,25
0,30
0,35
0,40
0,45
0,50
1,3
1,2
1,1
1,0
0,9
0,8
0,7
0,6
0,5
0,4
0,3
0,2
0,1
0,0
0,1
0,2
0,3
0,4
0,5
0,6
0,7
0,8


33
$\frac{A_f}{A_w} = 0,70$
$\rho_w = 0,2\%$
n
m
$\alpha = 0,01$
0,02
0,05
0,10
0,15
0,20
0,25
0,30
0,35
0,40
0,45
0,50
1,4
1,3
1,2
1,1
1,0
0,9
0,8
0,7
0,6
0,5
0,4
0,3
0,2
0,1
0,0
0,1
0,2
0,3
0,4
0,5
0,6
0,7
0,8


34
$\frac{A_f}{A_w} = 0,80$
$\rho_w = 0,2\%$
n
m
$\alpha = 0,01$
0,02
0,05
0,10
0,15
0,20
0,25
0,30
0,35
0,40
0,45
0,50
1,4
1,3
1,2
1,1
1,0
0,9
0,8
0,7
0,6
0,5
0,4
0,3
0,2
0,1
0,0
0,1
0,2
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0,4
0,5
0,6
0,7
0,8
0,9

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BULGARIAN DESIGN EXAMPLE

FRAME STRUCTURE

1. Introduction
2. Notations
3. Determination of Actions
4. Structural Analysis
5. Calculation of Reinforcement
6. Detailing
7. Analysis of the Results of the Calculations

1. Introduction

An apartment house of 16 stories, one garage story, one ground floor and one attic is selected as a design example.

The building is situated in an area of seismic intensity VIII by the MSK scale and is grounded on a large-grain, medium dense sand soil.

The structural scheme is skeleton-beamless.It consists of reinforced concrete columns and shear walls.The building is constructed with reinforced concrete M20 and has two stairwells.The partition walls between apartments and the stairwell walls are of reinforced concrete and bear the seismic forces, as well as the corresponding part of the vertical load.The vertical loads are borne by the reinforced concrete columns.

The inner partition walls (within an apartment) are 80 mm thick gypsum masonry.The exterior nonstructural walls are of light - weight concrete (ceramzitoconcrete M 15) and are hanged off the reinforced concrete slabs.From a technological viewpoint an equal thickness of 160 mm is adopted for the reinforced concrete shear walls along the whole height of the building.All the reinforced concrete elements are executed according to the large formwork system.

The inner story height is 2.64 m.The entrance doors of the apartments are 2.10 m high.A vitrage is envisaged over them, reaching up to the lower edge of the floor slabs.

Horizontal section through the stories is shown on Figure 1.Figure 2 illustrates the scheme of the elements, bearing the vertical and horizontal loads.Figure 3 shows schematically the cross-section of the building with notation of the individual story levels.The calculations are made under the following assumptions:

.The partition walls do not participate in the resisting of the horizontal forces, due to the low strength indices of the gypsum masonry.

.The exterior nonstructural walls do not resist the horizontal action, due to the mode adopted for their connection to the bearing structure.

.The shear walls with door openings are considered as individual, non-connected shear walls since the vitrage over the doors requires the two shear wall parts between the opening to be connected with the slab only, whose rigidity is not sufficient to equalize their deformations.

.The strains in the vertical elements bearing the seismic forces are determined considering the translation and rotation.

.The shear walls bearing the horizontal forces are erected throughout the whole building height and are with constant cross

.The seismic forces are assumed as equivalent static loads, concentrated at the story levels.

.The seismic forces act in a horizontal plane along the longitudinal and transversal axes of the building and the calculations are made separately for each direction.

.Floor slabs represent rigid horizontal non-deformable diaphragms which are able to distribute the horizontal forces over the vertical bearing elements proportional to their rigidities.

.Due to the low rigidity of the columns as compared to the shear walls rigidity, in their plane, it is assumed that the seismic forces are borne by the shear walls only.

.From a static point of view, a cantilever beam is assumed for the calculation model, which is fixed to the base, and the story masses taken as concentrated forces are accumulated along its height.

.The building is laid on solid soil foundations and the soil is assumed to be practically non-deformable.

2. Notations

According to Part II	Part I	
A	-	- coefficient taking in account the type of structure;
a	a	- shear wall length;
a_{jr}	-	- distance from the centroid of the element j to the center of rigidity of the examined story level;
a_{jx}, a_{jy}	-	- distance from the centroid of the element j to axes x and y respectively, through the rigidity center of the examined story level;
a_m	-	- distance from the respective axis of the accepted coordinate system to the mass center of the examined story level;
a_{ms}	-	- distance from the center of loading to the respective axis of the accepted coordinate system
a_r	-	- distance from the respective axis of the accepted coordinate system to the center of rigidity of the examined story level; for axes x and y the respective distances are x_r and y_r;
a_{rs}	-	- distance from the element rigidity center to the respective axis of the accepted coordinate system;
a_o, a'_o	-	- concrete cover of the tensile and compression reinforcement, resp.;
B, L	-	- width and length of the story outline of the building;
b	b	- thickness of the shear walls;
E_b	-	- concrete elasticity modulus;
$E_{jk}^{(1)}$	-	- seismic force correspondent to element j at level k for the first mode of vibration;
$\Delta E_{jk}^{(1)}$	-	- additional seismic force due to rotation in element j at level k for the first mode of vibration

$E_k^{(1)}$	E	- seismic force at level k for the first mode of vibration
e	e	- eccentricity of the normal force to the centroid of the tensile reinforcement
F	A	- transversal cross-section;
F_j	-	- cross-section area of element j ;
F_s	A_s	- cross-section area of the tension steel;
F'_s	A'_s	- cross-section area of the compression steel;
f_s	A_o	- cross-section of stirrups;
G	-	- young modulus for concrete;
H	H	- shear wall height;
H	-	- buckling design height of shear wall;
H_k	-	- height between upper foundation edge to story level ;
h	h	- cross-section height at bending;
h_o	-	- useful height of cross-section at bending;
I_j^t	-	- inertia moment of element j in plane perpendicular to the bearing plane of the shear wall
k_1	-	- coefficient for the shape of the cross-section of the element; for rectangular cross-section $k_1 = 1.20$
K_c	C_d	- seismic coefficient;
k_{jk}	-	- rigidity of element j , at level k ;
k_{jk}^t	-	- rigidity of element j at level k in plane perpendicular to the bearing plane of the shear wall;
k_{jx}, k_{jy}	-	- rigidity of element j along directions x and y , resp.;
$\sum_{s=1}^{m} k_s$	-	- sum of the element rigidities from 1 to m ;
k_φ	-	- rigidity of rotation;
m	-	- total number of the vertical bearing elements for a given story level;
$M_{tk}^{(i)}$	-	- torsion moment at level k for i-th mode of vibration;
N	N	- normal force on element;
N_1	-	- normal force from permanent acting loads;
N_2	-	- normal force from continuously acting variable loads;
N_3	-	- normal force from short term acting variable loads;
N_4	-	- normal force from short term acting peculiar loads;
N_c	-	- critical force;
n	n	- number of story levels;
n_s	-	- number of stirrup branches;
n_1	-	- coefficient of conditions of work;
Q_b	-	- bearing shear force by pure concrete cross-section;
Q_{bs}	-	- bearing shear force by concrete and stirrups
Q_j	-	- vertical load on element j ;
Q_k	-	- total vertical load on k th floor;
q_k	-	- computed loading of 1 sq.m. of the story slabs;
R_p	f_{cd}	- prismic design strength of concrete;
q_s	-	- bearing shear force by stirrups;
q_k^c	-	- loading of 1 sq.m. of the story slabs;
R_s	f_{yd}	- design tensile strength of the reinforcement;
R_{st}	-	- design strength of stirrup reinforcement;
R'_s	-	- design strength of compression reinforcement;
T_i	T	- period of the i-th mode of free vibration;

u	-	- distance between the stirrups;
$ß_i$	-	- dynamic coefficient of i -th mode of free vibration;
$б_{jk}$	-	- displacement of element j at level k from force = 1, applied at level k , along searched horizontal displacement;
η	-	- coefficient taking in account the eccentricity by buckling;
η_{ik}	-	- coefficient for the i -th mode of free vibration at level k ;
μ_{jk}	-	- distribution coefficient for element at level k ;
ε_o	-	- characteristics of the zone of compression;
ε_r	-	- relative limit characteristics of the zone of compression;

3. Determination of Actions

The seismic forces at story level are determined with the following formula:

$$E_{ik} = ß_i \eta_{ik} A K_c Q_k$$

The period of free vibrations is determined with the formula (for building with shear walls):

$$T = 0.09 \frac{H}{\sqrt{B}}$$

where H - building height in meters from the upper edge of the foundations to the top of the building;
B - the size in plan of the building in meters perpendicular to the considered direction; for the case $H = 51.50$ m; $B = 11.90$ m

$$T = 0.09 \frac{51.50}{11.90} = 1.3436 < 1.5 \text{ sec}$$

According to the Bulgarian code, at $T < 1.5$ sec , only the seismic forces for the first mode of vibration are determined.

The dynamic coefficient is determined with the formula:

$$ß_1 = \frac{0.7}{T_1} ; \quad (0.80 \leq ß_i \leq 2.40)$$

$$ß_1 = \frac{0.7}{1.3436} = 0.521 < 0.80 \quad \text{Accepted } ß_1 = 0.80$$

The coefficients for the modes of free vibration are determined as the ordinates of a triangle with base at the top of the building. They are shown on Figure 4.

The value of the coefficient A is accepted to be 1 for buildings with shear walls, in accordance with the requirements of the Bulgarian code.

The seismic coefficient K_c is determined as a function of two factors: the category of the building and the type of ground-base upon which the building is founded.

According to the Bulgarian code, the buildings and structures , depending on their importance and consequences due to their failure, are classified into four groups:

For buildings and structures classified in group A, such as monumental buildings and structures of national importance, monuments and buildings of special historic importance, theatres with spectator halls with 1200 seats, important government buildings, radio-broadcast buildings and like, industrial and energy supply facilities, on whose production the economy of the country is largely dependent, the design seismic intensity is accepted as increased, as compared to the seismic intensity of the area.

For buildings and structures classified in group B, such as industrial buildings for basic production, warehouses for goods and products of national importance, cinemas, theatres not included in group A, museums, libraries, residential buildings , hotels, schools, universities, etc., the calculated seismic intensity is accepted to be equal to the seismic intensity of the area.

For buildings and structures classified in group C,such as service buildings for the industry of small importance for the basic production, energy supply facilities of local importance, multilevel garages, etc., the design seismic intensity is accepted lower than the seismic intensity of the area.

For buildings and structures classified in group D, such as agricultural buildings of local importance, temporary buildings and facilities, garages not included in group C, etc., no seismic resistance is required.

The present building design example studied above is classified in group B, i.e., the calculated seismic intensity is accepted equal to the seismic intensity of the area, this being VIII.

The soils upon which the structures are founded, are classified in four groups in agreement with the Bulgarian code.

In this particular case, the soil upon which the studied building is founded(large-grain, medium density sand) is of the third group, whose value of the coefficient K_c for seismic intensity VIII is 0.05.

The loading of the story levels Q_k is determined by the formula:

$$Q_k = B\ L\ q_k$$

where $$q_k = n\ q_k^o$$

The coefficient of loading n are accepted:

- for the gravity weight of the structure and permanently acting loads n=1
- for variable loads n=0.5
- for snow load n=0.8

The values obtained for the loading on the story levels are given in Table 1.

The adopted calculation scheme with the determined story seismic forces and diagrams of section forces Q and M are given in Figure 5.

4. Structural Analysis

The distribution for the story level seismic forces from translation to the elements for level k is obtained with the formula:

$$E_{jk} = \frac{E_k^{(1)} k_{jk}}{\sum\limits_{s=1}^{m} k_s} = \mu_{ik} E_k^{(1)}$$

$$\mu_{jk} = \frac{k_{jk}}{\sum\limits_{s=1}^{m} k_s}$$

Rigidities k_{jk} are determined as a reciprocal value of displacements δ_{jk}. Displacements are determined by taking into account deformations due to slipping and bending of the structure with the following formula:

$$\delta_{jk} = \frac{k_1 H_k}{F_j G} + \frac{H_k^3}{3E_b I_j}$$

$$k_{jk} = \frac{1}{\delta_{jk}}$$

In this particular case it is assumed that the concrete strength is 20, where:

$$E_b = 2.4\text{x}10^4 \text{MPa};\ G = 0.425E_b = 1.02\text{x}10^4 \text{MPa}$$

Table 2 gives the geometrical characteristics of the shear walls, bearing the seismic forces and their coordinates with respect to the selected coordinate system, shown on Figure 2.

Since the shear walls extend throughout all the stories and do not change their geometrical dimensions, displacements δ_{jk} and, therefore, rigidities k_{jk} depend on the height H_k of the examined story level only. Considering that the change of rigidity k_{jk} and the distribution coefficients μ_{jk} are insignificant, and in order to simplify the calculations, we divide conventionally the building along three levels (Fig.3): from the 19-th to the 14-th story level incl., we operate with rigidities of the elements valid for level 19; from the 13-th to the 7-th story level incl., we operate with rigidities valid for level 13; and from the 6-th to the first story level we operate with rigidities valid for story level 6. The calculated displacements and rigidities are given in Tables 3,4 and 5. For the determination of distribution coefficients μ_{jk} the rigidity of the elements is considered only in their bearing plane, while the rigidity of the elements in the direction perpendicular to their plane is neglected due to its insignificant influence.

For the determination of the rigidity on rotation, the rigidities in both bearing planes of the shear walls and in the direction perpendicular to the bearing planes are considered.

The rigidity of the elements in the plane perpendicular to the bearing plane of the shear walls is determined with the formula

$$k_{jk}^{t} = \frac{3E_b y_j^t}{H_k^3}$$

The values obtained for k_{jk}^{t} and those obtained for the conventional division of the building into three levels are given in Tables 6,7 and 8. The values of the seismic forces and the distribution coefficient for shear walls 1 and 6 are given in Table 9. Seismic forces due to translation and section forces Q and M generated by them for shear walls 1 and 6 are shown on Figs. 6 and 7, respectively.

As a result of noncoincidence between the mass center and the rigidity center, additional forces generatedin the elements, are determined by the formulae:

$$\Delta E_{jk}^{(i)} = \pm \frac{k_{jk} a_{jr}}{k_{\varphi}} M_{tk}^{(i)} ;$$

$$k_{\varphi} = \sum_{j=1}^{m} (k_{jx} a_{jx}^2 + k_{jy} a_{jy}^2);$$

$$M_{tk}^{(i)} = E_k^{(i)} (a_r - a_m);$$

$$a_r = \frac{\sum_{j=1}^{m} k_j a_{rs}}{\sum_{j=1}^{m} k_j} ;$$

$$a_m = \frac{\sum_{j=1}^{m} Q_j a_{ms}}{\sum_{j=1}^{m} Q_j}$$

Values for the rigidity center for levels 19, 13 and 6 are given in Tables 10, 11 and 12, resp.

Values for the center of masses for levels 1 - 17, 18 and 19 are given in Tables 13 and 14, resp.

The rigidity of rotation for levels 19, 13 and 6 is determined with Tables 15, 16 and 17, resp.

The values for the torsional moments for the different levels are given in Table 18.

The additional seismic forces from rotation for shear walls 1 and 6 are given in Table 19.and Figs. 8 and 9 respectively.

The summation seismic forces from translation and rotation and forces Q and M for shear wall 6 are given in Table 20.

5. Calculation of Reinforcement

As an example, the reinforcement for the bottom cross-section of shear wall 6 (SW6), will be calculated.The maximum forces Q and M from translation and rotation are taken either from Figure 10 or Table 20.

The normal force N is determined with the formulae (in accordance with the acting Bulgarian code):

$$N = N_1 + N_2 + 0.8N_3 + N_4$$

The vertical reinforcement in the section is determined taking into consideration the compression normal force and the bending moment in the eccentric loaded section.

Dimensions of cross-section: b=0.16m; h=6.30m.

$N = 6497kN$; $M = 10231.96kNm$; $Q = 297.83kN$; $R_s = 360MPa$;

$R_s' = 1.2x360 = 432MPa > 400MPa$; assumed $R_s' = 400MPa$;

$R_p = n_1 R_p = 1.2x9 = 10.80MPa$;

$H = 51.50m$; $H' = 1.15x51.50 = 59.22m$;

$a_o = a_o' = 20cm$; $h_o = a - a_o = 630 - 20 = 610cm$;

$\frac{H'}{h} = \frac{59.225}{6.30} = 9.40 < 10; \rightarrow 0.4 < \frac{H}{h} < 10$;

$$N_c = 0.15\frac{E_b F}{\left(\frac{H'}{h}\right)^2} = 0.15\frac{2.4x10^4x16x6,30}{9.40^2} = 41068.356kN;$$

$$\eta = \frac{1}{1-\frac{N}{N_c}} = \frac{1}{1-\frac{6497}{41068.356}} = 1.1879;$$

$$e_o = \frac{M}{N} = \frac{10231.96}{6497} = 1.57487m;$$

$$e = e_o\eta+\frac{h_o-a_o}{2} = 1.57487x1.1879+\frac{6.10-0.20}{2} = 4.82m;$$

$$\bar{n} = \frac{N}{R'_p bh_o} = \frac{6497x10^{-3}}{10.8x0.16x6.1} = 0.6163;$$

$$\varepsilon_r = \frac{\varepsilon_o}{1+\frac{R_s}{400}\left(\frac{\varepsilon_o}{1,1}\right)}; \quad \varepsilon_o = 0.85-0.008R_p = 0.85-0.008x10.8 = = 0.7636;$$

$$R_s = 400MPa; \quad \varepsilon_r = \frac{0.7636}{1+\frac{400}{400}\left(1-\frac{0.7636}{1.1}\right)} = 0.58477; \quad \bar{n} > \varepsilon_r;$$

$$m = \frac{Ne}{R_p bh_o^2} = \frac{6497x4.821}{10.8x0.16x6.10^2}x10^{-3} = 0.487;$$

$$\delta = \frac{a'_o}{h_o} = \frac{20}{610} = 0.03278;$$

$$\alpha = \frac{m-\bar{n}\left(1-\frac{\bar{n}}{2}\right)}{1-\delta} = \frac{0.487-0.6163\left(1-\frac{0.6163}{2}\right)}{1-0.03278} = 0.06267;$$

$$\varepsilon = \frac{\bar{n}(1-\varepsilon_r)+2\alpha\varepsilon_r}{1-\varepsilon_r+2\alpha} = \frac{0.6163(1-0.58477)+2x0.06267x0.58477}{1-0.58477+2x0.06267} = 0.6089$$

$$F_s = F'_s = \frac{R'_p bh_o}{R_s} x \frac{m-\varepsilon\left(1-\frac{\varepsilon}{2}\right)}{1-\delta} = \frac{10.8x0.16x6.10}{400} x$$

$$x\frac{0.487-0.6089\left(1-\frac{0.6089}{2}\right)}{1-0.03278} x 10^4 = 17.295cm^2 - 6N20\,stAIII$$

Calculation for Q forces

$Q^{max} = 197.83kN$

$Q_b = 0.6xbh_o R'_t$; $R'_t = n_1 R_t = 1.2x0.75 = 0.90MPa$;

$Q_b = 0.6x0.16x6.10x0.90x10^3 = 527.04kN > 297.83kN$;

Assumed lateral reinforcement Φ 6.5 through 150mm

6. Detailing

Figure 11 shows the calculated reinforcement of shear wall 6.

Figures 12, 13 and 14 show different cases of connecting the reinforcing mesh of the shear walls.

Figure 15 shows the construction of columns at the intersection of the shear walls.

Figure 16 shows the anchoring of the end fields in the shear walls for the uppermost story slab.

Figure 17 shows the mode of reinforcing the lintels in cases when there is no vitrage over the wall openings.

7. Analysis of the Results Obtained from the Calculations

The calculation results show that the accepted dimensions of the shear walls are sufficient to carry the strains occuring in them.The value of the base shear coefficient amounts to 0.02756. As a result of non-coincidence of the centres of masses with the centres of rigidity, additional strains from rotation have occurred in the elements.The size of the rotation strains in percentages from the translation strains for the lowest cross-section 0 of shear wall 6 amounts to:

For Q - 43.6276 percent

For M - 44.0082 percent

It can be seen that additional rotation strains are significant and cannot be neglected.

For shear wall 6 the added strains from translation and rotation for the lowest cross-section 0 amount to:

Q = 297.83 kN; M = 10231.96 kNm

For the sake of comparison we shall give the strains for the cross-section of shear wall 6, obtained with the help of a computer programme, with which the spatial action of the structure is recorded. The programme takes into consideration the fact that at the places of intersecting, shear walls act as independent, unconnected shear walls, a prerequisite which is accepted with the manual calculations as well:

Q = 347.78kN; M = 10332.06 kNm

It can be seen that in both cases the difference in strains is not large.

TABLE OF THE STOREY SEISMIC FORCES

$$E_{ik} = \beta_i \eta_{ik} A K_c Q_k$$

Table 1

Storey Levels	$\beta_i K_c A$	η_{ik}	Q_k (kN)	E_{ik} (kN)
1	2	3	4	5
1	0.04	0.06291	6700	16.8599
2	0.04	0.12582	6700	33.7198
3	0.04	0.19922	6700	53.4044
4	0.04	0.27262	6700	73.0621
5	0.04	0.34602	6700	92.7334
6	0.04	0.41941	6700	112.4019
7	0.04	0.49281	6700	132.0731
8	0.04	0.56621	6700	151.7443
9	0.04	0.63961	6700	171.4155
10	0.04	0.71300	6700	191.0840
11	0.04	0.78640	6700	210.7552
12	0.04	0.85980	6700	230.4264
13	0.04	0.93320	6700	250.0976
14	0.04	1.00659	6700	269.7661
15	0.04	1.07999	6700	289.4373
16	0.04	1.15339	6700	309.1085
17	0.04	1.22679	6700	328.7797
18	0.04	1.30019	5180	269.3994
19	0.04	1.35000	3640	196.5600

TABLE OF THE SHEAR WALLS

Table 2

Shear Wall №	№ of the Kind	Sizes		Directia	Coordinates		Level	Angle to Axis x-x
		a(m)	b(m)		x(m)	y(m)		
1	1	5.32	0.16	x-x	-22.137	+4.750	1-19	0
2	2	3.45	0.16	x-x	-21.200	0	1-19	0
3	3	4.45	0.16	y-y	-22.925	-2.225	1-19	Π/2
4	4	5.20	0.16	y-y	-12.175	+3.400	1-19	Π/2
5	5	0.75	0.16	y-y	-12.175	+0.375	1-19	Π/2
6	6	6.30	0.16	y-y	-16.000	-3.150	1-19	Π/2
7	7	6.90	0.16	x-x	-12.475	0	1-19	Π/2
8	8	5.70	0.16	y-y	-7.725	+4.500	1-19	Π/2
9	9	0.75	0.16	y-y	-7.725	+0.375	1-19	Π/2
10	10	4.50	0.16	x-x	-5.850	0	1-19	0
11	11	6.30	0.16	y-y	-3.600	-3.150	1-19	Π/2
12	12	13.00	0.16	y-y	0	0	1-19	Π/2
13	13	6.30	0.16	y-y	+3.600	-3.150	1-19	Π/2
14	14	4.70	0.16	x-x	+5.950	0	1-19	0
15	15	5.70	0.16	y-y	+7.725	+4.500	1-19	Π/2
16	16	0.75	0.16	y-y	+7.725	+0.375	1-19	Π/2
17	17	5.70	0.16	y-y	+12.175	+4.500	1-19	Π/2
18	18	0.75	0.16	y-y	+12.175	+0.375	1-19	Π/2
19	19	3.40	0.16	x-x	+10.975	0	1-19	0
20	20	6.30	0.16	y-y	+12.675	-3.150	1-19	Π/2
21	21	6.30	0.16	y-y	+20.025	+0.825	1-19	Π/2

STIFFNESS K_J OF THE ELEMENTS AT LEVEL 19

$$\sigma_{jk} = 605887\text{x}10^{-3}/F_j + 18970955/I_j$$

Table 3

Shear Wall №	Dimensions		F_j	I_j	$\sigma_{jk}^{(19)}$	$k_{jk}^{(19)}=\frac{1}{\sigma_{jk}^{(19)}}$
	a(m)	b(m)	(m^2)	(m^4)	(m/kN)	(kN/m)
1	5.325	0.16	0.852	2.01320	0.94940	1.05330
2	3.450	0.16	0.552	0.54750	3.47600	0.28770
3	4.450	0.16	0.712	1.17490	1.62320	0.61610
4	5.200	0.16	0.832	1.87470	1.01920	0.98110
5	0.750	0.16	0.120	0.00536	337.01240	0.00297
6	6.300	0.16	1.008	3.33400	0.57500	1.73910
7	6.900	0.16	1.104	4.18010	0.43860	2.27990
8	5.700	0.16	0.912	2.46920	0.77490	1.29040
9	0.750	0.16	0.120	0.00563	337.01240	0.00297
10	4.500	0.16	0.720	1.21500	1.56980	0.63700
11	6.300	0.16	1.008	3.33400	0.57500	1.73910
12	13.000	0.16	2.080	29.29330	0.06767	14.77650
13	6.300	0.16	1.008	3.33400	0.57500	1.73910
14	4.700	0.16	0.752	1.38430	1.37850	0.72540
15	5.700	0.16	0.912	2.46920	0.77490	1.29040
16	0.750	0.16	0.120	0.00563	337.01240	0.00297
17	5.700	0.16	0.912	2.46920	0.77490	1.29040
18	0.750	0.16	0.120	0.00563	337.01240	0.00297
19	3.400	0.16	0.544	0.52410	3.63090	0.27540
20	6.300	0.16	1.008	3.33400	0.57500	1.73910
21	6.300	0.16	1.008	3.33400	0.57500	1.73910

STIFFNESS K_J OF THE ELEMENTS AT LEVEL 13

$$б_{jk} = 4.18822352 \times 10^{-3}/F_j + 0.62663911/I_j$$

Table 4

Shear Wall №	Dimensions		F_j	I_j	$б_{jk}^{(13)}$	$k_{jk}^{(13)} = \frac{1}{б_{jk}^{(13)}}$
	a(m)	b(m)	(m^2)	(m^4)	(m/kN)	(kN/m)
1	5.325	0.16	0.852	2.01320	0.31618	3.16275
2	3.450	0.16	0.552	0.54750	1.15213	0.86795
3	4.450	0.16	0.712	1.17490	0.53924	1.85447
4	5.200	0.16	0.832	1.87470	0.33929	2.94729
5	0.750	0.16	0.120	0.00563	111.33845	0.00898
6	6.300	0.16	1.008	3.33400	0.19211	5.20538
7	6.900	0.16	1.104	4.38010	0.14686	6.80927
8	5.700	0.16	0.912	2.46920	0.25837	3.87035
9	0.750	0.16	0.120	0.00563	111.33845	0.00898
10	4.500	0.16	0.720	1.21500	0.52157	1.91729
11	6.300	0.16	1.008	3.33400	0.19211	5.20538
12	13.000	0.16	2.080	29.29330	0.02340	42.72507
13	6.300	0.16	1.008	3.33400	0.19211	5.20538
14	4.700	0.16	0.752	1.38430	0.45824	2.18224
15	5.700	0.16	0.912	2.46920	0.25837	3.87035
16	0.750	0.16	0.120	0.00563	111.33845	0.00898
17	5.700	0.16	0.912	2.46920	0.25837	3.87035
18	0.750	0.16	0.120	0.00563	111.33845	0.00898
19	3.400	0.16	0.544	0.52410	1.20335	0.83102
20	6.300	0.16	1.008	3.33400	0.19211	5.20538
21	6.300	0.16	1.008	3.33400	0.19211	5.20538

STIFFNESS K_J OF THE ELEMENTS AT LEVEL 6

$$б_{jk} = 1.882353x10^{-3}/F_j + 0.056889/I_j$$

Table 5

Shear Wall №	Dimensions		F_j	I_j	$б_{jk}^{(6)}$	$k_{jk}^{(6)} = \frac{1}{б_{jk}^{(6)}}$
	a(m)	b(m)	(m^2)	(m^4)	(m/kN)	(kN/m)
1	5.325	0.16	0.852	2.01320	0.03047	32.82204
2	3.450	0.16	0.552	0.54750	0.10732	9.31819
3	4.450	0.16	0.712	1.17490	0.05106	19.58325
4	5.200	0.16	0.832	1.87470	0.03261	30.66723
5	0.750	0.16	0.120	0.00563	10.12030	0.09881
6	6.300	0.16	1.008	3.33400	0.01893	52.82425
7	6.900	0.16	1.104	4.38010	0.01469	68.05921
8	5.700	0.16	0.912	2.46920	0.02510	39.83520
9	0.750	0.16	0.120	0.00563	10.12030	0.09881
10	4.500	0.16	0.720	1.21500	0.04944	20.22793
11	6.300	0.16	1.008	3.33400	0.01893	52.82425
12	13.000	0.16	2.080	29.29330	0.00285	351.24383
13	6.300	0.16	1.008	3.33400	0.01893	52.82425
14	4.700	0.16	0.752	1.38430	0.04360	22.93631
15	5.700	0.16	0.912	2.46920	0.02510	39.83520
16	0.750	0.16	0.120	0.00563	10.12030	0.09881
17	5.700	0.16	0.912	2.46920	0.02510	39.83520
18	0.750	0.16	0.120	0.00563	10.12030	0.09881
19	3.400	0.16	0.544	0.52410	0.11201	8.92807
20	6.300	0.16	1.008	3.33400	0.01890	52.82425
21	6.300	0.16	1.008	3.33400	0.01890	52.82425

STIFFNESS K^{T}_{JK} OF THE ELEMENTS AT LEVEL 19

$$k_{jk} = 3EI_j/H^3$$

Table 6

Shear Wall №	Dimensions		Stiffness
	a(m)	b(m)	$k^{t(19)}_{jk} x10^4$
1	5.325	0.16	9.58100
2	3.450	0.16	6.20741
3	4.450	0.16	8.00666
4	5.200	0.16	9.35610
5	0.750	0.16	1.34943
6	6.300	0.16	11.33520
7	6.900	0.16	12.41480
8	5.700	0.16	10.25570
9	0.750	0.16	1.34943
10	4.500	0.16	8.09662
11	6.300	0.16	11.33520
12	13.000	0.16	23.39020
13	6.300	0.16	11.33520
14	4.700	0.16	8.45647
15	5.700	0.16	10.25570
16	0.750	0.16	1.34943
17	5.700	0.16	10.25570
18	0.750	0.16	1.34493
19	3.400	0.16	6.11745
20	6.300	0.16	11.33520
21	6.300	0.16	11.33520

STIFFNESS K^{T}_{JK} OF THE ELEMENTS AT LEVEL 13

$$k_{jk} = 3EI_j/H^3$$

Table 7

Shear Wall №	Dimensions		Stiffness
	a(m)	b(m)	$k^{t(13)}_{jk} x 10^3$
1	5.325	0.16	2.90057
2	3.450	0.16	1.87924
3	4.450	0.16	2.42395
4	5.200	0.16	2.83248
5	0.750	0.16	0.40853
6	6.300	0.16	3.43166
7	6.900	0.16	3.75848
8	5.700	0.16	3.10483
9	0.750	0.16	0.40853
10	4.500	0.16	2.45118
11	6.300	0.16	3.43166
12	13.000	0.16	7.08120
13	6.300	0.16	3.43166
14	4.700	0.16	2.56012
15	5.700	0.16	3.10483
16	0.750	0.16	0.40853
17	5.700	0.16	3.10483
18	0.750	0.16	0.40853
19	3.400	0.16	1.85200
20	6.300	0.16	3.43166
21	6.300	0.16	3.43166

STIFFNESS K_{JK}^{T} OF THE ELEMENTS AT LEVEL 6

$$k_{jk} = 3EI_j/H^3$$

Table 8

Shear Wall №	Dimensions		Stiffness
	a(m)	b(m)	$k_{jk}^{t(6)}$
1	5.325	0.16	0.0319499
2	3.450	0.16	0.0206999
3	4.450	0.16	0.0266999
4	5.200	0.16	0.0311999
5	0.750	0.16	0.0044999
6	6.300	0.16	0.0377999
7	6.900	0.16	0.0413999
8	5.700	0.16	0.0341999
9	0.750	0.16	0.0044999
10	4.500	0.16	0.0269999
11	6.300	0.16	0.0377999
12	13.000	0.16	0.0779998
13	6.300	0.16	0.0377999
14	4.700	0.16	0.0281999
15	5.700	0.16	0.0341999
16	0.750	0.16	0.0044999
17	5.700	0.16	0.0341999
18	0.750	0.16	0.0044999
19	3.400	0.16	0.0203999
20	6.300	0.16	0.0377999
21	6.300	0.16	0.0377999

SEISMIC FORCES FOR SHEAR WALLS 1 AND 6 DUE TO TRANSLATION

Table 9

	Seismic Forces for SW1 (kN)	Seismic Forces for SW6 (kN)		Seismic Forces for SW1 (kN)	Seismic Forces for SW6 (kN)
0	0	0	10	38.3123	11.6752
1	3.4091	1.1330	11	42.2564	12.8771
2	6.8081	2.2660	12	46.2005	14.0791
3	10.7984	3.5888	13	50.1446	15.2810
4	14.7732	4.9098	14	54.0341	16.2129
5	18.7507	6.2317	15	57.9743	17.3952
6	27.7277	7.5534	16	61.9139	18.5774
7	26.4806	8.0697	17	65.8546	19.7597
8	30.4247	9.2716	18	53.9607	16.1909
9	34.3688	10.4735	19	39.3710	11.8133

$$\mu_j = \frac{k_j}{\sum_{j=1}^{n} k_j}$$

$$\mu_1^{(19)} = \frac{1.0533}{5.2587} = 0.2003 \qquad \mu_1^{(13)} = \frac{3.1627}{15.7705} = 0.2005$$

$$\mu_1^{(6)} = \frac{32.8220}{162.2918} = 0.2022 \qquad \mu_6^{(19)} = \frac{1.7391}{28.9523} = 0.0601$$

$$\mu_6^{(13)} = \frac{5.2054}{85.2007} = 0.0611 \qquad \mu_6^{(6)} = \frac{52.8243}{785.5164} = 0.0672$$

CENTRE OF STIFFNESS AT LEVEL 19

Table 10

Shear Wall №	x_k (m)	$k^{(19)}_{jk,x}$ (MN/m)	y_k (m)	$k^{(19)}_{jk,y}$ (MN/m)	$x_k x k^{(19)}_{jk,x}$	$y_k x k^{(19)}_{jk,y}$
1			4.75	1.0533		5.00318
2			0	0.2877		0
3	-22.925	0.6161			-14.12409	
4	-12.175	0.9811			-11.94498	
5	-12.175	0.0029			-0.03612	
6	-16.000	1.7391			-27.82560	
7			0			0
8	-7.725	1.2904			-9.96834	
9	-7.725	0.0029			-0.02292	
10			0	0.6370		0
11	-3.600	1.7391			-6.26076	
12	0	14.7765			0	
13	3.600	1.7391			6.26076	
14			0	0.7254		0
15	7.725	1.2904			9.96834	
16	7.725	0.0029			0.02292	
17	12.175	1.2904			15.71061	
18	12.175	0.0029			0.03612	
19			0	0.2754		0
20	12.675	1.7391			22.04309	
21	20.025	1.7391			34.82550	
Σ		28.9523		5.2587	18.68447	5.00318

$$x_r^{(19)} = \frac{18.68447}{28.95230} = 0.6454\text{m} \qquad y_r^{(19)} = \frac{5.00318}{5.25870} = 0.9514\text{m}$$

CENTRE OF STIFFNESS AT LEVEL 13

Table 11

Shear Wall №	x_k (m)	$k^{(13)}_{jk,x}$ (MN/m)	y_k (m)	$k^{(13)}_{jk,y}$ (MN/m)	$x_k \times k^{(13)}_{jk,x}$	$y_k \times k^{(13)}_{jk,y}$
1			4.75	3.162746		15.023044
2			0	0.867950		0
3	-22.925	1.85447			-42.5137	
4	-12.175	2.94729			-35.8832	
5	-12.175	0.00898			-0.1093	
6	-16.000	5.20538			-83.2860	
7			0	6.809270		0
8	-7.725	3.87035			-29.8984	
9	-7.725	0.00898			-0.0693	
10			0	1.917290		0
11	-3.600	5.20538			-18.7393	
12	0	42.72507			0	
13	3.600	5.20538			18.7393	
14			0	2.18224		0
15	7.725	3.87035			29.8984	
16	7.725	0.00898			0.0693	
17	12.175	3.87035			47.1215	
18	12.175	0.00898			0.1093	
19			0	0.831016		0
20	12.675	5.20538			65.9781	
21	20.025	5.20538			104.2377	
Σ		85.20070		15.770512	55.6544	15.023044

$$x_r^{(13)} = \frac{55.6544}{85.2007} = 0.6532\text{m} \qquad y_r^{(13)} = \frac{15.023044}{15.770512} = 0.9526\text{m}$$

CENTRE OF STIFFNESS AT LEVEL 6

Table 12

	x_k (m)	$k^{(6)}_{jk,x}$ (MN/m)	y_k (m)	$k^{(6)}_{jk,y}$ (MN/m)	$x_k x k^{(6)}_{jk,x}$	$y_k x k^{(6)}_{jk,y}$
1			4.75	32.82204		155.90469
2			0	9.31819		0
3	-22.925	19.58325			-448.9460	
4	-12.175	30.66723			-373.3735	
5	-12.175	0.09881			-1.2030	
6	-16.000	52.82425			-845.1880	
7			0	68.05921		0
8	-7.725	39.83520			-307.7269	
9	-7.725	0.09881			-0.7633	
10			0	20.22793		0
11	-3.600	52.82425			-190.1673	
12	0	351.24383			0	
13	3.600	52.82425			190.1673	
14			0	22.93631		0
15	7.725	39.83552			307.7269	
16	7.725	0.09881			0.7633	
17	12.175	39.83520			484.9935	
18	12.175	0.09881			1.2030	
19			0	8.92807		0
20	12.675	52.82425			669.5473	
21	20.025	52.82425			1057.8056	
Σ		785.51640		162.29175	544.8388	155.90469

$$x_r^{(6)} = \frac{544.8388}{785.5164} = 0.6936\text{m} \qquad y_r^{(6)} = \frac{155.90469}{162.29175} = 0.9606\text{m}$$

DETERMINATION OF THE CENTRE OF THE MASSES AT LEVELS 1-17

Table 13

Shear Wall №	x_j(m)	Q_j(kN)	x_jQ_j
3	-22.925	191.90	-4399.3075
4	-12.175	210.60	-2564.0550
5	-12.175	30.37	-369.7548
6	-16.000	287.43	-4598.8800
8	-7.125	242.25	-1726.0313
9	-7.125	31.87	-227.0738
11	-3.600	283.50	-1020.6000
12	0	585.00	0
13	+3.600	283.50	1020.6000
15	+7.725	228.00	1761.3000
16	+7.725	29.99	231.6728
17	+12.175	228.00	2775.5000
18	+12.175	29.99	365.1283
20	+12.675	358.31	4541.5793
21	+20.025	141.75	2838.5438
Σ		3162.46	-1370.9783
	0.5x44.825-24.80 =-2.3875	6700-3162.46 =3537.54	-8445.8800
		6700	-9816.8583

$$x_m^{1-17} = \frac{\sum_{j=1}^{n} x_jQ_j}{\sum_{j=1}^{n} Q_j} = \frac{-9816.8583}{6700} \qquad x_m^{1-17} = -1.4652\text{m}$$

CONTINUATION

Table 13

Shear Wall №	y_j(m)	Q_j(kN)	y_jQ_j
1	+4.75	149.80	711.55
2	0	193.80	0
7	0	386.81	0
10	0	289.70	0
14	0	299.62	0
19	0	127.50	0
Σ		1447.23	711.55
	0	6700-1447.23 =5252.77	0
		6700	711.55

$$y_m^{1-17} = \frac{\sum\limits_{j=1}^{n} y_jQ_j}{\sum\limits_{j=1}^{n} Q_j} = \frac{711.55}{6700}$$

$$y_m^{1-17} = 0.1062 \text{ m}$$

DETERMINATION OF THE CENTRE OF THE MASSES FOR LEVELS 18,19

Table 14

	x_j(m)	Q_j^{18}(kN)	$Q_j^{18}x_j$	Q_j^{19}(kN)	$Q_j^{19}x_j$
3	-22.925	148.61	-3401.884	104.40	-2393.370
4	-12.175	163.09	-1985.621	114.56	-1394.768
5	-12.175	23.52	-286.356	16.52	-201.131
6	-16.000	222.59	-3561.440	156.37	-2501.920
8	-7.125	187.60	-1336.650	131.78	-938.932
9	-7.125	24.68	-175.845	17.34	-125.547
11	-3.600	219.54	-790.344	154.22	-555.192
12	0	453.02	0	318.24	0
13	+3.600	219.54	+790.344	154.22	+555.192
15	+7.725	176.56	+1363.926	124.03	+958.132
16	+7.725	23.23	+179.452	16.31	+125.995
17	+12.175	176.56	+2149.618	124.03	+1510.065
18	+12.175	23.23	+282.825	16.31	+198.574
20	+12.675	277.48	+3517.059	194.92	+2470.611
21	+20.025	109.77	+2198.144	77.11	+1544.128
		2449.02	-1061.772	1720.36	-746.164
	0.5x44.825 -24.80 =-2.3875	5180 -2449.02 =2730.98	-6520.215	3640 -1720.36 =1919.64	-4583.140
		5180	-7581.987	3640	-5329.304

$$x_m^{18} = \frac{-7581.9868}{5180} = -1.4637\text{m} \qquad x_m^{19} = \frac{-5239.3045}{3640} = -1.4641\text{m}$$

It is assumed that $x_m = -1.465$ m

CONTINUATION

Table 14

	y_j(m)	Q_j^{18}(kN)	$Q_j^{18} y_j$	Q_j^{19}(kN)	$Q_j^{19} y_j$
1	+4.75	116.00	551.00	81.50	387.125
2	0	150.11	0	105.40	0
7	0	299.54	0	210.42	0
10	0	224.33	0	157.60	0
14	0	232.03	0	163.00	0
19	0	98.74	0	69.40	0
		1120.75	551.00	787.32	387.125
		5180 -1120.75 =4059.25	0	3640 -787.32 =2852.68	0
		5180	551.00	3640	387.125

$$y_m^{18} = \frac{551}{5180} = 0.10637 \text{ m}$$

$$y_m^{19} = \frac{387.125}{3640} = 0.10635 \text{ m}$$

STIFFNESS K_φ AT LEVEL 19

Table 15

Shear Wall №	$k_{jy} \times 10^{-3}$	a_{jy}	$k_{jy}a_{jy}^2 \times 10^{-3}$	$k_{jx} \times 10^{-3}$	a_{jx}	$k_{jx}a_{jx}^2$
1	1053.300	3.7986	15194.400	0.958	-22.7829	0.4973
2	287.700	-0.9514	260.400	0.620	-21.8454	0.2962
7	2279.900	-0.9514	2063.700	1.241	-13.1204	0.2137
10	637.000	-0.9514	576.600	0.809	-6.4954	0.0341
14	725.400	-0.9514	656.600	0.845	-5.3046	0.0237
19	275.400	-0.9514	249.300	0.611	-10.3296	0.0652
3	0.800	-3.1764	8.078	616.100	-23.5704	342.2828
4	0.935	2.4486	5.609	981.100	-12.8204	161.2562
5	0.134	0.5764	4.483	2.967	-12.8204	0.4876
6	1.133	-4.1014	19.067	1739.100	-16.6454	481.8512
8	1.025	3.5486	12.915	1290.400	-8.3704	90.4100
9	0.134	0.5764	4.483	2.967	-8.3704	0.2078
11	1.133	-4.1014	19.067	1739.100	-4.1454	31.3445
12	2.339	-0.9514	2.117	14776.500	-0.6454	6.1550
13	1.133	-4.1014	19.167	1739.100	2.9546	15.1817
15	1.025	3.5486	12.915	1290.400	7.0796	64.6757
16	0.134	-0.5764	0.045	2.967	7.0796	0.1487
17	1.025	3.5486	12.915	1290.400	11.5296	171.5303
18	0.134	-0.5764	0.045	2.967	11.5296	0.3944
20	1.133	-4.1014	19.067	1739.100	12.0296	251.6673
21	1.133	-0.1264	0.018	1739.100	19.3796	653.1518
Σ			19136.020			2271.8809

$$k_\varphi^{(19)} = \sum_{j=1}^{n} k_{jx} a_{jx}^2 + \sum_{j=1}^{n} k_{jy} a_{jy}^2 = 2291.01684 \text{ kNm}$$

STIFFNESS K_{φ} AT LEVEL 13

Table 16

Shear Wall №	$k_{jy} x 10^{-3}$	a_{jy}	$k_{jy} a_{jy}^2 x 10^{-3}$	$k_{jx} x 10^{-3}$	a_{jx}	$k_{jx} a_{jx}^2$
1	3162.700	3.7974	45607.580	2.900	-22.7907	1.5066
2	867.900	-0.9526	787.620	1.879	-21.8532	0.8974
7	6809.300	-0.9526	6179.050	3.758	-13.1282	0.6477
10	1917.300	-0.9526	1739.840	2.451	-6.5032	0.1036
14	2182.200	-0.9526	1980.270	2.560	5.2968	0.0718
19	831.000	-0.9526	754.103	1.852	10.3218	0.1973
3	2.423	-3.1776	24.475	1845.500	-23.5782	1030.9583
4	2.832	2.4474	16.955	2947.300	-12.8282	485.0140
5	0.408	-0.5776	0.136	8.982	-12.8282	1.4781
6	3.431	-4.1026	57.759	5205.400	-16.6532	1443.6031
8	3.104	3.5474	39.071	3870.400	-8.3782	271.6762
9	0.408	-0.5776	0.236	8.982	-8.5782	0.6504
11	3.431	-4.1026	57.759	5205.400	-4.2532	94.1638
12	7.081	-0.9526	6.425	42725.100	-0.6532	18.2295
13	3.431	-4.1026	57.759	5205.400	2.9468	45.2016
15	3.104	3.5474	39.071	3870.400	7.0718	193.5575
16	0.408	-0.5776	0.136	8.982	7.0718	0.4491
17	3.104	3.5474	39.071	3870.400	11.5218	513.7962
18	0.408	-0.5776	0.136	8.982	11.5218	1.1923
20	3.431	-4.1026	57.759	5205.400	12.0218	752.3006
21	3.431	-0.1276	0.558	5205.400	19.3718	1953.4054
Σ			57439.240			6809.0814

$$k_{\varphi}^{(13)} = 6866.52066 \text{ kNm}$$

STIFFNESS K_{φ} AT LEVEL 6

Table 17

Shear Wall №	$k_{jy} \times 10^{-3}$	a_{jy}	$k_{jy} a_{jy}^2 \times 10^{-3}$	k_{jx}	a_{jx}	$k_{jx} a_{jx}^2$
1	32822.000	3.789	471309.800	0.031	-22.831	16.6541
2	9318.200	-0.960	8598.380	0.020	-21.893	9.9220
7	68059.200	-0.960	62801.790	0.041	-13.168	7.1792
10	20227.900	-0.960	18665.370	0.027	-6.543	1.1561
14	22936.300	-0.960	21164.530	0.028	5.256	0.7791
19	8928.100	-0.960	8238.390	0,020	10.281	2.1564
3	26.700	-3.185	270.950	19.583	-23.618	10924.2862
4	31.200	2.439	185.660	30.667	-12.868	5078.5198
5	4.499	-0.585	1.543	0.098	-12868	16.3602
6	37.800	-4.110	638.706	52.824	-16.693	14720.8655
8	34.200	3.539	428.435	39.835	-8.418	2823.2332
9	4.499	-0.585	1.543	0.098	-8.418	7.0029
11	37.800	-4.110	638.706	52.824	-4.293	973.8151
12	78.000	-0.960	71.974	351.243	-0.693	168.9767
13	37.800	-4.110	638.706	52.824	2.906	4467.1494
15	34.200	3.539	428.435	39.835	7.031	1969.4756
16	4.499	-0.585	1.543	0.098	7.031	4.8852
17	34.200	3.539	428.435	39.835	11.481	5251.1774
18	4.499	-0.585	1.543	0.098	11.481	13.0253
20	37.800	-4.110	638.706	52.824	11.981	7583.1295
21	37.800	-0.135	695.041	52.824	19.331	19740.5820
Σ			595103.860			69759.4000

$$k_{\varphi}^{(6)} = 70354.50386 \text{ kNm}$$

TORSION MOMENTS DUE TO ROTATION UNDER TRANSVERSE SEISMIC FORCE

Table 18

Level	Storey Seismic Forces (kN)	$A_x = x_r - x_m$ (m)	M_{tk} (kNm)
19	196.5600	0.6454-(-1.465)=2.111	414.9382
18	269.3994	2.111	568.7021
17	328.7797	2.111	694.0539
16	309.1075	2.111	652.5259
15	289.4373	2.111	611.0021
14	269.7661	2.111	569.4762
13	250.0976	0.6532-(-1.465)=2.118	529.7067
12	230.4264	2,118	488.0431
11	210.7552	2.118	446.3795
10	191.0840	2.118	404.7159
9	171.4155	2.118	363.0508
8	151.7443	2.118	321.3944
7	132.0731	2.118	279.7308
6	112.4019	0.6936-(-1.465)=2.159	242.6757
5	92.7334	2.159	200.2114
4	73.0621	2.159	157.7411
3	53.4044	2.159	115.3001
2	33.7198	2.159	72.8010
1	16.8599	2.159	36.4005

ADDITIONAL SEISMIC FORCES $\Delta E_{JK}^{(1)}$ FOR SHEAR WALLS 1 AND 6

AS A RESULT OF ROTATION

$$E_{jk}^{(1)} = \pm \frac{k_{jk}^{(1)} a_{jr}}{k_{\varphi k}} M_{tk}^{(1)}$$

Table 19

Level	$M_{tk}^{(1)}$ (kNm)	$k_{\varphi k}$ (kNm)	Shear Wall 1 k_{jk} (kN/m)	Shear Wall 1 a_{jr} (m)	Shear Wall 1 $\Delta E_{jk}^{(1)}$ (kN)	Shear Wall 6 k_{jk} (kN/m)	Shear Wall 6 a_{jr} (m)	Shear Wall 6 $\Delta E_{jk}^{(1)}$ (kN)
19	414.9382	2291.0169	1.053	3.799	0.724	1.739	16.645	5.249
18	568.7021	2291.0169	1.053	3.799	0.993	1.739	16.645	7.185
17	694.0539	2291.0169	1.053	3.799	1.212	1.739	16.645	8.769
16	652.5259	2291.0169	1.053	3.799	1.139	1.739	16.645	8.244
15	611.0021	2291.0169	1.053	3.799	1.067	1.739	16.645	7.720
14	569.4762	2291.0169	1.053	3.799	0.994	1.739	16.645	7.195
13	529.7067	6866.5207	3.162	3.797	0.926	5.205	16.653	6.687
12	488.0431	6866.5207	3.162	3.797	0.853	5.205	16.653	6.161
11	446.3795	6866.5207	3.162	3.797	0.780	5.205	16.653	5.635
10	404.7159	6866.5207	3.162	3.797	0.707	5.205	16.653	5.109
9	363.0580	6866.5207	3.162	3.797	0.635	5.205	16.653	4.583
8	321.3944	6866.5207	3.162	3.797	0.562	5.205	16.653	4.057
7	279.7308	6866.5207	3.162	3.797	0.489	5.205	16.653	3.531
6	242.6757	70354.5039	32.822	3.789	0.429	52.824	16.694	3.041
5	200.2114	70354.5039	32.822	3.789	0.353	52.824	16.694	2.509
4	157.7411	70354.5039	32.822	3.789	0.278	52.824	16.694	1.977
3	115.3001	70354.5039	32.822	3.789	0.203	52.824	16.694	1.445
2	72.8010	70354.5039	32.822	3.789	0.128	52.824	16.694	0.912
1	36.4005	70354.5039	32.822	3.789	0.064	52.824	16.694	0.456

EFFORTS IN SHEAR WALL 6 DUE TO TRANSLATION AND ROTATION

Table 20

	$E_{jk}^{(1)}$ (kN)	Q (kN)	M (kNm)
19	17.0562	17.0562	0
18	23.3765	40.4327	32.4068
17	28.5292	68.9619	145.6183
16	26.8222	95.7841	338.7117
15	25.1153	120.8994	606.9071
14	23.4083	144.3077	945.4255
13	21.9682	166.2759	1349.4870
12	20.2403	186.5162	1815.0595
11	18.5124	205.0286	2337.3049
10	16.7845	221.8131	2911.3850
9	15.0569	236.8700	3532.4617
8	13.3290	250.1990	4195.6977
7	11.6011	261.8001	4896.2549
6	10.5952	272.3953	5629.2952
5	8.7412	281.1365	6392.0020
4	6.8870	288.0235	7179.1842
3	5.0340	293.0575	7985.6500
2	3.1785	296.2360	8806.2110
1	1.5893	297.8253	9517.1774
0			10231.9581

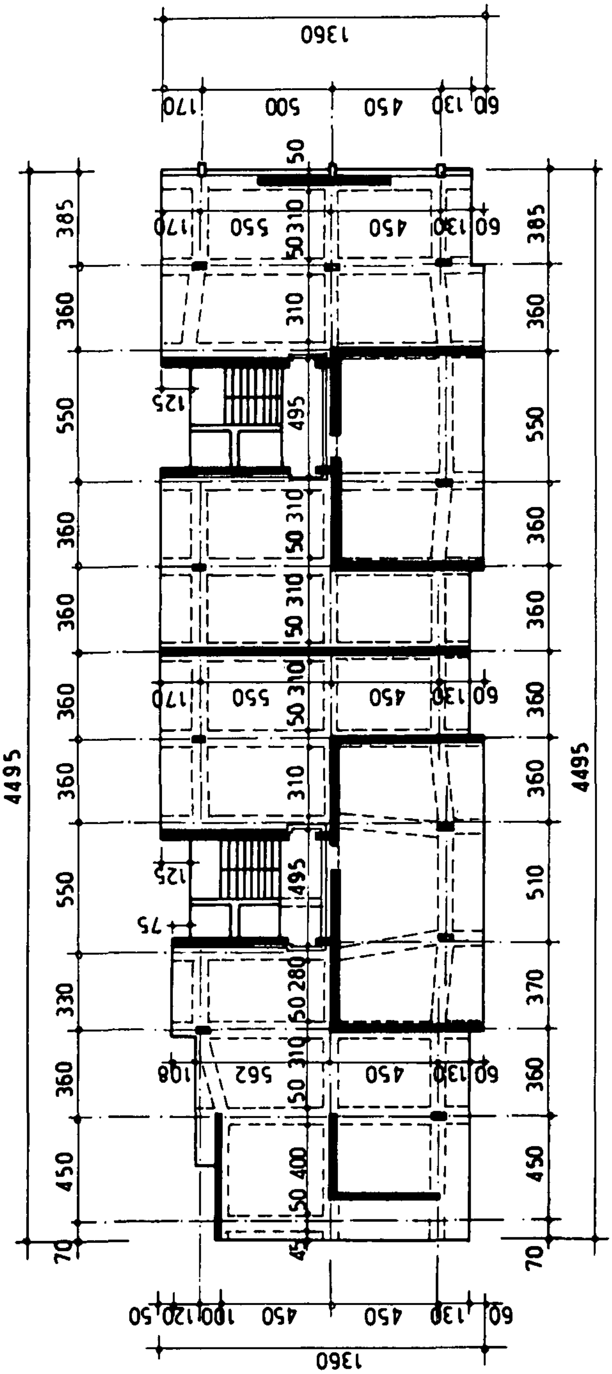
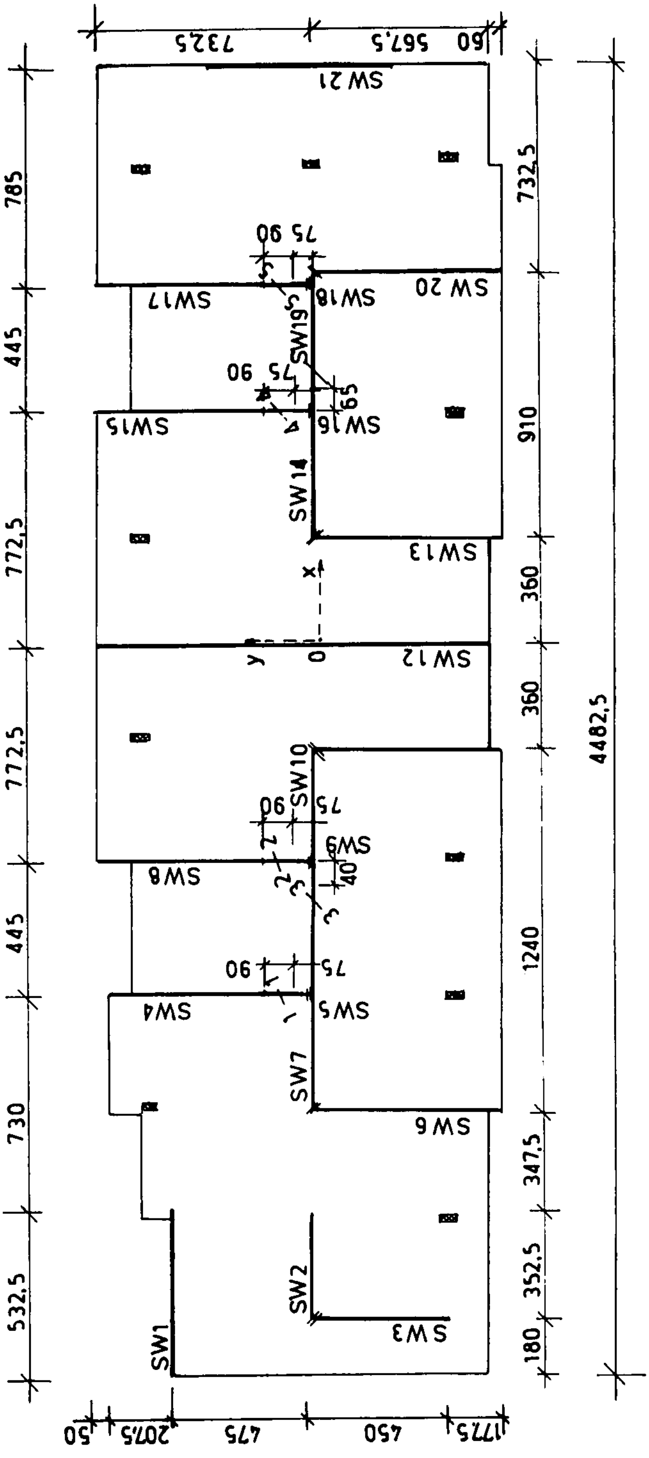
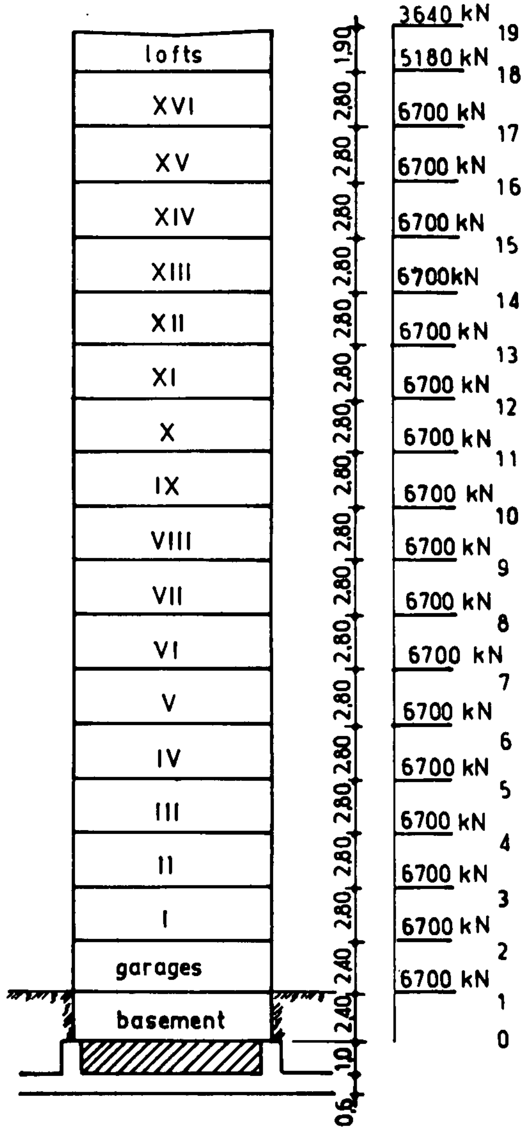

1360
170
500
450
130
60
4495
385
360
550
330
450
70
510
370
495
125
75
310
50
280
400
562
108
120
100

Fig.1



732,5
567,5
60
785
445
772,5
772,5
445
730
532,5
732,5
910
360
360
1240
347,5
352,5
180
4482,5
50
207,5
475
450
177,5
SW1
SW2
SW3
SW4
SW5
SW6
SW7
SW8
SW9
SW10
SW12
SW13
SW14
SW15
SW16
SW17
SW18
SW19
SW20
SW21
x
y
0

Fig. 2


lofts
XVI
XV
XIV
XIII
XII
XI
X
IX
VIII
VII
VI
V
IV
III
II
I
garages
basement
1,90
2,80
2,80
2,80
2,80
2,80
2,80
2,80
2,80
2,80
2,80
2,80
2,80
2,80
2,80
2,80
2,80
2,40
2,40
1,0
0,6
3640 kN
19
5180 kN
18
6700 kN
17
6700 kN
16
6700 kN
15
6700kN
14
6700 kN
13
6700 kN
12
6700 kN
11
6700 kN
10
6700 kN
9
6700 kN
8
6700 kN
7
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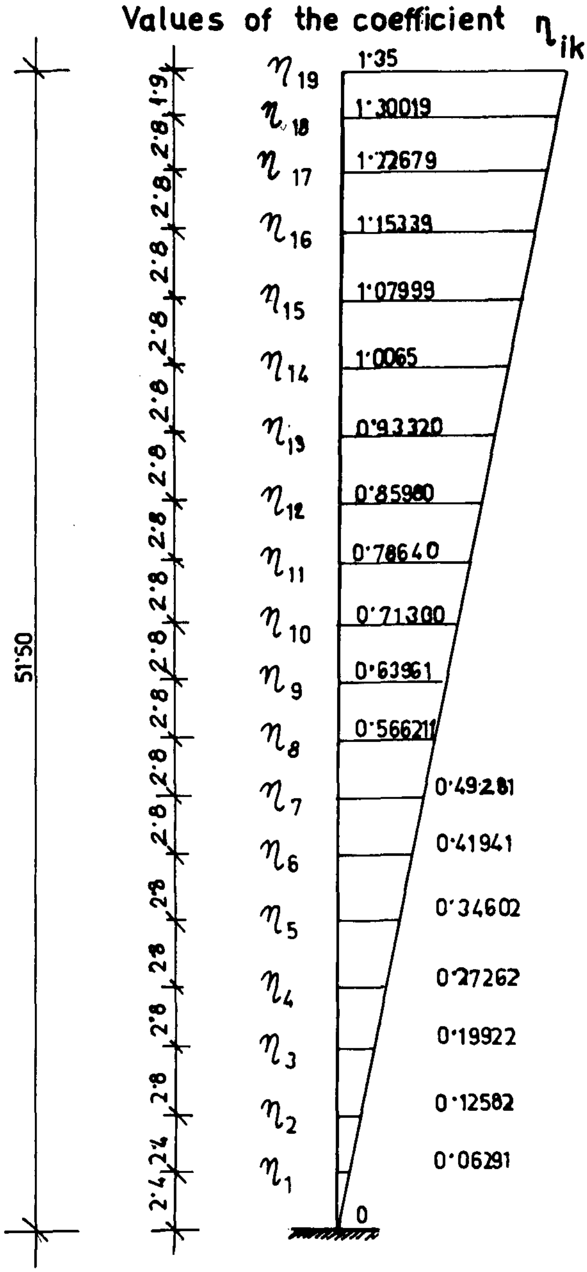
2
6700 kN
1
0

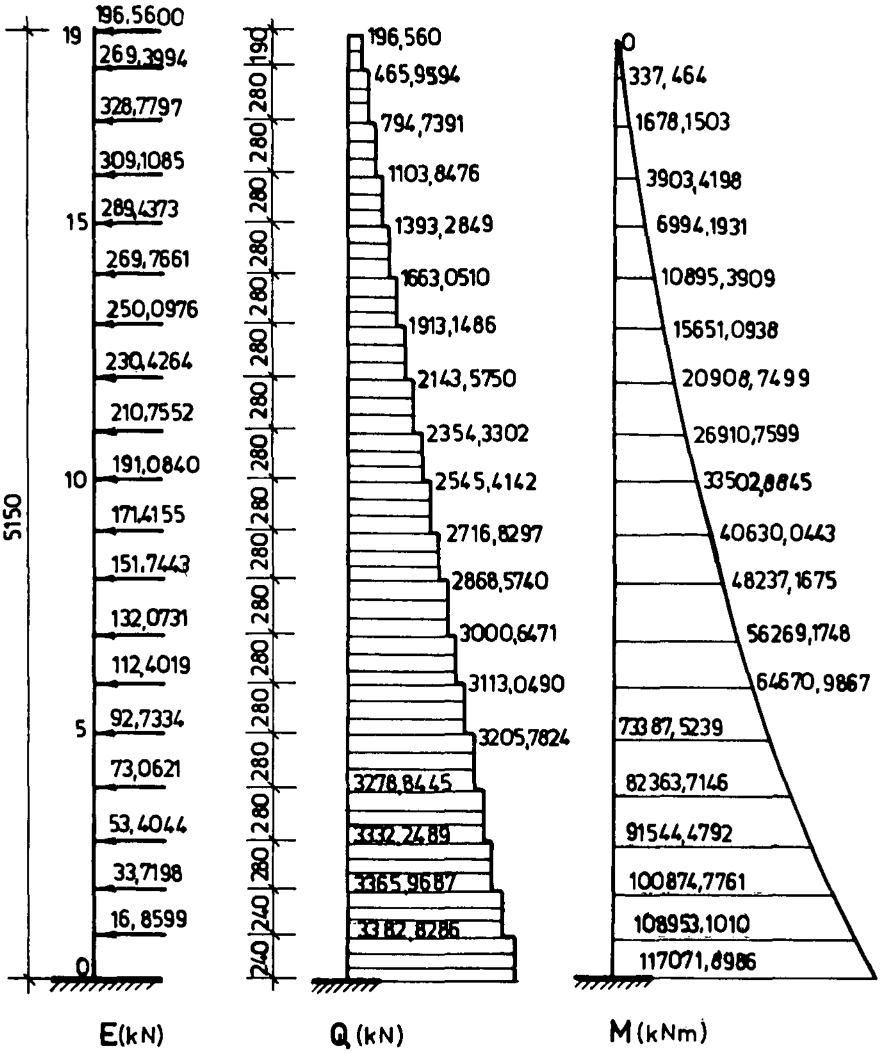
Fig. 3



Values of the coefficient η_{ik}
η_{19} 1·35
η_{18} 1·30019
η_{17} 1·22679
η_{16} 1·15339
η_{15} 1·07999
η_{14} 1·0065
η_{13} 0·93320
η_{12} 0·85980
η_{11} 0·78640
η_{10} 0·71300
η_{9} 0·63961
η_{8} 0·56621
η_{7} 0·49281
η_{6} 0·41941
η_{5} 0·34602
η_{4} 0·27262
η_{3} 0·19922
η_{2} 0·12582
η_{1} 0·06291
0
51·50
1·9
2·8
2·4
2·4

Fig.4

Seismic storey forces and section forces



base shear
coefficient = 2756 %

Fig. 5.

Seismic forces and section forces for shear wall 1 due to translation

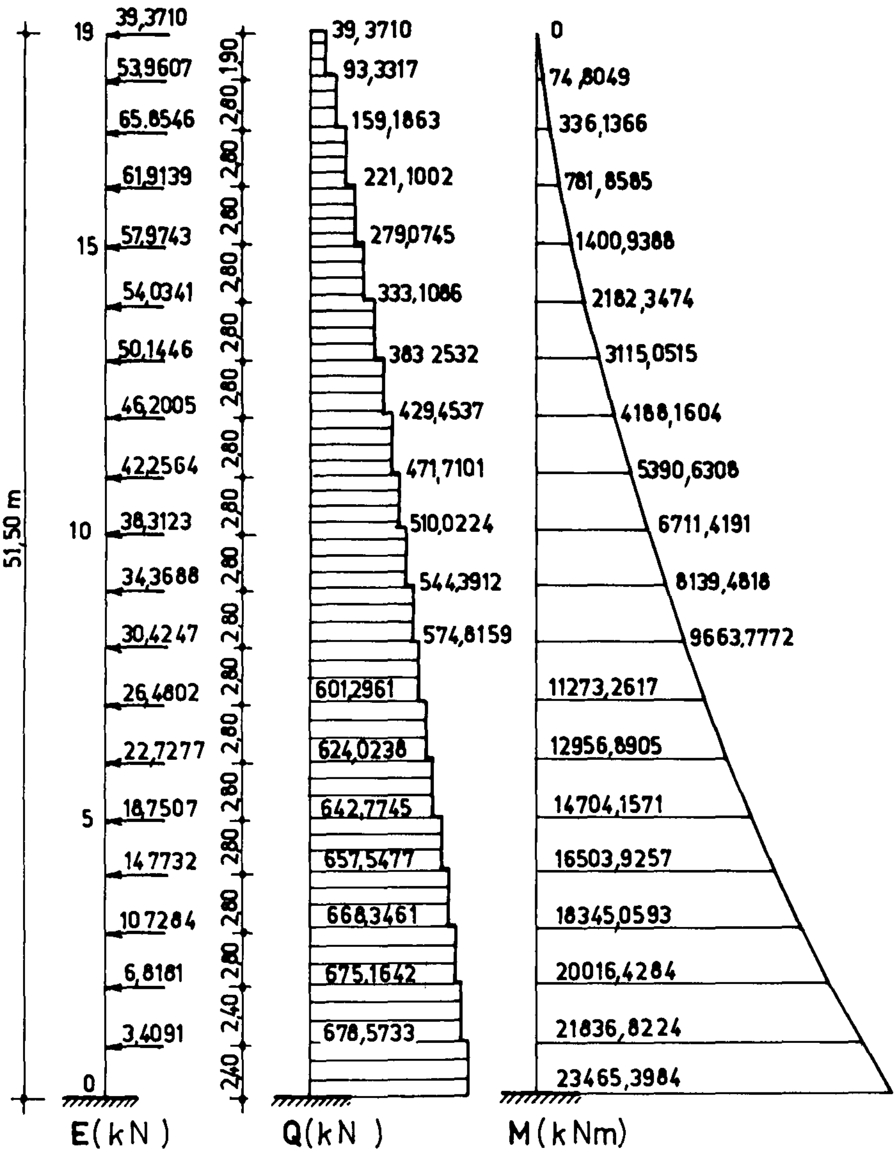


Fig. 6

Seismic forces and section forces for shear wall 6 due to translation

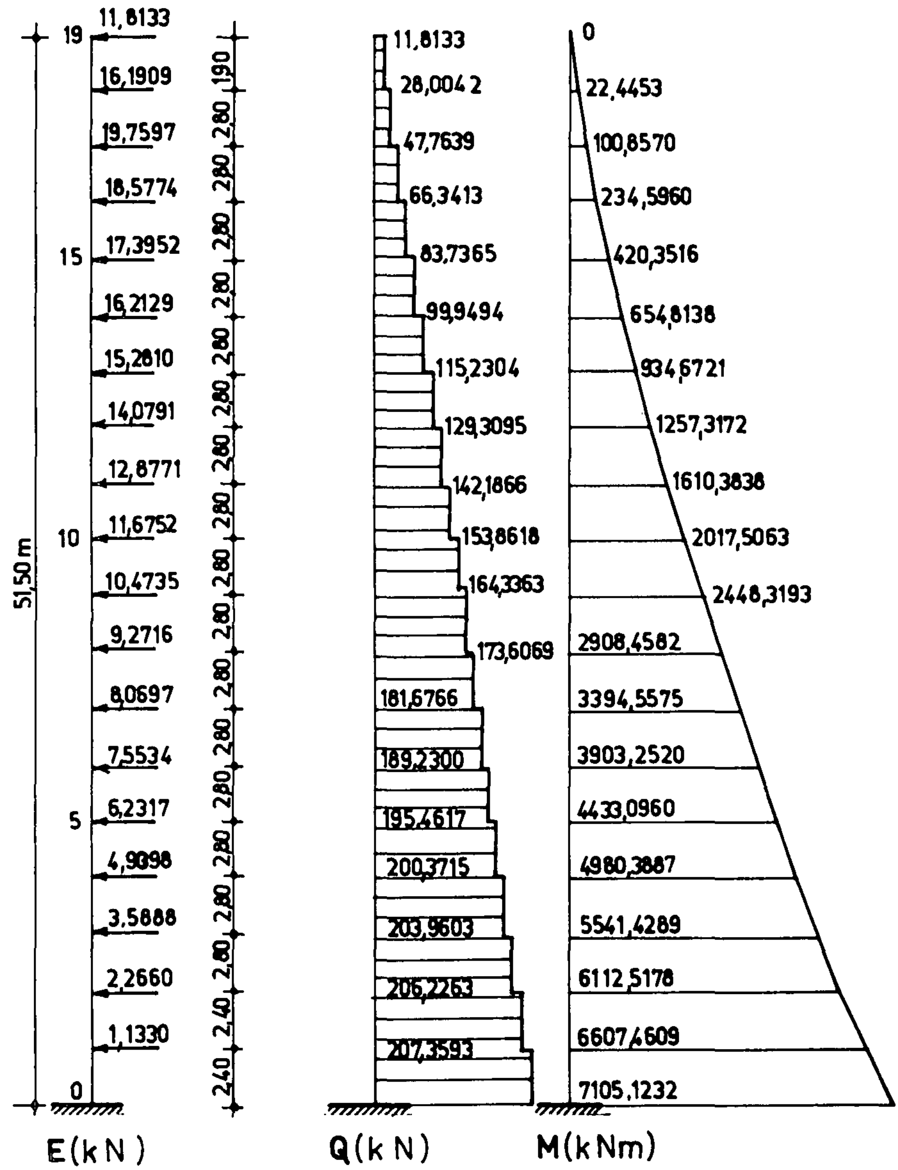


Fig. 7

Seismic forces and section forces for shear wall 1 due to rotation

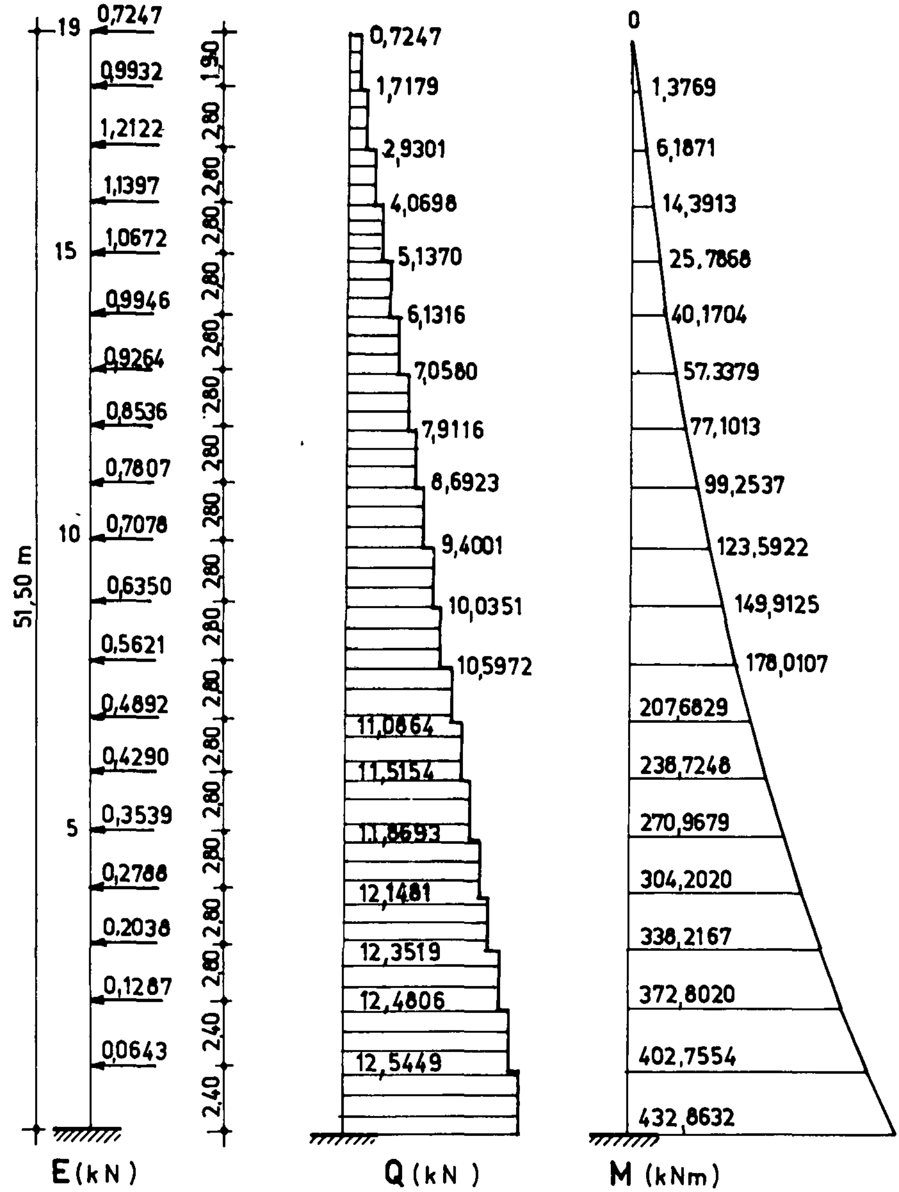


Fig. 8

Seismic forces and section forces for shear wall 6 due to rotation

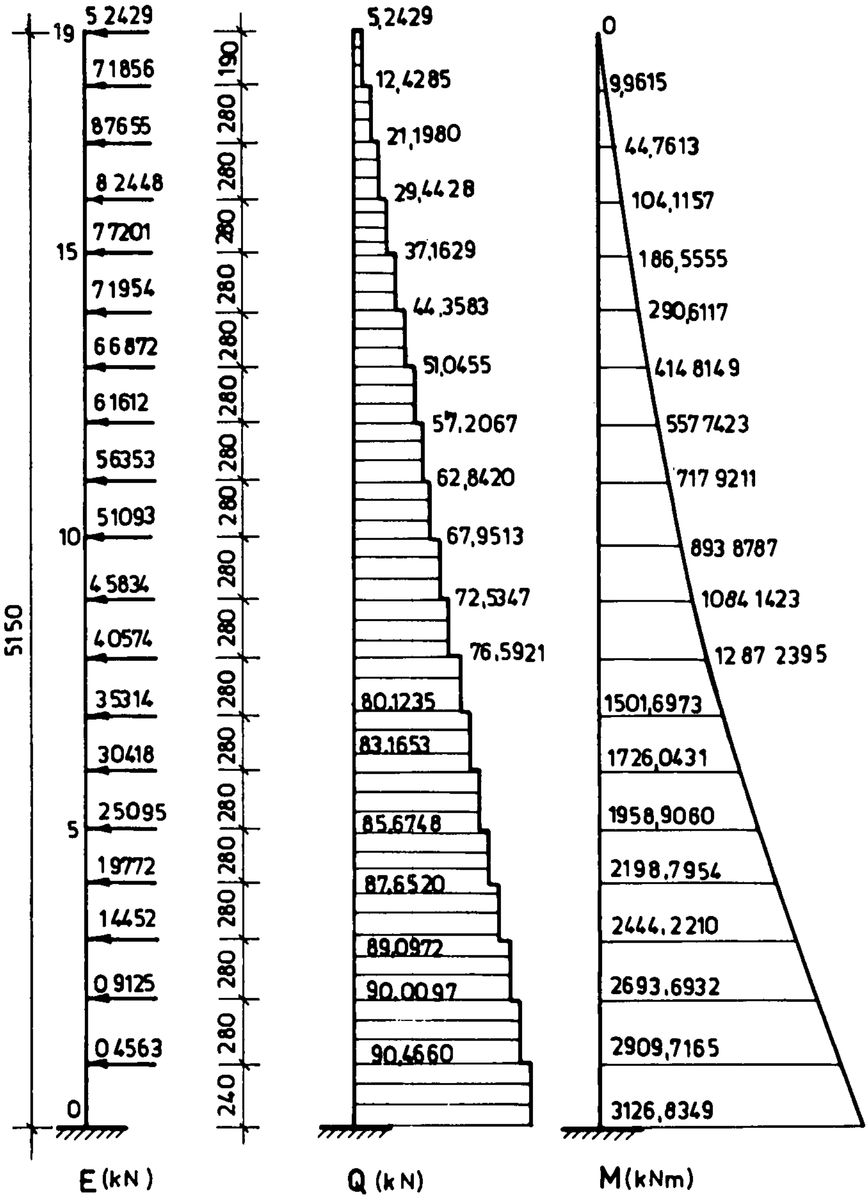


Fig.9.

Total seismic forces and section forces for shear wall 6

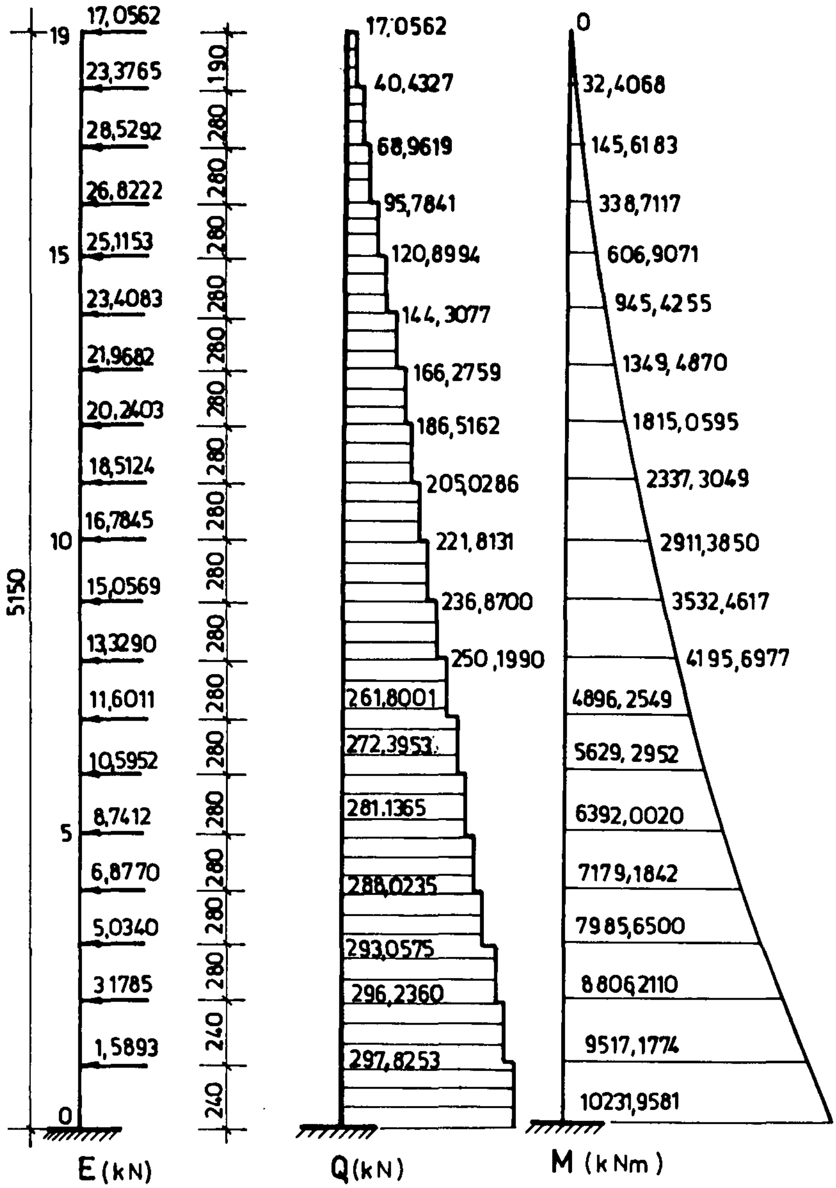


Fig.10.

Reinforcement for SW 6

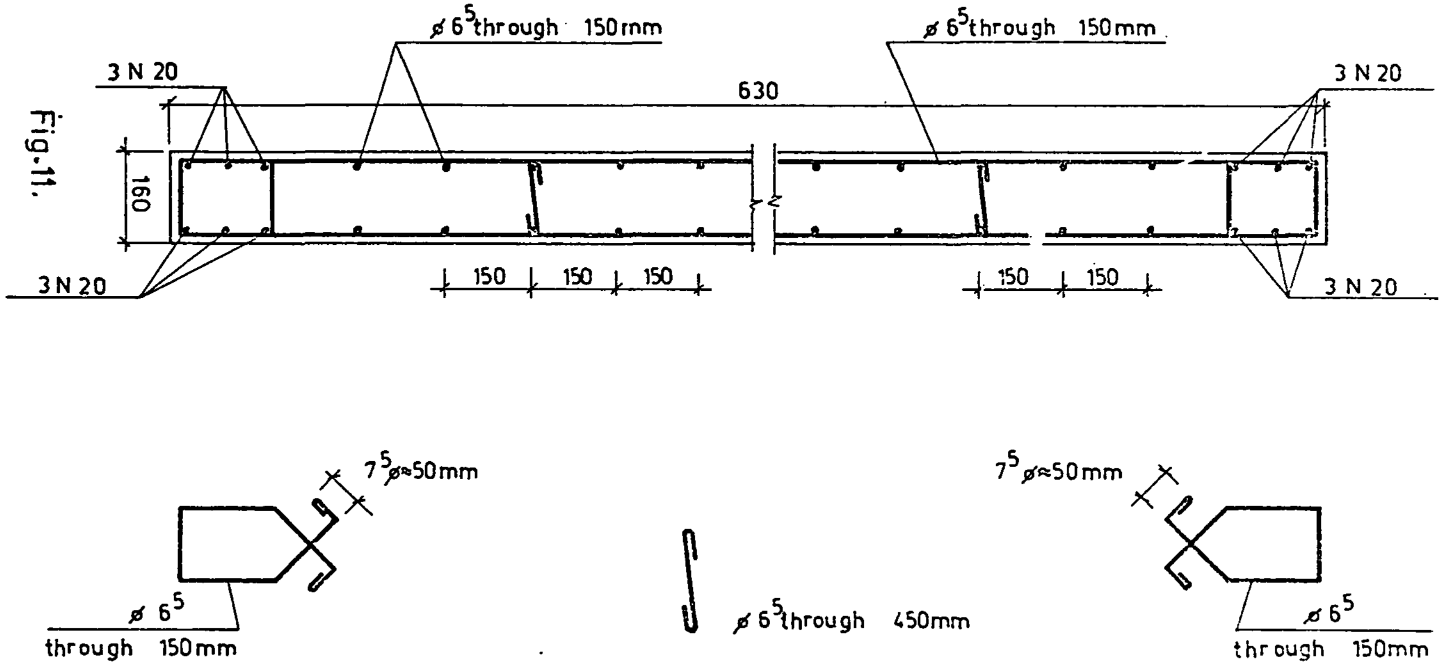
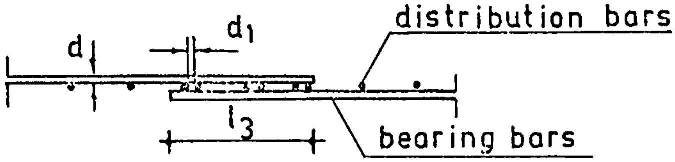
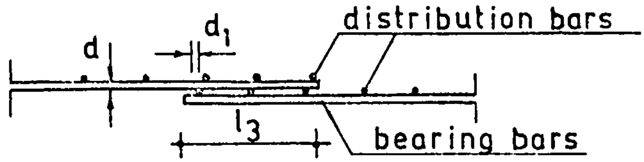


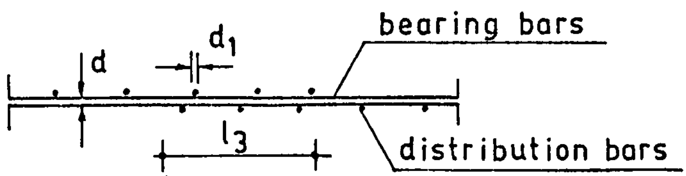
Fig.11.

Overlapping of welded grids from smooth reinforcement in the direction of the bearing bars



Disposition of the distribution bars of the two grids in one plane.

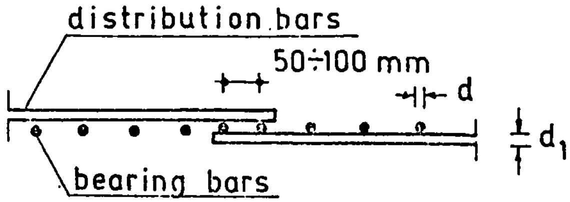




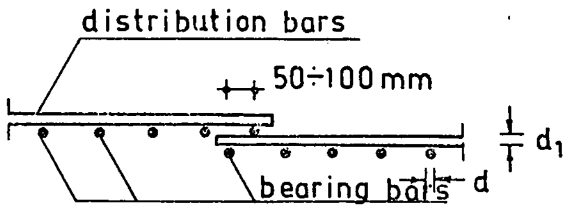
Disposition of the distribution bars in different planes.

Fig.12

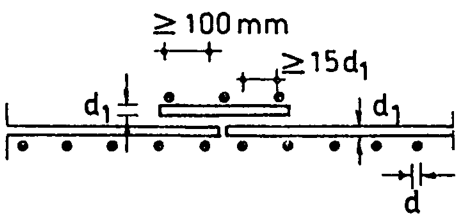
Connection of welded grids in the direction of the distribution bars



The bearing bars are in one plane.



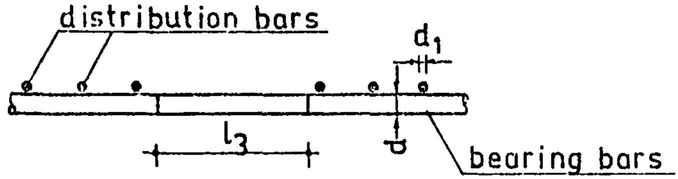
The bearing bars are in two different planes.



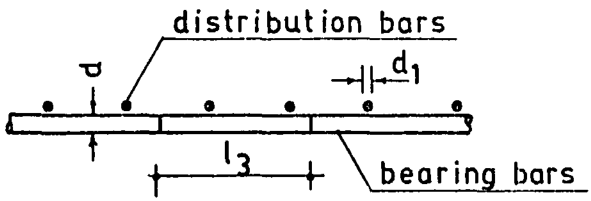
Connection with additional grid.

Fig.13

Connection of welded grids of reinforcement with a periodical profile



A connection in the direction of bearing bars for the case when in the region of the overlapping the transverse bars lack in both grids

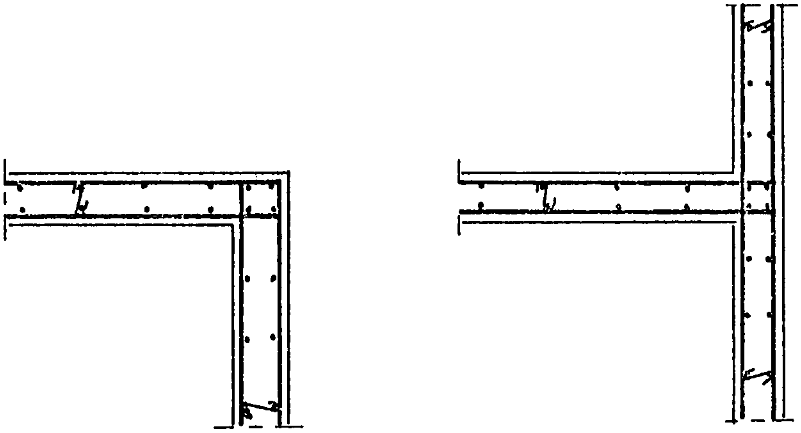


The same for the case of transverse bars in one of the connected grids.

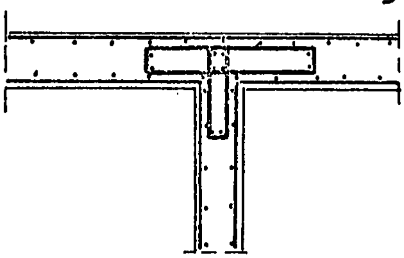
Fig.14

Scheme for columns in the reinforced concrete walls

reinforcement of walls with separate horizontal and vertical bars



reinforcement of walls with welded grids



reinforcement of walls surrounded with belts

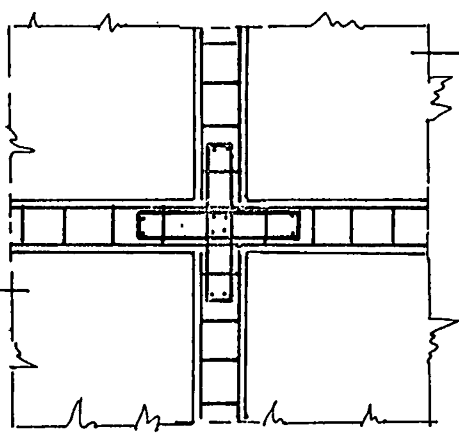


Fig. 15

Anchorage of reinforcement in the end fields of slabs at the last storey

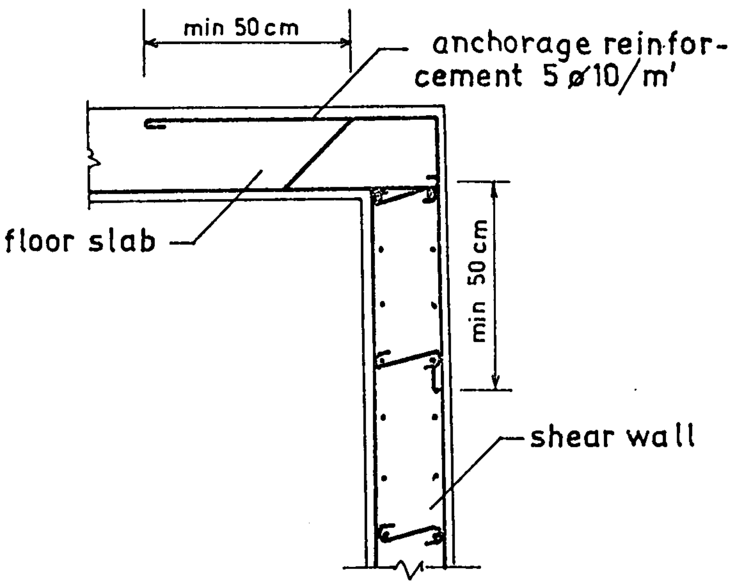
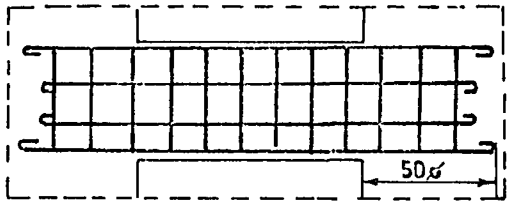
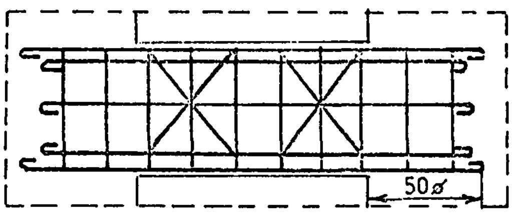


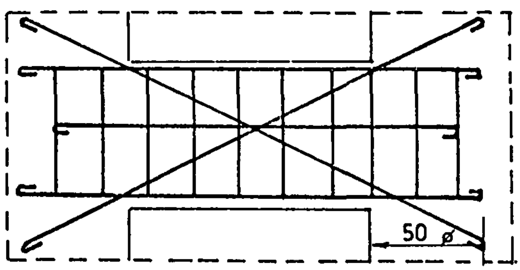
Fig. 16

Scheme for reinforcement of the lintels



reinforcement of straight bars
and stirrups





reinforcement of straight bars,
stirrups and bent bars

Fig. 17

YUGOSLAV DESIGN EXAMPLE

MIXED FRAME-SHEAR WALL STRUCTURE

(DUAL SYSTEM)

1. Introduction
2. Determination of Actions
3. Structural Analysis
4. Stiffness Verification
5. Calculation of Reinforcement

1. INTRODUCTION

The aseismic design of reinforced concrete structures should be based on methods verified through static and dynamic analysis.

Because of the uncertainties related to magnitudes and characteristics of the future earthquakes, usually it is economically unjustified to design buildings so that the stresses are in the elastic range during strong earthquakes.

This example shows the design procedure of one reinforced concrete building constructed in mixed frame-shear wall system. It is worth to note that besides taking into account relevant codes for design, the procedure in this example is improved considering ultimate flexural and shear capacities of the structural members.

1.1. Description of Building Structure

The residential building of total height of eight stories 2.89 m each, has been constructed in "Aerodrom II" settlement in Skopje, for seismic performance category C.

It is a reinforced concrete frame structure with framed reinforced concrete walls in the central part of the building (Fig.1). Structural analysis was carried out for vertical and seismic loads in both orthogonal directions, while the frames R5 and R6 were taken as calculation examples and representative parts of the whole structure.

Seismic analysis will be carried out applying the provisions of the aseismic design Code [1] being in force since 1982.

Proportioning of structural elements for both vertical and seismic loads will be accomplished, applying standards and provisions specified for concrete and reinforced concrete [2] , applying also the ultimate strength capacity concept.

1.2. Material

The material used in this structure has the following characteristics:

Normal weight concrete	C 30 (f_{ck} = 30 MPa)
Mild structural steel	S 240 (f_{yk} = 240 MPa)
Welded reinforced nets	ČBM ($f_{y,w}$ = 500 MPa)

1.3. Structural Set-up of Frames

The frame elements are of prismatic shape and the joints are treated as stiff zones of finite dimensions (Fig. 2).

2. DETERMINATION OF ACTIONS

The building was designed according to the existing design Codes of Yugoslavia [1] and [2].

2.1. Vertical Loads

The floor structure was designed for static and live vertical loads as follows:

Static load	$g = 6.50$ kN/m^2
Live load	$p = 1.50$ kN/m^2
Total	$q = 8.00$ kN/m^2

2.2. Seismic Loads

The building structure was preliminarily analyzed in the elastic range applying TABS [3] computer program using vertical loads and equivalent seismic forces determined by dynamic analysis. For comparison purposes, a calculation example of seismic forces according to the Provisions [1] , for seismicity of IX degrees and soil category II has been given.

2.2.1. Equivalent Static Loads

The dynamic scheme of the structure with concentrated loads and masses at floor levels is given below:

$$G_1 = 5610 \text{ kN},\ G_2 - G_8 = 5415 \text{ kN}$$

The total seismic horizontal force S has been determined according to the equation:

$$S = K \cdot G \quad \text{, where}$$

K - total seismic coefficient

G - total weight of the building

The total seismic coefficient K will be determined according to equation

$$K = K_o \cdot K_s \cdot K_d \cdot K_p, \text{ where}$$

K_o - coefficient of building category

K_s - seismic intensity coefficient

K_d - dynamic coefficient

K_p - ductility and damping coefficient

The building category coefficient is determined according to the importance of the building and the Provisions [1] classify the buildings into four categories. For residential buildings the coefficient $K_o = 1.0$.

The seismic intensity coefficient is:

for VII degree $K_s = 0.025$
for VIII degree $K_s = 0.05$
for IX degree $K_s = 0.10$

The dynamic coefficient is determined according to the soil category:

Soil category I,
(rocks, marlstone, well cemented conglomerates) - $K_d = \frac{0.50}{T}$ $(0.33 < K_d < 1.0)$

Soil category II
(gravel, sand, hard clay) - $K_d = \frac{0.70}{T}$ $(0.47 < K_d < 1.0)$

Soil category III
(slightly compacted and soft soil) - $K_d = \frac{0.90}{T}$ $(0.60 < K_d < 1.0)$

T - natural period of the building

The coefficient of ductility and damping depends upon the type of the structure and amounts from 1.0 to 2.0. For modern reinforced concrete and steel structures it is $K_p = 1.0$.

The natural period of the building in the considered direction is $T_1 = 0.50$ sec.; thus the dynamic coefficient is $K_d = 1.0$.

The total horizontal seismic force is:

$S = K_o \cdot K_s \cdot K_d \cdot K_p \cdot G = 1.0 \cdot 0.10 \cdot 1.0 \cdot 1.0 \cdot 43515 = 4351.5$ kN

The distribution of the total seismic force along the height of the building can be done applying 1) the dynamic method for civil engineering structures and 2) approximate equation method depending upon the height of the building. For buildings five-story high, the distribution of the total seismic force follows the equation:

$$S_i = S \frac{G_i \cdot H_i}{\sum_{i=1}^{n} G_i \cdot H_i} ,$$ where

S_i - seismic force of the i-th story
G_i - weight of the i-th story
H_i - height of the i-th story from the building base

For buildings taller than five stories distribution of the total seismic force is done so that 0.15 S is taken as concentrated force at the top of the building while the rest of 0.85 S is distributed according to the above equation, as follows:

$$S_8 = 0,15 \cdot 4351,5 + 0,85 \cdot 4351,5 \frac{5415 \cdot 8 \cdot 2,89}{(35 \cdot 5415 + 5610) \cdot 2,89} =$$

$$= 653 + 3698,5 \frac{43320,0}{195135,0} = 653 + 821 = 1474,0 \text{ kN}$$

$S_7 = 3698,5 \cdot \frac{5415 \cdot 7}{195135} = 718,4$ kN 2192,4 (story shear)

$S_6 = 3698,5 \cdot \frac{5415 \cdot 6}{195135} = 615,8$ kN 2808,2

$S_5 = 3698,5 \cdot \frac{5415 \cdot 5}{195135} = 513,2$ kN 3321,4

$S_4 = 3698,5 \cdot \frac{5415 \cdot 4}{195135} = 410,5$ kN 3731,9

$S_3 = 3698,5 \cdot \frac{5415 \cdot 3}{195135} = 307,9$ kN 4039,8

$S_2 = 3698.5 \cdot \frac{5415 \cdot 2}{195135} = 205.3$ kN 4245.1

$S_1 = 3698.5 \cdot \frac{5610 \cdot 1}{195135} = 106.4$ kN 4351.5 kN (base shear)

2.2.2. Dynamic Response Method

The structural dynamic analysis was carried out applying the computer program VIBR [4] which uses nonlinear bilinear diagram of the shear forces - and the story drift of the structure. The nonlinear dynamic analysis was carried out for ground acceleration at the site of $a_{max} = 0.23$ g, and acceleration time histories were selected of El Centro 1940 earthquake, N-S component and Parkfield 1966 earthquake, N-S component.

Acceleration level of 0.23 g corresponds to the design earthquake while the maximum displacements causing this earthquake should be within certain limits, the same as the ductility of separate stories. According to Provisions [2], the maximum story drift should not be larger than $\Delta_{max} = h_i/150$, where h_i is the height of the i-th story in cm. The ratio between maximum story drift Δ_{max} and the yield displacement of the main reinforcement, Δ_y, has been defined as displacement ductility, $D_\Delta = \Delta_{max}/\Delta_y$. The reinforced concrete members like beams, columns, walls and joints have different deformational capacities which result in different ductilities. The aseismic structures should have sufficient ductility, which is one of the objectives of the aseismic design. On the other hand, the dynamic response of structures to actual earthquakes will show which ductility relates to which earthquake. In this example it was adopted that favourable structural response is obtained if the required ductility is within 3.0 to 3.5 limits. For further design, the proportioning of elements and the design of reinforcement details should comply with the required ductility level.

The results of the dynamic analysis for x-direction will be given hereafter.

The story stiffnesses of the building were computed applying TABS[3] program which, for calculation of lateral story displacements under given lateral loads, takes into account the deformations due to bending, shear and axial forces of columns and walls and bending deformations for beams.

The initial stiffness of the bilinear Q-Δ diagram has been determined by the relation $K_e = Q/\Delta e$, while the stiffness after the yield point is defined as 10% of the initial stiffness.

The dynamic analysis results will be presented in Table 2.1:

Table 2.1 Results of the Dynamic Analysis

Story	Yield displacement in cm.	Maximum displacement		Ductility	
		El Centro 1940, N-S	Parkfield 1966, N-S	El Centro 1940, N-S	Parkfield 1966, N-S
1	0.097	0.320	0.257	3.30	2.65
2	0.166	0.528	0.442	3.18	2.66
3	0.204	0.641	0.575	3.14	2.82
4	0.224	0.663	0.645	2.96	2.88
5	0.238	0.677	0.723	2.84	3.04
6	0.232	0.650	0.763	2.80	3.29
7	0.220	0.633	0.803	2.87	3.65
8	0.206	0.548	0.753	2.66	3.66

The maximum story displacements and ductilities are within the allowable limits while the level of horizontal seismic loads for each story has been determined according to the story shear force Q_y in the Q - Δ diagram. In this way, the required level of the seismic shear force of the new building has been determined under the condition that yielding takes place in the main vertical reinforcement of columns and walls.

3. STRUCTURAL ANALYSIS

The further design procedure is in the analysis for vertical and horizontal seismic loads in order to obtain M, Q and N values in each structural element as well as for proportioning and design of reinforcement in these members.

The following table gives comparisons between seismic shear forces according to the Provisions [1] and as obtained by the dynamic analysis, as well as parts of the shear forces belonging to walls and frames in x-direction (Table 3.1).

Table 3.1 Seismic Shear Forces

Story	Q_s, Code kN	Q_s, dyn. kN	$\frac{Q_{s,c}}{Q_{s,d}}$	$Q_{s,w}$	$Q_{s,F}$	$\frac{Q_{s,F}}{Q_{s,d}}$
1	4351,5	6100	0.71	4010	2090	0.34
2	4245,1	5600	0.76	3300	2300	0.41
3	4039,8	5050	0.80	2816	2234	0.44
4	3731,9	4450	0.84	2000	2450	0.55
5	3321,4	3750	0.89	2012	1738	0.46
6	2808,2	2950	0.95	1266	1684	0.57
7	2192,4	2050	1.07	570	1480	0.72
8	1474,0	1100	1.34	-470	1570	1.43

The seismic shear force defined by the Codes, $Q_{s,c}$, is larger by 34% at the top of the building compared to the seismic force obtained by the dynamic response of the structure $Q_{s,d}$. The reverse is observed at the bottom floor; the seismic shear force based on the dynamic response is higher by 29%. Since the dynamic response of structures has advantages over the definition of seismic loads by empirical equations, the dynamic response method was adopted for determination of the required seismic loads.

The shear force distribution in certain structural elements was accomplished applying the TABS [3] computer program, and it is peculiar that negative seismic force is obtained in the walls of the top floor. Fig. 3 shows shear force distribution in frames and walls along the height of the building.

Moment diagrams due to seismic forces for frames R_5 and R_6 are shown in Fig. 4. Axial force diagrams due to vertical loads, N_v and the combination of vertical and horizontal load $N_v + N_s$ are shown in Fig. 5 for frame R_6 taken as an example for proportioning and design of reinforcement.

3.1. Member Forces According to Muto's Method

A case of a shear wall connected on all sides with frames will be studied. The deflection characteristics in such a case can be considered as deformations due to bending and shear in a free standing shear wall being restrained by beams connected to the wall.

The method described here is a modified version in which, using the lateral forces applied to the entire framework of the building, the shear forces and bending moments of shear wall and columns are calculated directly.

(1) Assumptions for Calculation

The main assumptions for calculations are the following:

a) For beams connected to the wall in the same plane, the rotation angles and vertical deformations of joints on the opposite side are considered to be zero.

b) The shear forces distributed at each story of frame columns, other than the shear wall under consideration, are proportional to the relative displacement δ or rotation angle R ($Q = D.\ \delta = D.\ Rh$, D is the shear distribution coefficient).

The resisting moment M_R of an adjoining beam when there is a rotation angle θ, at the wall, will be as follows (see Fig. 5b):

$$M_R = M_A + Q_{AB} \cdot L_a = M_A + (M_A + M_B) \cdot L_a / L \tag{3.1}$$

where:

$$M_A = 2EK\ K_b(2\ \theta_A - 3\ R_{AB}) \tag{3.2}$$

$$M_B = 2EK\ K_b(\theta_A - 3\ R_{AB})$$

The rotation angle of the beam is:

$$R_{AB} = -L_a \cdot \theta\ /L \tag{3.3}$$

So that inserting these in Eq. 3.2. and further in Eq. 3.1., the following equation is obtained:

$$M_R = 6EK\theta\ .\ K_{be}, \tag{3.4}$$

$$\text{where}\quad K_{be} = \left[\frac{2}{3} + 2\,\frac{L_a}{L} + 2\,\left(\frac{L_a}{L}\right)^2\right] .\ K_b \tag{3.5}$$

(2) Method for Calculation

Using the above effective stiffness ratios for beams connected to the wall, the stiffness ratios for other beams, columns and walls, the shear distribution coefficients for frame columns and the joint rotation angle of the wall are obtained from the following equation:

$$K_{wn}(2-3B_n)\ \phi_{n-1}+\left[K_{wn}(4-3B_n)+K_{wn+1}(4-3B_{n+1})+6\Sigma K_{ben}\right]\phi_n +$$

$$+K_{wn+1}(2-3B_{n+1})\ \phi_{n+1}=\frac{B_n}{S_n}\ Q_n\cdot h_n+\frac{B_{n+1}}{S_{n+1}}\,Q_{n+1}\cdot h_{n+1} \qquad (3.6)$$

where

$$B_n = \frac{1}{1+\Sigma D_{cn}/S_nK_{wn}}$$

$$S_n = 1+\frac{\Sigma D_{cn}\ 12\,\kappa\,EK}{G\cdot A_{wn}.hn}, \qquad \text{where}$$

ΣD_{cn}: is a sum of shear distribution coefficients of columns of n-th story

K_{wn}: stiffness ratio of n-story wall

For top story, the equation of the joint rotation angle is

$$K_{wm}(2-3B_m)\phi_{m-1}+\left[K_{wm}(4-3B_m)+6\Sigma K_{bem}\right]\ \phi_m=\frac{B_m}{S_m}\,Q_m\cdot h_m \qquad (3.7)$$

For the first story (wall base fixed) is:

$$K_{m2}(2-3B_2)\ \phi_2+\left[K_{w2}\ (4-3B_2)+K_{w1}(4-3B_1)+6\Sigma K_{b1_1}\right]\phi_1 =$$
$$=\frac{B_2Q_2h_2}{S_2}+\frac{B_1Q_1h_1}{S_1} \qquad (3.8)$$

The solution of the above equations is obtained relatively easily, as the solution of simultaneous linear equations consisting of three terms.

From the rotation angles of the wall obtained, distributions of shear forces and bending moments of the wall and frames are calculated from the following equations:

$$Q_{wn}=Q_n-\frac{2\Sigma D_{cn}}{hn}\cdot 3\psi_n \qquad (3.9)$$

$$M_{R_n}=E\Sigma K_{b1n}\cdot\phi_n \qquad (3.10)$$

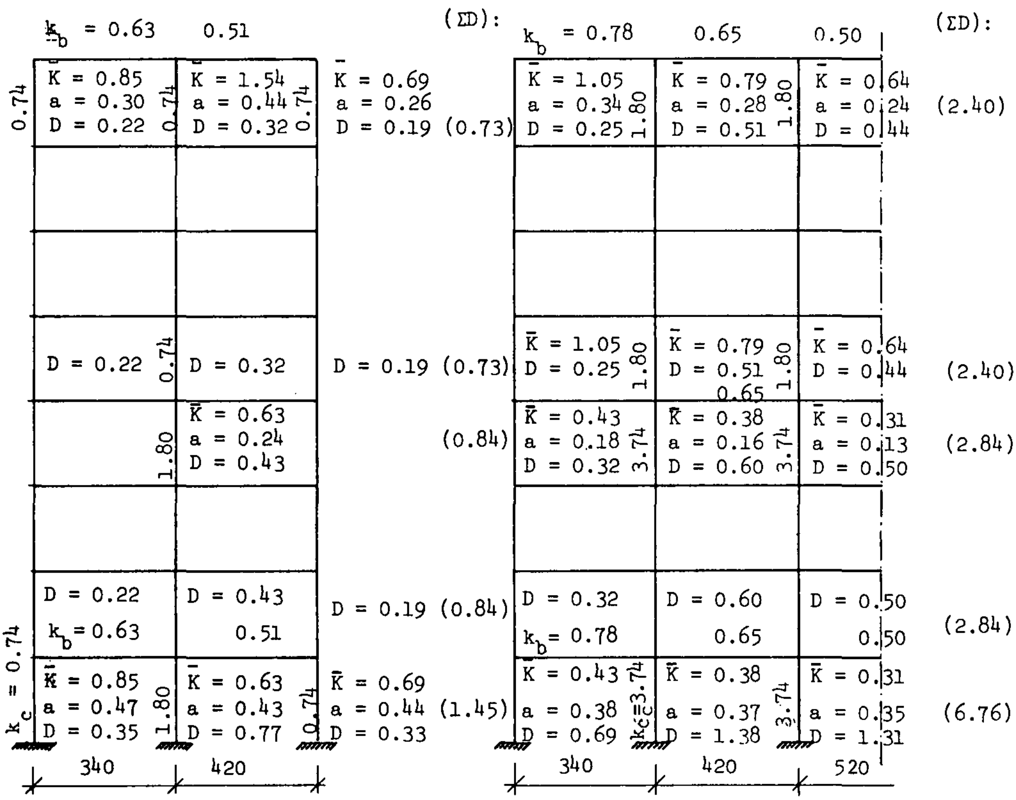
$$\text{where}\quad 3\psi_n=\frac{B_n}{2}\left(3\phi_n+3\phi_{n-1}+\frac{Q_n\cdot h_n}{S_nK_{wn}}\right)$$

$$M_{n,n-1} = K_{wn}(2\phi_n + \phi_{n-1} - 3\psi_n) \qquad (3.11)$$

$$M_{n-1,n} = K_{wn}\,(2\phi_{n-1} + \phi_n - 3\psi_n) \qquad (3.12)$$

3.2 Distribution Coefficients for Frames

Frame R_4 — Frame R_5



For frames:

$$\bar{K} = \frac{\Sigma \cdot K_b}{2 \cdot K_c}\,, \qquad K_{b1} = \frac{26.67}{34.0} = 0.78, \quad K_{b2} = \frac{26.67}{42.0} = 0.65$$

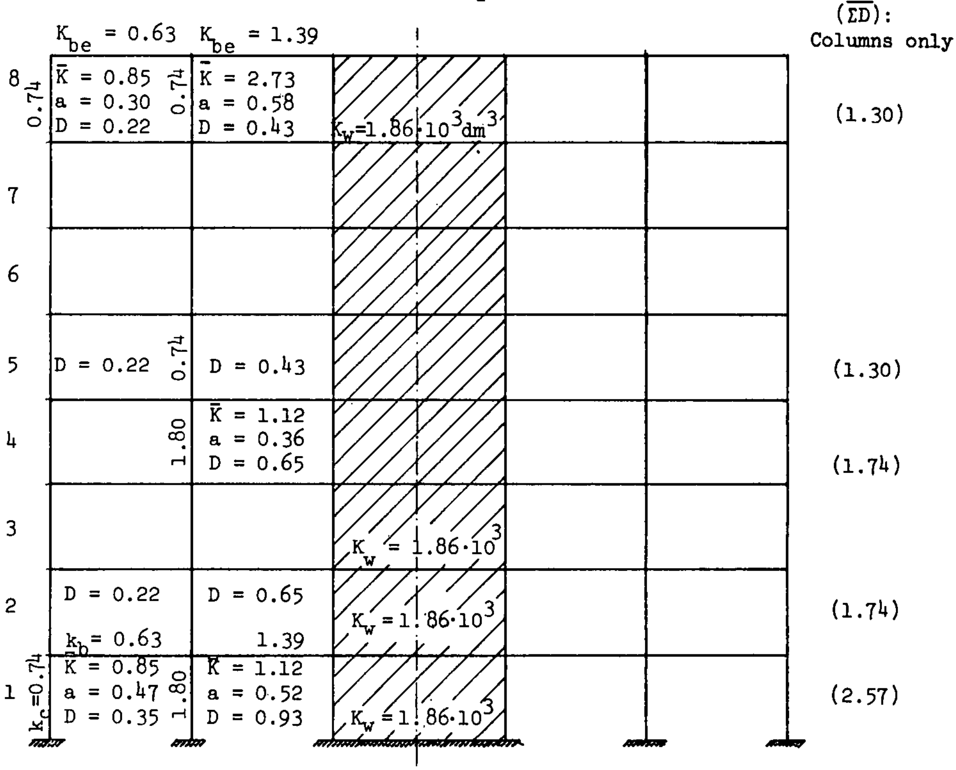
$$a = \frac{\bar{K}}{2+\bar{K}}\,,$$

$$a = \frac{0.5+\bar{K}}{2+\bar{K}}\,, \quad \text{for first story}\,,$$

$$D = a \cdot K_c$$

Frame R_6

$$K_{be} = \left[\frac{2}{3} + 2\,\frac{2.65}{4.20} + 2\left(\frac{2.65}{4.20}\right)^2 \right] \times 0.51 = 1.39\ dm^3$$



$\Sigma D_{c8} = 4 \cdot 0{,}73 + 2 \cdot 2{,}40 + 2 \cdot 1{,}30 = 10{,}32$

$\Sigma D_{c4} = 4 \cdot 0{,}84 + 2 \cdot 2{,}84 + 2 \cdot 1{,}74 = 12{,}52$

$\Sigma D_{c1} = 4 \cdot 1{,}45 + 2 \cdot 6{,}76 + 2 \cdot 2{,}57 = 24{,}46$

$\Sigma D_{c8} = 0{,}5^{*} \cdot 10{,}32 = 5{,}16 \cdot 10^{3**}$

$\Sigma D_{c4} = 0{,}5 \cdot 12{,}52 = 6{,}26 \cdot 10^3$

$\Sigma D_{c1} = 0{,}5 \cdot 24{,}46 = 12{,}23 \cdot 10^3$

* one of the two R_6 frames is considered

** 10^3 is equal to the standard stiffness

3.3. Shear and Bending in Shear Wall

Table 3.2 $K_o = 10^3$

n	A_{wn} cm^2	h_n cm	ΣD_{cn}	K_{wn}	S_n	B_n
8	$12,17\cdot10^3$	289	5.16	$1.86\cdot10^3$	1.0486	0.9974
7	"	"	"	"	"	"
6	"	"	"	"	"	"
5	"	"	"	"	"	"
4	"	"	6.26	"	1.0589	0.9968
3	"	"	"	"	"	"
2	"	"	"	"	"	"
1	"	"	12.23	"	1.1151	0.9941

$$S_n = 1 + \Sigma D_{cn} \frac{27.6 \; . \; 1.2}{A_{wn} \; . \; h_n}$$

$$B_n = \frac{1}{1 + \Sigma D_{cn}/S_n \; . \; K_{wn}}$$

Table 3.3

n	$3B_n$	$2-3B_n$	$K_{wn}x$ $x(4-3B_n)$	$6\Sigma K_{ben}$	a_n	b_n
8	2.9922	-0.9922	$1.875\cdot10^3$	$16.68\cdot10^3$	-1.845	1.892
7	"	"	"	"	"	3.767
6	"	"	"	"	"	"
5	"	"	"	"	"	"
4	2.9904	-0.9904	1.878	"	-1.842	3.770
3	"	"	"	"	"	3.773
2	"	"	"	"	"	"
1	2.9823	-0.9823	1.893	"	1.827	3.788

$$a_n = K_{wn}(2-3B_n)$$

$$b_n = K_{wn+1}(4-3B_{n+1}) + K_{wn}(4-3B_n) + 6\Sigma K_{ben}$$

Table 3.4

n	Q_n kN	$Q_n \cdot h_n$ kN·cm	$\frac{B_n}{S_n}$	$\frac{B_n}{S_n} Q_n h_n$	Load Term	ϕ_n
8	550	$159 \cdot 10^3$	0.9512	$151.24 \cdot 10^{-3}$	$151.24 \cdot 10^{-3}$	5.866
7	1025	296	"	281.56	432.80	5.934
6	1475	426	"	405.21	686.77	6.014
5	1875	542	"	515.55	920.76	5.973
4	2225	643	0.9414	605.29	1120.84	5.692
3	2525	729	"	686.28	1291.57	5.067
2	2800	809	"	761.59	1447.87	3.987
1	3050	881	0.8915	785.40	1546.99	2.331

Table 3.5

n	$3\phi_n$	$S_n \cdot K_{wn}$	$Q_n \cdot h_n$	$\frac{Q_n \cdot h_n}{S_n \cdot K_{wn}}$	$\frac{B_n}{2}$	$6EK_oR_n$
8	17.598	$1950 \cdot 10^3$	$159 \cdot 10^3$	$81.54 \cdot 10^{-3}$	0.4987	17.695
7	17.802	"	296	151.80	"	17.951
6	18.042	"	426	218.46	"	18.043
5	17.919	"	542	277.95	"	17.591
4	17.076	1970	643	326.40	0.4984	16.250
3	15.201	"	729	370.05	"	13.722
2	11.961	"	809	410.66	"	9.651
1	6.993	2074	881	424.78	0.4971	3.687

$$6EK_oR_{mn} = 3\psi_{mn} = \frac{B_n}{2}\left(3\phi_n + 3\phi_{n-1} + \frac{Q_n \cdot h_n}{S_n \cdot K_{wn}}\right)$$

Table 3.6

n	Q_n kN	$\Sigma Dc_n \frac{2}{h_n}$	$\frac{2\Sigma Dc_n}{h_n} 3\psi_n$	Q_{wn} kN	ΣK_{ben}	M_{Rn} kN·cm
8	550	35.71	632	-82	$2.78 \cdot 10^3$	$48.92 \cdot 10^3$
7	1025	"	641	384	"	49.49
6	1475	"	644	831	"	50.16
5	1875	"	628	1247	"	49.81
4	2225	43.32	704	1521	"	47.47
3	2525	"	594	1931	"	42.26
2	2800	"	418	2382	"	33.25
1	3050	85.26	315	2735	"	19.44

$$Q_{wn} = Q_n - \frac{2\Sigma Dc_n}{h_n} \cdot 3\psi_n$$

$$M_{Rn} = 3\Sigma K_{ben} \cdot \phi_n$$

Table 3.7

n	K_{wn}	ϕ_n	ϕ_{n-1}	$6EK_oR_n$	$M_{n,n-1}$ kN·cm	$M_{n-1,n}$ kN·cm
8	$1.86 \cdot 10^3$	5.866	5.934	17.695	$-0.0539 \cdot 10^6$	$0.0725 \cdot 10^6$
7	"	5.934	6.014	17.951	-0.1283	+0.0205
6	"	6.014	5.973	18.043	-0.0781	-0.1544
5	"	5.973	5.692	17.591	0.0874	-0.4352
4	"	5.692	5.067	16.250	0.3740	-0.7886
3	"	5.067	3.987	13.722	0.7421	-1.2667
2	"	3.987	2.331	9.651	1.2164	-1.8637
1	"	2.331	0	3.687	1.8136	-2.5221

$$M_{n,n-1} = K_{wn}(2\phi_n + \phi_{n-1} - 6EK_oR_n); \quad M_{n-1,n} = K_{wn}(2\phi_{n-1} + \phi_n - 6EK_oR_n)$$

Table 3.8 Shear in frame columns

n	ΣQ_{Fn} kN	ΣD_{cn}	R_4 Q_1 kN	R_4 Q_2	R_5 Q_1	R_5 Q_2	R_5 Q_3	R_6 Q_1	R_6 Q_2
8	632	5.16	26.95	39.20	30.60	62.50	53.90	26.95	52.70
7	641	"	27.30	39.75	31.05	63.35	54,65	27.30	53.40
6	831	"	35.40	51.50	40.30	82.10	70.90	35.40	69.20
5	628	"	28.95	38.95	30.40	62.10	53.55	26.80	52.50
4	704	6.26	24.75	48.35	36.00	67.50	56.20	24.75	73.10
3	594	"	20.90	40.80	30.35	56.95	47.45	20.90	61.70
2	418	"	14.70	28.70	21.35	40.10	33.40	14.70	43.40
1	315	12.23	9.00	19.80	17.80	35.55	33.75	9.00	23.95

Table 3.9 Inflexion point height

n	$R_{4,1}$	$R_{4,2}$	$R_{5,1}$	$R_{5,2}$	$R_{5,3}$	$R_{6,1}$	$R_{6,2}$
8	0.32	0.38	0.35	0.30	0.27	0.32	0.43
7	0.40	0.43	0.40	0.40	0.37	0.40	0.48
6	0.42	0.45	0.45	0.40	0.40	0.42	0.48
5	0.45	0.48	0.45	0.45	0.45	0.45	0.50
4	0.45	0.45	0.45	0.45	0.40	0.45	0.46
3	0.47	0.45	0.45	0.45	0.45	0.47	0.50
2	0.50	0.50	0.55	0.55	0.55	0.50	0.50
1	0.65	0.70	0.80	0.80	0.85	0.65	0.64

It is worth to note that moments according to Eq. 3.10 are for beams in the centroid line of the wall and for beams at both sides It is necessary to calculate beam moments at the face of the wall.

Moment diagrams for frames R5 and R6, according to the described method, due to horizontal seismic forces are shown in Fig. 5c.

The comparison of the results obtained according to the TABS program and Muto's method (Fig. 4 and Fig. 5c) shows that in the case of the latter members in the frame R5 and columns in the frame R6 will be under-designed, but the shear wall in the middle of R6 will be over-designed.

For comparison, another structural analysis was accomplished by neglecting the beams connecting the wall of the frame R6 and the adjacent columns. In this way, a free standing wall is obtained, fixed at base level. The results of this analysis are shown in Fig. 5d as moment diagrams due to horizontal forces, for the frames R5 and R6, respectively.

In this case, wall moments are larger by about 20%, compared to moments in Fig. 4, which would require considerably more reinforcement, while the columns of the frame R6 would be under-reinforced.

The moments in the frame R5 are larger by 15% to 40%, in the lower and upper stories, respectively, compared to those when adequate connection of the wall to adjacent beams is provided, which would however mean an uneconomic design of this frame.

Taking into account the three analyses carried out for this building, it is concluded that the analysis applying TABS programme gives optimum results, both in respect to accuracy and time consumption. The hand calculation takes much effort, and in the case of more complex frame system with shear walls and coupled shear walls, the hand calculation is almost impossible.

4. STIFFNESS VERIFICATION

Verification of adequate stiffness of the building in x-direction is performed considering inelastic behaviour of the structure. The maximum story drift obtained for ground acceleration of 0.23 g is 0.80 cm which is 0.0028 h_i. The required displacement ductility, for the same acceleration level, is from 3.00 to 3.66.

5. CALCULATION OF REINFORCEMENT

The building structure was proportioned for a combination of vertical and seismic loads applying the method of ultimate load carrying capacity, according to the Provisions for concrete and reinforced concrete [2] .

The structural members like columns, walls and beams were designed for the known values of moments and axial forces while the shear force is covered by stirrups in columns and beams and by horizontal reinforcement in walls.

Equilibrium equations for eccentrically loaded columns and walls were derived for the following assumptions:

1. Plane sections before bending remain plane after bending
2. The σ -ε diagram for reinforcement is known
3. Tensile stresses in the concrete are neglected
4. The σ -ε diagram for concrete is known.

The first assumption implies that the longitudinal strain in the concrete and in the steel at the various points across a section is proportional to the distance from the neutral axis. A large number of tests on RC members have indicated that this assumption is very nearly correct at all stages of loading up to flexural failure.

The second assumption means that the stress-strain properties of the steel are well defined. Normally, a bilinear stress-strain curve is assumed.

The third assumption is very nearly correct. Any tensile stress that exists in the concrete just below the neutral axis is small and has a small lever arm.

The fourth assumption is necessary to assess the true behaviour of the section.

A rectangular section with reinforcement at both ends and uniformly distributed reinforcement in between has been considered (Fig. 6).

The characteristics (yield stresses) of the end reinforcement or the reinforcement between them, can be either similar or different.

According to Fig. 6, the equilibrium equation obtained as a sum of the internal forces is:

$$N_u = C_c + C_s + (C_v' - C_v'') - T_s - (T_v' - T_v'') \qquad (5.1)$$

where $C_c = \alpha_1 . b . x . f_c''$, α_1 - is a coefficient of mean stress of the working σ -ε diagram for concrete.

$$C_s = A'_s \cdot f_y$$

$$C'_v = (x - c') \frac{A_v}{100} \cdot f_{a,v}$$

$$C''_v = \frac{1}{2}\, y\, \frac{A_v}{100} \cdot f_{y,v}\,, \quad \text{where:} \quad y = x \cdot \varepsilon_{y,v}/\varepsilon_{cu}$$

$$T_s = A_s \cdot f_y$$

$$T'_v = (h - x - c)\, \frac{A_v}{100} \cdot f_{y,v}, \quad T''_v = C''_v$$

After dividing all terms of the equation (5.1) by bhf''_c and introducing the following notation,

$$\rho = \frac{A_s}{bh}\,, \quad \rho' = \frac{A'_s}{bh}\,, \quad \rho_v = \frac{A_v}{100b}\,, \quad \xi = \frac{x}{h}$$

$$K = \frac{\rho \cdot f_y}{f''_c}\,, \quad K_v = \frac{\rho_v \cdot f_{y,v}}{f''_c}\,, \quad n = \frac{N_u}{bhf''_c}\,,$$ the following is obtained:

$$n = \alpha_1 \cdot \xi + K(\rho'/\rho) + K_v(\xi - c'/h) - K - K_v(1 - \xi - c/h) \qquad (5.2)$$

Solving the above equation for ξ, an expression for the height of the neutral axis is obtained:

$$\xi = \frac{K(1-\rho'/\rho) + K_v(1 - c/h + c'/h) + n}{\alpha_1 + 2\,K_v} \qquad (5.3)$$

For eccentrically loaded columns and walls as shown in Fig. 6, the equilibrium of moments about the neutral axis will give:

$$N \cdot e_x = M_c + C_s(x-c') + C_v' \, (x-c') \, \frac{1}{2} - C_v'' \cdot \frac{y}{3} +$$

$$+ T_s(h-x-c) + T_v' \, (h-x-c) \, \frac{1}{2} - T_v'' \cdot \frac{y}{3} \qquad (5.4)$$

where:

$M_c = \alpha_2 . b . x^2 . f_c''$, α_2 is a static moment coefficient of the working diagram area of the concrete ($\alpha_2 = \alpha_1 . x_c / x$).

After the internal force expression will be introduced, and dividing all terms of equation (5.4) by $bh^2 f_c''$, the following is obtained:

$$M_u = bh^2 f_c'' \, \{\alpha_2 . \xi^2 + K \left[1-\xi(1-\rho'/\rho) - \frac{c}{h} - \frac{c'}{h} \cdot \frac{\rho}{\rho}\right] + \eta(0.5-\xi) +$$

$$+ K_v \left[0.5(\xi - c'/h)^2 + 0.5(1-\xi-c/h)^2 - \left(\frac{\varepsilon_{y,v}}{\varepsilon_{cu}}\right)^2 \frac{\xi^2}{3}\right]\} \qquad (5.5)$$

In equation (5.5) a moment correction has been introduced about the center of the section: $N.e_o = N.e_x + N(h/2-x)$ where $N.e_o = M_u$.

The above equation for ultimate moment applies for bending with or without axial force. It also applies to axial compressive or tensile force in the section, however, a sign change should be made for tension in the equations 5.3 and 5.5, in front of the term η.

The equations for neutral axis and the ultimate moment have been derived for the case when yielding takes place in the compressed reinforcement.

Coefficients α_1 and α_2 depend upon the adopted $\sigma - \varepsilon$ working diagram for concrete. For the diagram proposed by the European Commission for Concrete (CEB), according to Fig. 7, the coefficients α_1 and α_2 have the following values:

$$\alpha_1 = \frac{\varepsilon_c}{\varepsilon_0} - \frac{\varepsilon_c^2}{3\varepsilon_0^2}, \quad \text{for the parabolic part,} \quad (5.6)$$

$$\alpha_1 = 1 - \frac{\varepsilon_0}{3\varepsilon_c}, \quad \text{for horizontal line} \quad (5.7)$$

$$\alpha_2 = \frac{2}{3} \cdot \frac{\varepsilon_c}{\varepsilon_0} - \frac{\varepsilon_c^2}{4\varepsilon_0^2}, \quad \text{for the parabolic part} \quad (5.8)$$

$$\alpha_2 = 0.5 - \frac{1}{12} \cdot \frac{\varepsilon_0^2}{\varepsilon_c^2}, \quad \text{for the horizontal line} \quad (5.9)$$

The values of coefficients α_1 and α_2 for ultimate state and ultimate strains in concrete $\varepsilon_{cu} = 3.5‰$ and for $\varepsilon_o = 2‰$, are

$$\alpha_1 = 0.81 \qquad \text{and} \qquad \alpha_2 = 0.473$$

For proportioning of structural members equations 5.3 and 5.5 have been used, taking into account the symmetric reinforcement $A_s = A_s'$ and the uniform thickness of the concrete cover $c = c'$. Also, the safety coefficient γ_u, has been introduced complying with the provisions [1] and [2].

The code [2] for earthquake resistant design of structures, prescribes $\gamma_u = 1.30$ for seismic loads, for ultimate state design purposes.

Multiplying the design moment by the safety coefficient and replacing $K=\rho.f_y/f_c''$,

$$M_d \cdot \gamma_u = bh^2 f_c'' \{\alpha_2 . \xi^2 + \rho \cdot f_y/f_c'' \ (1-2c/h) + n(0.5-\xi) +$$

$$+ K_v \left[0.5(\xi-c/h)^2 + 0.5(1-\xi-c/h)^2 - \left(\frac{\varepsilon_{y,v}}{\varepsilon_{cu}} \right)^2 \frac{\xi^2}{3} \right] \}$$

Solving the above equation for ρ, the following is obtained:

$$A_s = A_s' = \frac{bh^2 f_c''}{f_y(h-2c)} \left\{ \frac{M_d \cdot \gamma_u}{bh^2 f_c''} - \alpha_2 \cdot \xi^2 - n(0.5 - \xi) - \right.$$

$$\left. - K_v \left[0.5(\xi-c/h)^2 + 0.5(1-\xi-c/h)^2 - \left(\frac{\varepsilon_{y,v}}{\varepsilon_{cu}} \right)^2 \frac{\xi^2}{3} \right] \right\} \qquad (5.10)$$

For application of the above equation the percentage of vertical reinforcement in webs ρ_v, should be precedingly defined. For reinforced concrete walls, welded net with equal vertical and horizontal reinforcement is most frequently used in webs. The quantity of horizontal reinforcement has been defined from the condition that the total seismic shear force in the possible nonlinear zones is taken by the horizontal reinforcement. This provision means that equivalent truss should be introduced, the same as it is for beams, as shown by Park and Paulay [6].

Proportioning of horizontal reinforcement has been made applying the following consideration, (Fig. 8):

$$T_s = Q_s \text{ and } T_s = A_h \cdot f_s$$

$$\frac{T_s}{s} = \frac{Q_s}{Z\cot} \quad ,$$

$$\frac{A_h \cdot f_s}{s} = \frac{Q_s}{Z\cot\alpha}, \text{ that is}$$

$$s = Z \cdot \cot\alpha \qquad A_h = \frac{Q_s \cdot s}{f_s \cdot Z\cot\alpha} \tag{5.11}$$

The angle α corresponds to the slope of the diagonal cracks of the wall. The experimental data show that this angle is about 45%, so that we can adopt:

$$A_h = \frac{Q_s \cdot s}{f_s(h-2c)} \tag{5.12}$$

The minimum percentage of horizontal reinforcement according to Provisions [1] is 0.2% of the vertical cross sectional area of the wall, the same as for the vertical reinforcement of the wall.

Proportioning of beams was accomplished in the sections adjacent to columns (at the supports) for combined vertical loads, and seismic loads acting in both directions (Fig. 9).

After the proportioning, the condition that the compressive reinforcement equals to or is greater than half the tensile reinforcement, $A_s' \geq 0.5A_s$, has been controled.

The adopted reinforcement for the elements of frame R_6 has been shown in Fig. 10.

It should be mentioned that proportioning of elements on the basis of the above explained methods was carried out applying the computer programme which avails static values directly from the TABS programme.

In order to meet the basic aseismic design principle, that is, plastic hinges developing in beams and not in columns of the frames, the ultimate load carrying capacity of both beams and columns in one joint should be computed to satisfy the following condition:

$$1.2 \cdot \Sigma M_{u,b} \leq \Sigma M_{u,c} \tag{5.13}$$

Computation of ultimate load carrying capacity of frame elements was done applying equation 5.5, and if the condition 5.13 has not been satisfied, the column reinforcement is gradually increased until the above condition is satisfied.

The plastic hinge mechanism is evaluated by the DRAIN-2D [7] computer programme. For this purpose, frame R_6 was analyzed as plane structure subjected at the base to El Centro 1940 earthquake, N-S component. A four-second earthquake duration having highest accelerations has been selected, while the sequence and position of plastic hinges has been given in Fig. 11.

As it is observed, plastic hinges formed only in frame beams, while they did not form in columns and walls.

The nonlinear analysis verified the favourable behaviour of R_6 frame under earthquake effects.

Based on the calculated areas of required reinforcement for the elements of frame R_6, the beam reinforcement was selected as shown in Fig. 12. Also, column reinforcement was selected as shown in Fig. 13. It should be emphasised that reinforcement selection, especially for beam elements should be based on the calculated reinforcement area, so that the adopted plastic hinge formation mechanism in the structure is not disturbed.

Frame R6 : Beams

Table 5.1. Exterior spans $Z.f_y = 32.240 = 7680$

n	b/h cm	M_ℓ kNm	M'_ℓ	$M_{\ell/2}$	A_s	A'_s	$A_{s,\ell/2}$	A_{smin}	$A'_{s,min}$	$A_{s,\ell/2}$	A'_s
8	40/40	56	20	16.9	9.48	3.39	2.86	4.0	4.74	4.0	4.74
6	"	85	41	17.5	14.39	6.94	2.96	4.0	7.19	4.0	7.19
4	"	89.6	54.4	17.8	15.17	9.21	3.01	4.0	7.58	4.0	9.21
2	"	78.6	47.3	17.6	13.32	8.01	2.98	4.0	6.66	4.0	8.01

Table 5.2. Interior spans - left supports

n	b/h	M_ℓ	M'_ℓ	$M_{\ell/2}$	A_s	A'_s	$A_{s,\ell/2}$	A_{smin}	$A'_{s,min}$	$A_{s\ell/2}$	A'_s
8	40/40	97.5	78.5	18.4	16.50	13.29	3.11	4.0	8.25	4.0	13.29
6	"	134.2	101.8	22.6	22.72	17.23	3.83	4.0	11.36	4.0	17.23
4	"	159.4	116.6	23.3	26.98	19.74	3.94	4.0	13.49	4.0	19.74
2	"	136.6	85.4	23.3	23.12	14.46	3.94	4.0	11.56	4.0	14.46

Table 5.3. Interior spans - right supports

n	b/h	M_r	M'_r	A_s	A'_s	$A'_{s,min}$	/	/	/	/	A'_s
8	40/40	167.4	74.6	28.34	12.63	14.17					14.17
6	"	193.0	91.0	32.67	15.40	16.33					16.33
4	"	198.4	109.6	33.58	18.55	16.79					18.55
2	"	158.2	77.8	26.78	13.17	13.39					13.39

Frame R6: Columns (from capacity of beams)

Table 5.4 Exterior columns:

$M_{d,c} = 1.2\ M_{n,b}$ - top column

$M_{d,c} = 0.6\ M_{n,b}$ - other columns

n	b/h	$M_{n,b}$ kN·m	$M_{d,c}$	N_d	η	ξ	$\frac{M_{d,c}}{bh^2 \cdot f_c''}$	η(0.5-ξ)	$A_s=A_s'$	$A_{s,min}$
8	40/40	72.81	87.37	65	0.019	0.028	0.0650	0.0090	9.80	4.80
6	"	110.52	66.31	198	0.059	0.089	0.0493	0.0242	4.40	"
4	"	116.51	69.90	307	0.091	0.137	0.0520	0.0330	3.33	"
2	"	102.30	61.38	394	0.117	0.176	0.0457	0.0379	1.36	"

Table 5.5 Interior columns:

$M_{d,c} = 1.2(M_{n,b} + M'_{n,b})$ - top column

$M_{d,c} = 0.6(M_{n,b} + M'_{n,b})$ - other columns

n	b/h	$M_{n,b}$	$M'_{n,b}$	$M_{d,c}$	N_d	η	$\frac{M_{d,c}}{bh^2 f_c''}$	η(0.5-ξ)	$A_s=A_s'$	$A_{s,min}$
8	40/40	126.72	102.07	274.54	84	0.025	0.2043	0.0116	33.72	4.80
6	"	174.49	132.33	184.09	334	0.099	0.1369	0.0347	17.89	"
4	50/50	207.21	151.60	215.29	589	0.112	0.0820	0.0372	11.66	7.50
2	"	177.56	111.05	173.17	865	0.165	0.0660	0.0416	6.35	"

Table 5.6 Interior column: $f_y \cdot Z = 240 \times 42 = 10080$

$50 \times 50^2 \times 21 = 2625 \times 10^3$ $bh^2 f''_y \cdot Z = 260.4$

n	b/h	M_d	N_d	η	ξ	$\frac{bh^2 f_c''}{f_y \cdot Z}$	$\frac{M_d \cdot \gamma_u}{bh^2 f_c''}$	$\alpha_2 \cdot \xi^2$	η(0.5-ξ)	$A_s=A_s'$	$A_{s,min}$
8	40/40	91	84	0.025	0.038	175.0	0.0880	0.0006	0.0116	13.27	4.80
7	"	73	209	0.062	0.093	"	0.0706	0.0036	0.0252	7.31	"
6	"	81	334	0.099	0.149	"	0.0783	0.0093	0.0347	6.00	"
5	"	77	460	0.137	0.206	"	0.0745	0.0177	0.0403	2.89	"
4	50/50	120	589	0.112	0.168	260.4	0.0594	0.0118	0.0372	2.71	7.50
3	"	97	722	0.138	0.207	"	0.0480	0.0179	0.0404	-	"
2	"	78	865	0.165	0.248	"	0.0386	0.0256	0.0416	-	"
1	"	71	1044	0.199	0.299	"	0.0351	0.0373	0.0400	-	"

Frame R6: Shear Walls

$$A_h = \frac{Q_d \cdot 100}{f_{y,w}(h-2c)} \; ; \qquad K_v = \frac{\rho_v \cdot f_{y,w}}{f''_c} \qquad \eta = \frac{N_d}{bhf''_c} \; ;$$

$f_{y,u}(h-2c) = 500(580-50) = 26500$; $f''_c = 21$ MPa

Table 5.7

n	b/h cm	M_d kN·m	N_d kN	Q_d kN	$A_h=A_v$ cm²/m'	K_v	η	$K_v+\eta$	α_1+2K_v	ξ	$A_{h,min}$
1	2	3	4	5	6	7	8	9	10	9/10	
8	20/580	1145	428	-211	0.80	0.0476	0.0176	0.0352	0.7619	0.046	4.0
7	"	1636	784	315	1.19	"	0.0322	0.0498	"	0.065	"
6	"	1215	1137	676	2.55	"	0.0467	0.0643	"	0.084	"
5	"	3030	1488	1065	4.02	"	0.0611	0.0787	"	0.103	"
4	"	5582	1834	1115	4.21	0.050	0.0753	0.1253	0.7667	0.163	"
3	"	9400	2176	1545	5.83	0.0694	0.0893	0.1587	0.8055	0.197	"
2	"	14595	2520	1981	7.47	0.0890	0.1034	0.1924	0.8447	0.228	"
1	"	20930	2860	2310	8.72	0.1038	0.1174	0.2212	0.8743	0.253	"

$bh^2f''_c = 20 \cdot 580^2 \cdot 21 = 141288 \cdot 10^3$, $\alpha_1 = 2/3$, $\alpha_2 = 5/12$, $c/h = 0.043$.

$$\left(\frac{\varepsilon_{y,v}}{\varepsilon_{cu}} \right)^2 = 0.1066$$

Table 5.8

n	$\frac{bh^2f''_c}{f_y \cdot Z}$	$\frac{M_d \cdot \gamma_n}{bh^2f''_c}$	$\alpha_2 \cdot \xi^2$	$\eta(\frac{1}{2}-\xi)$	$\frac{1}{2}(\xi-\frac{c}{h})^2$	$0.5(1-\xi-\frac{c}{h})^2$	$0.107\frac{\xi^2}{3}$	$K_v \cdot \Delta$	$A_s=A'_s$	$A_{s,min}$
11	12	13	14	15	16	17	18	7·(16+ +17+18)	20	
8	1111,0	0.0105	0.0009	0.0080						17.40
7	"	0.0151	0.0018	0.0140						"
6	"	0.0112	0.0029	0.0194						"
5	"	0.0279	0.0044	0.0242						"
4	"	0.0514	0.0111	0.0254	0.0072	0.315	0.0009	0.0161	-	"
3	"	0.0865	0.0162	0.0270	0.0119	0.289	0.0014	0.0208	25.00	
2	"	0.1343	0.0217	0.0281	0.0171	0.266	0.0018	0.0250	66.10	
1	"	0.1925	0.0267	0.0290	0.0221	0.248	0.0023	0.0278	121.10	

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NOTATIONS

A_s	Cross-sectional area of tension steel.
A_s'	Cross-sectional area of compression steel.
A_h	Cross-sectional area of horizontal steel in walls.
A_v	Cross-sectional area of vertical steel in walls.
b	Width of the cross-section.
e_o	Eccentricity to the centroid of the section.
e_x	Eccentricity to the neutral axis.
f_c''	Maximum compressive strength in the concrete.
f_{ck}	Characteristic compressive strength of concrete.
f_y	Characteristic yield strength of steel.
$f_{y,w}$	Characteristic yield strength of web steel.
f_s	Design stress of transverse steel in walls.
K_b	Stiffness ratio of a beam.
K_{be}	Effective stiffness ratio of a beam.
K_c	Stiffness ratio of a column.
M_s	Bending moment due to seismic forces.
M_u	Ultimate bending moment capacity.
N_s	Axial force due to seismic forces.
N_v	Axial force due to gravity load.
Q_s	Shear force in elements due to seismic forces.
Q_y	Shear force at yielding of main reinforcement.
Δ_e	Elastic interstory drift.
Δ_{max}	Maximum interstory drift in dynamic analysis.
Δ_y	Interstory drift at yielding of main reinforcement.
η	Ratio between axial load and axial load capacity.
$\varepsilon_{y,v}$	Yield strain in vertical web reinforcement.
ρ	Percentage of tension reinforcement.
ρ'	Percentage of compression reinforcement.

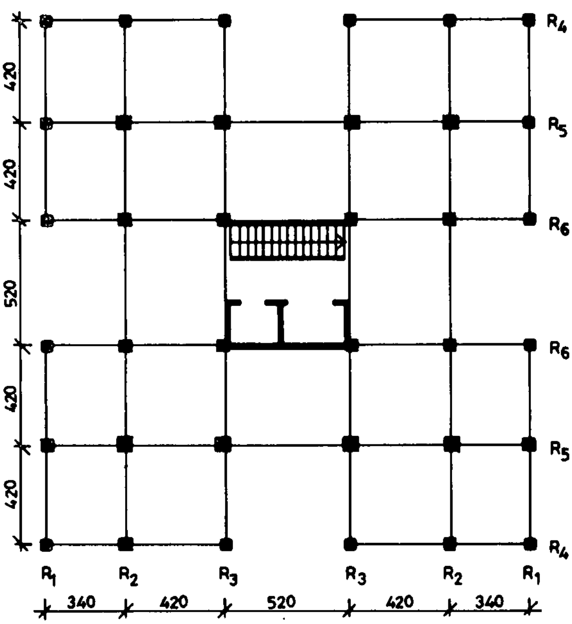


Fig. 1. Typical floor plan

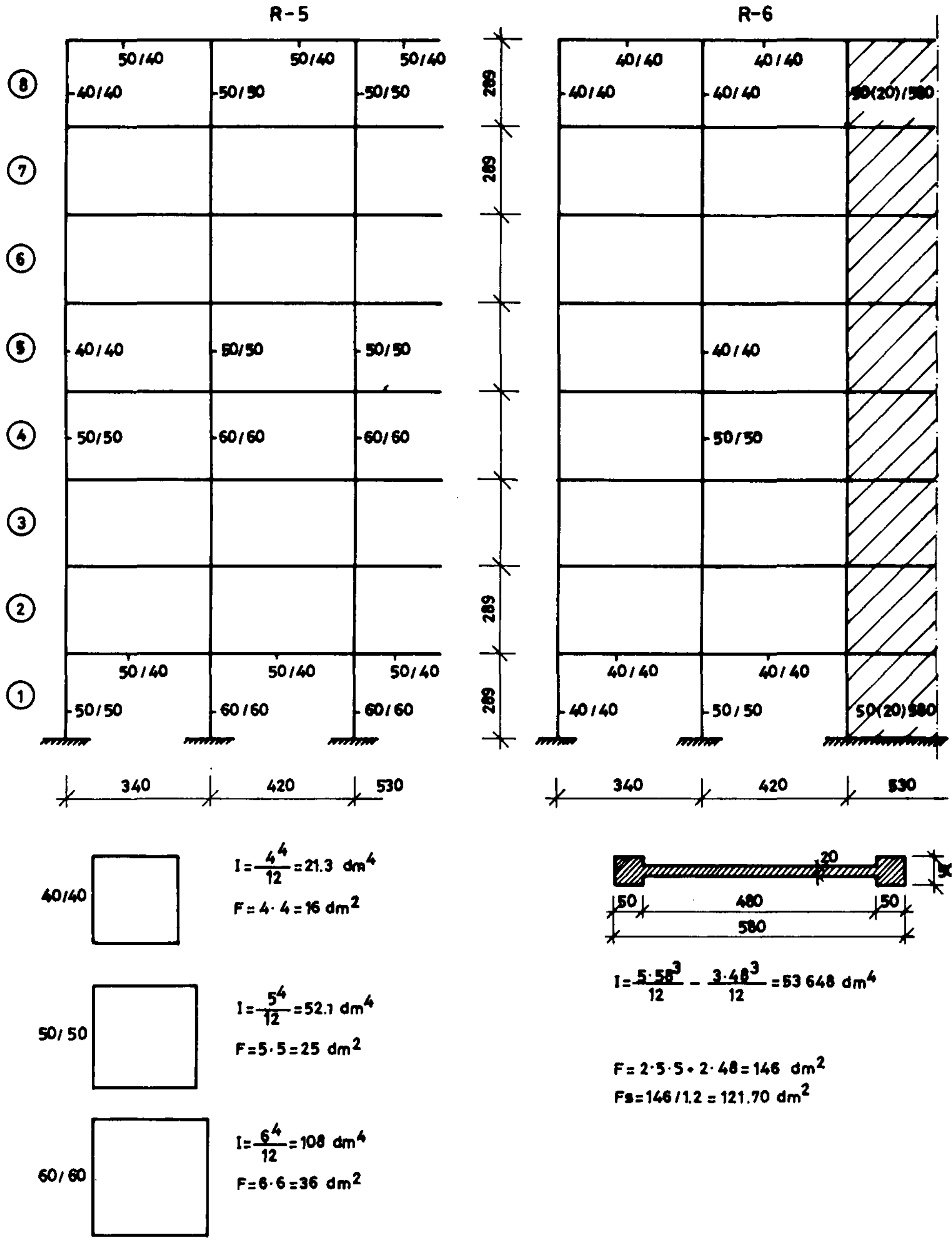


Fig. 2. Structural set-up of frames

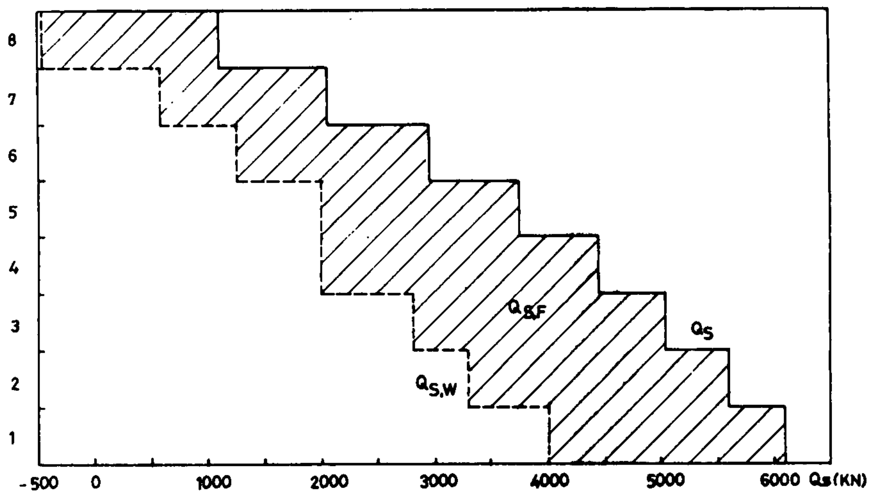


Fig. 3. Storey shear distribution

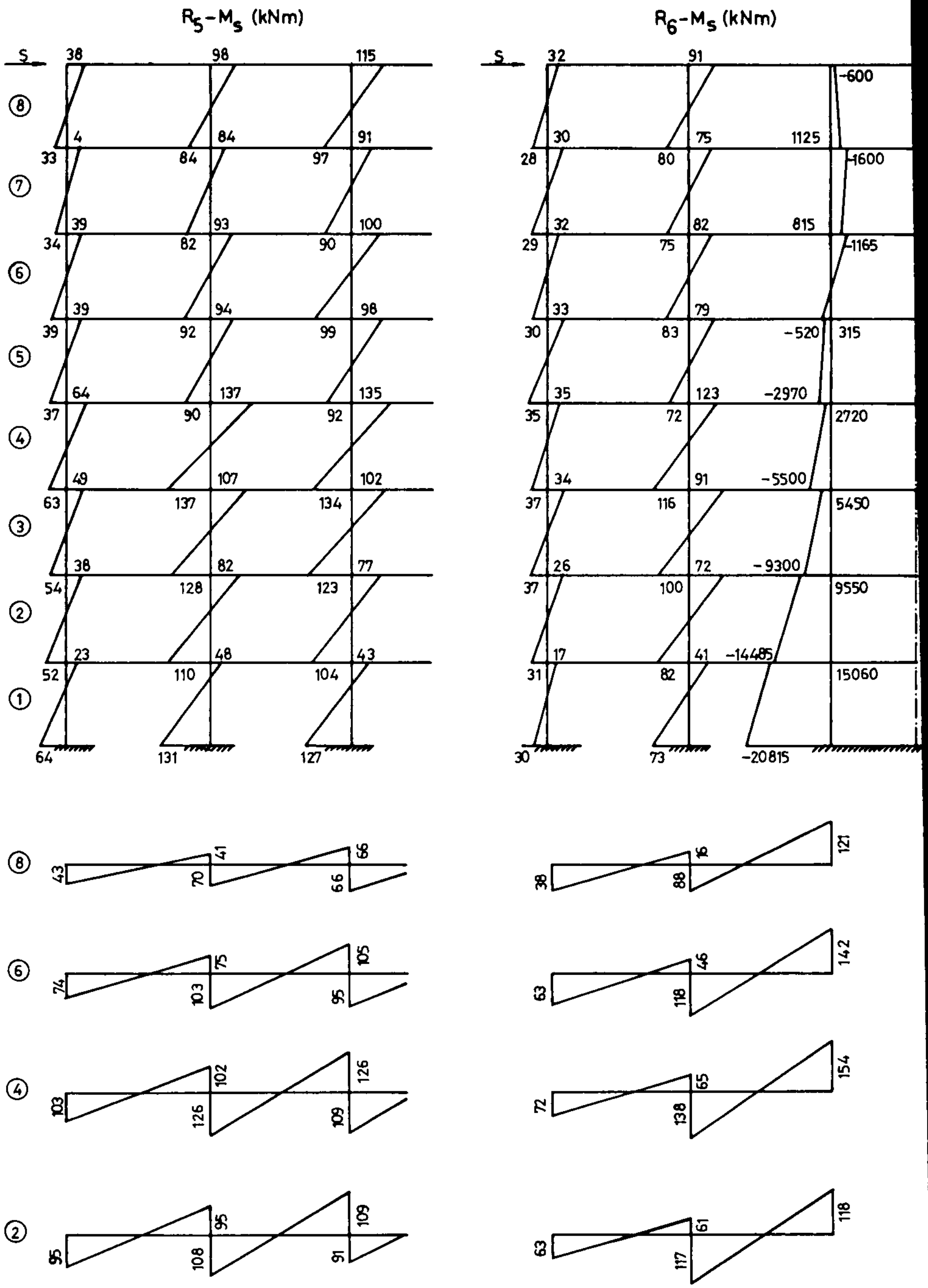


Fig. 4. Moment diagrams obtained by TABS

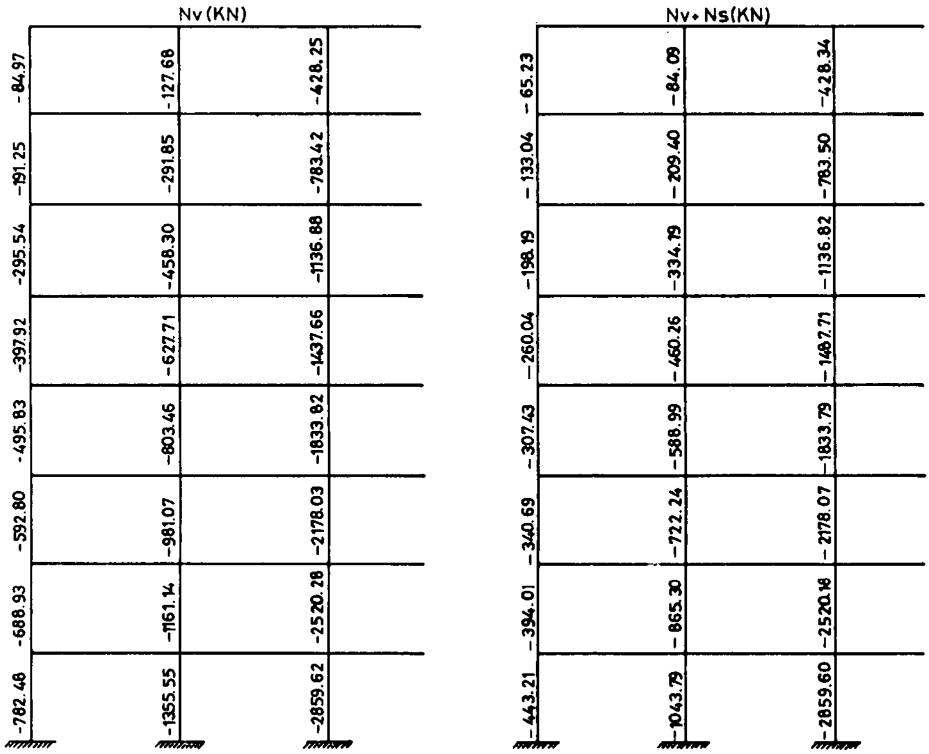


Fig. 5. Axial forces obtained by TABS

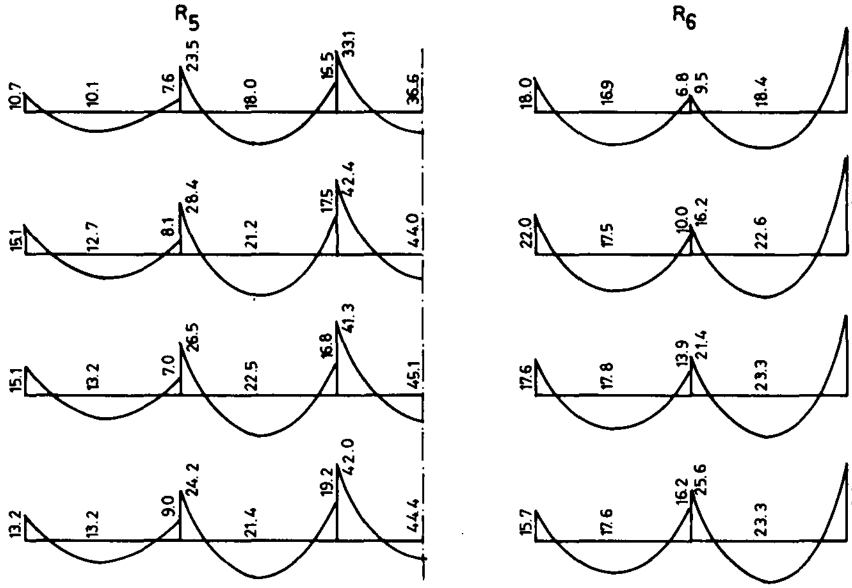


Fig. 5a. Moment diagrams for beams due to vertical loads

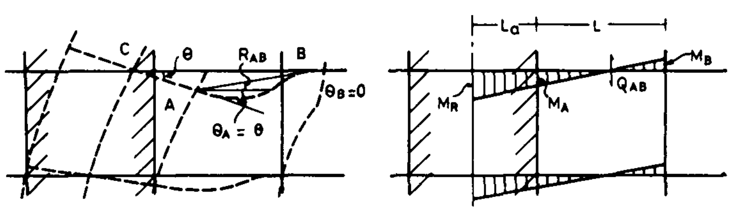


Fig. 5b. Deformation of shear wall and frame

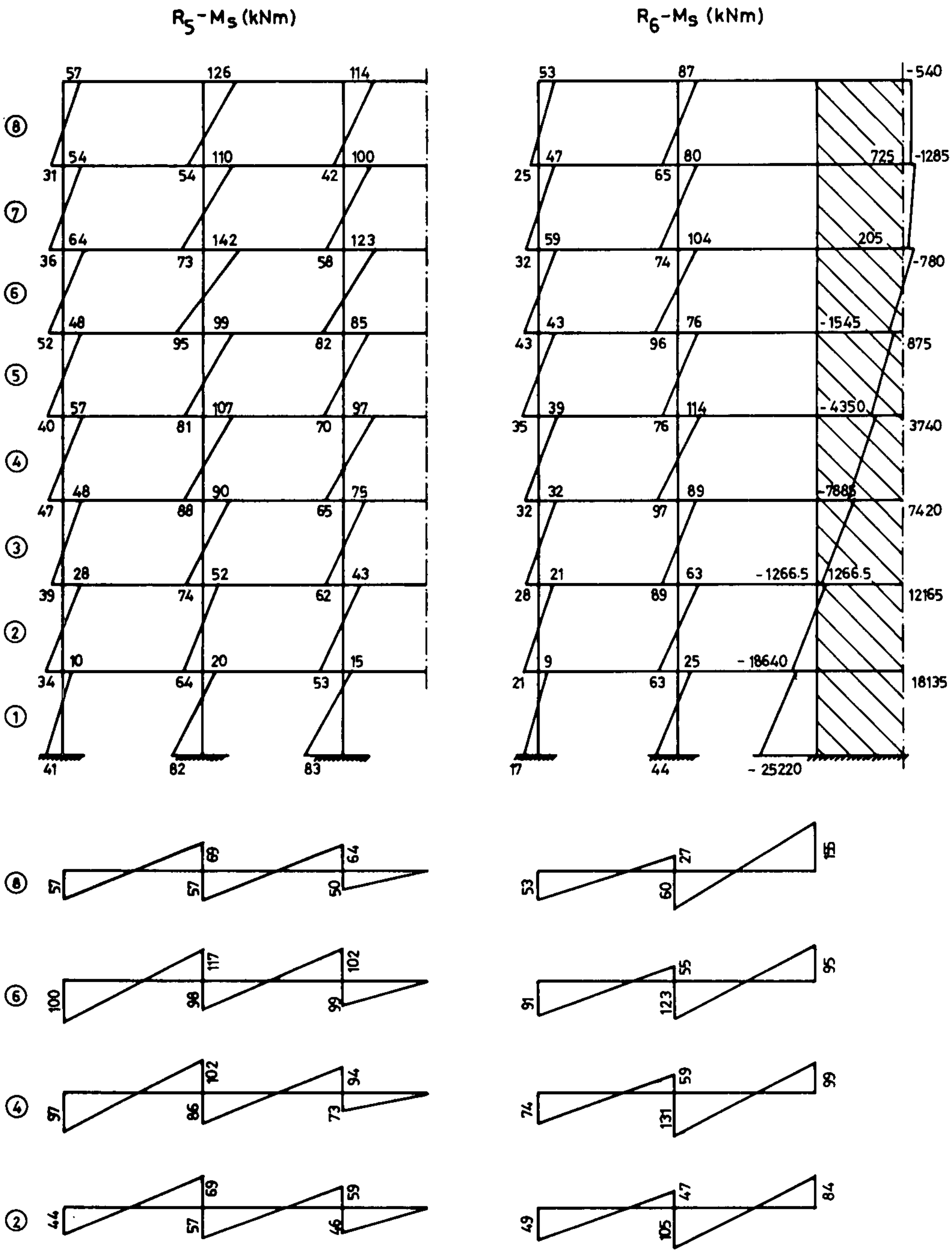


Fig. 5c. Moment diagrams by Muto's method

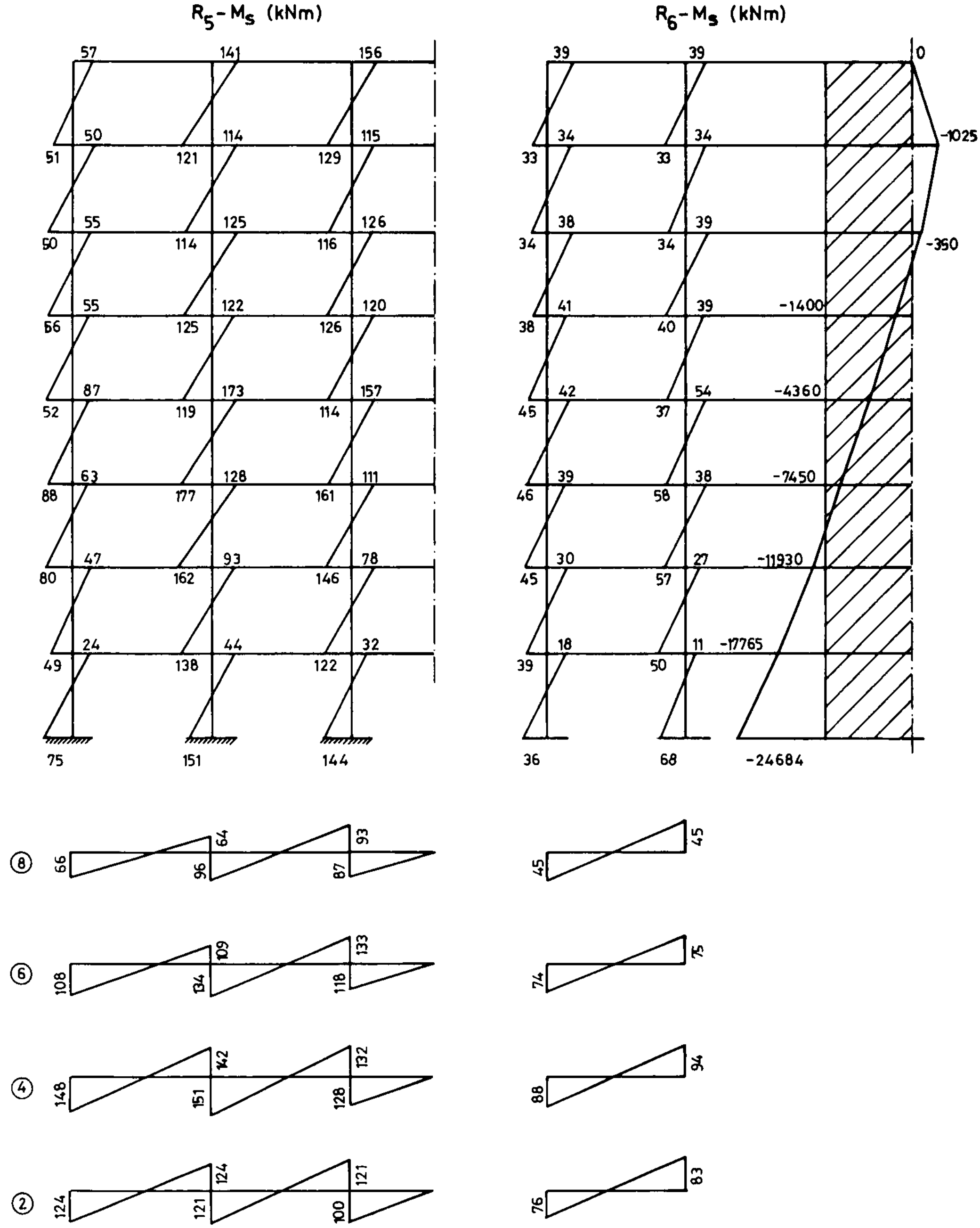


Fig. 5d. Moment diagrams with a cantilever shear wall

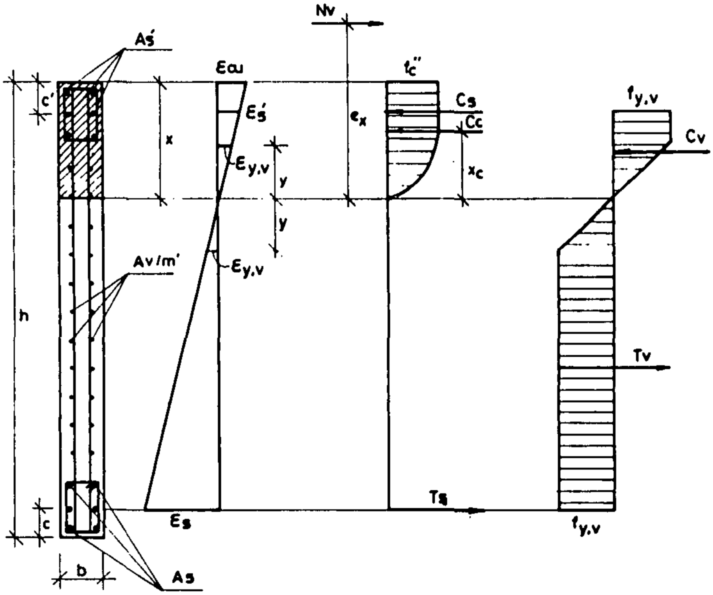


Fig. 6. Doubly reinforced concrete section at ultimate strength

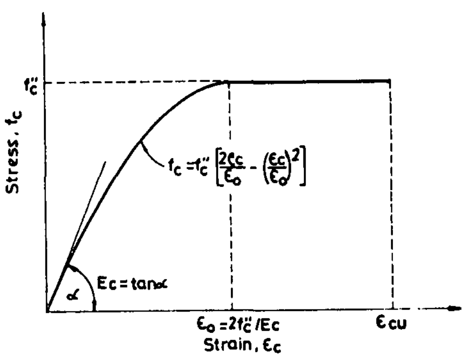


Fig. 7. Stress-strain curve for concrete in compression

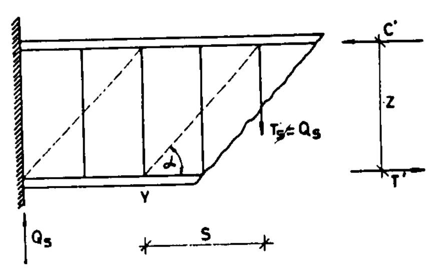


Fig. 8. Equivalent truss

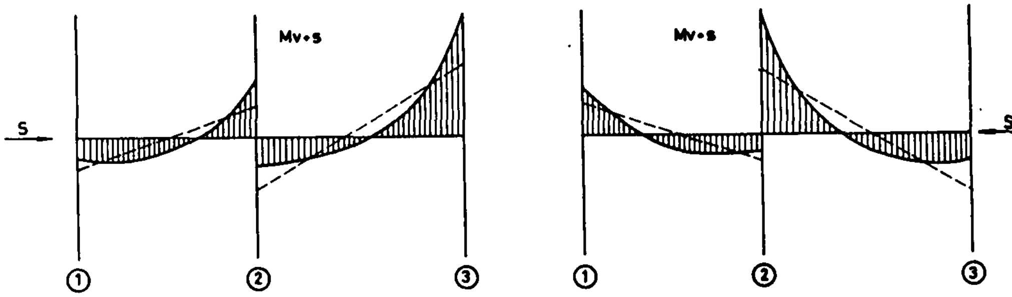


Fig. 9. Combined moment diagram

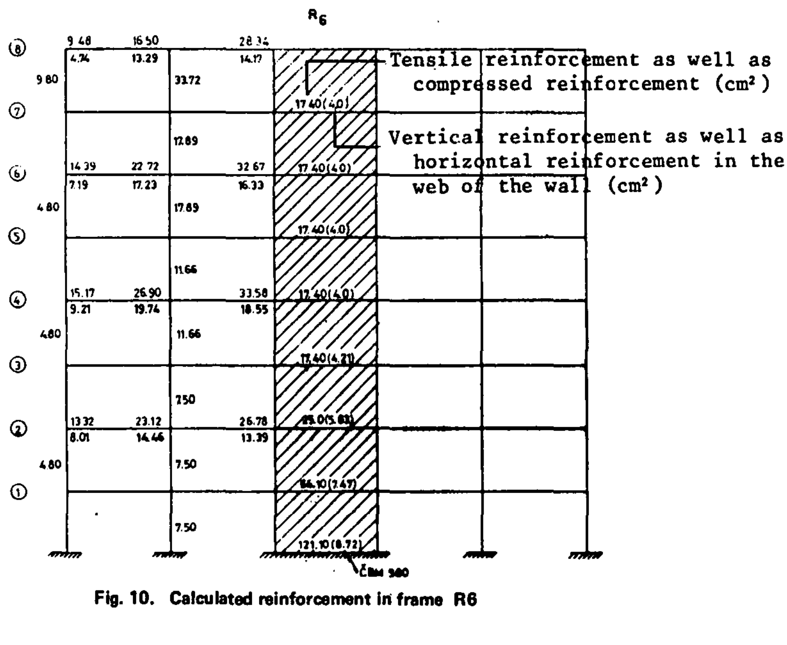


Fig. 10. Calculated reinforcement in frame R6

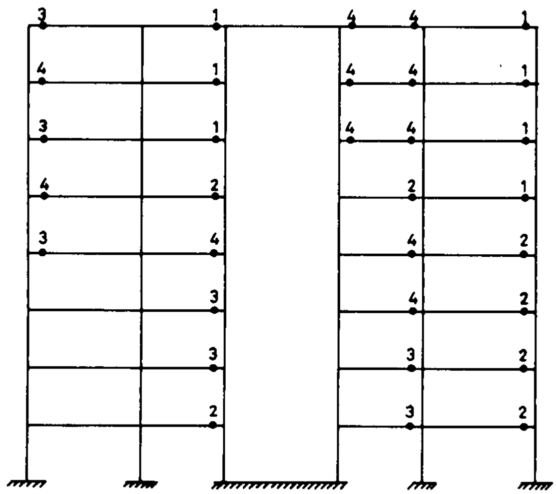


Fig. 11. PH due to El Centro 1940, N–S

6. DETAILING

Frame R6 (Beams)

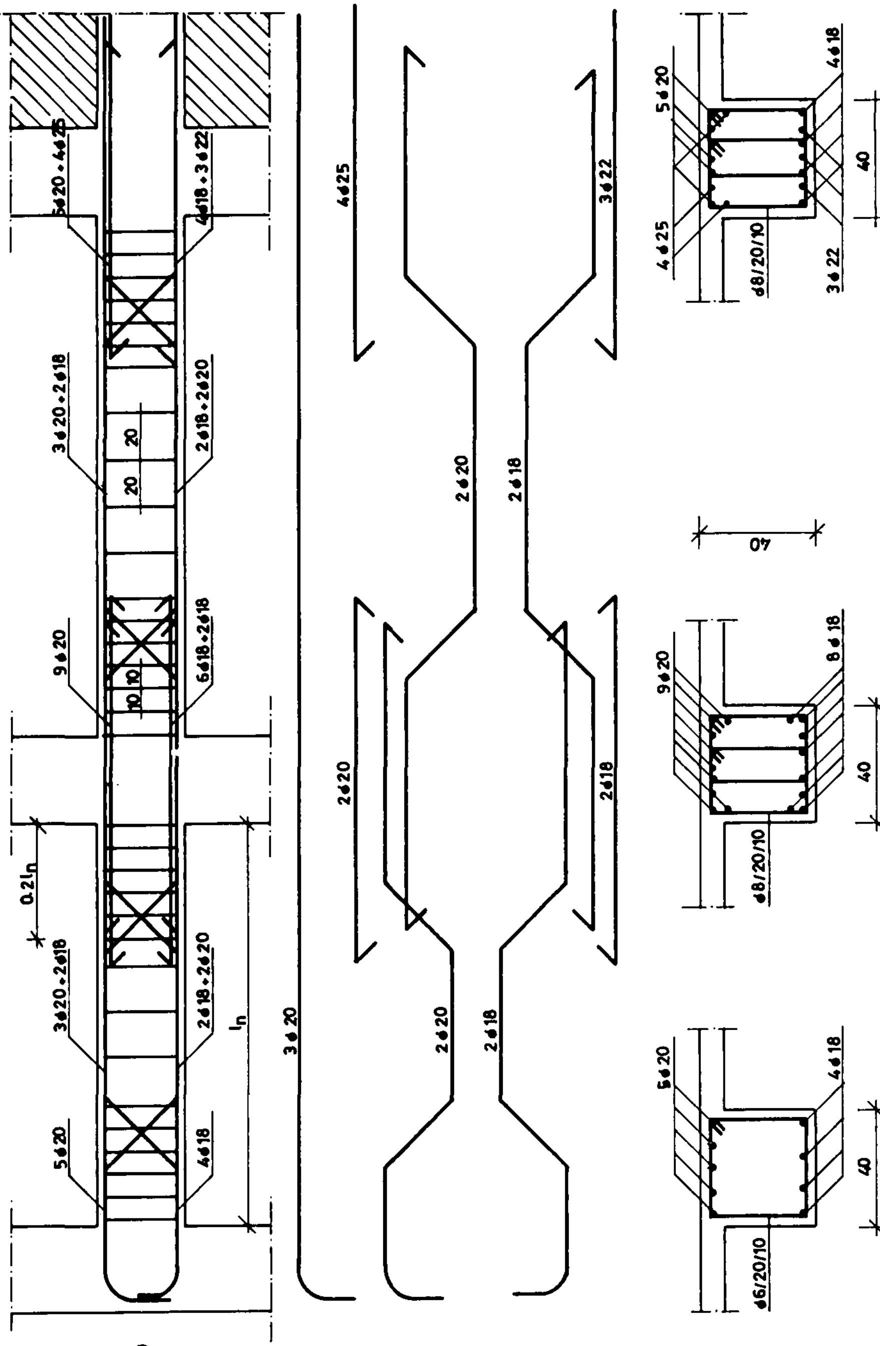


Fig. 12. Reinforcement in beams

Frame R_6 (columns)

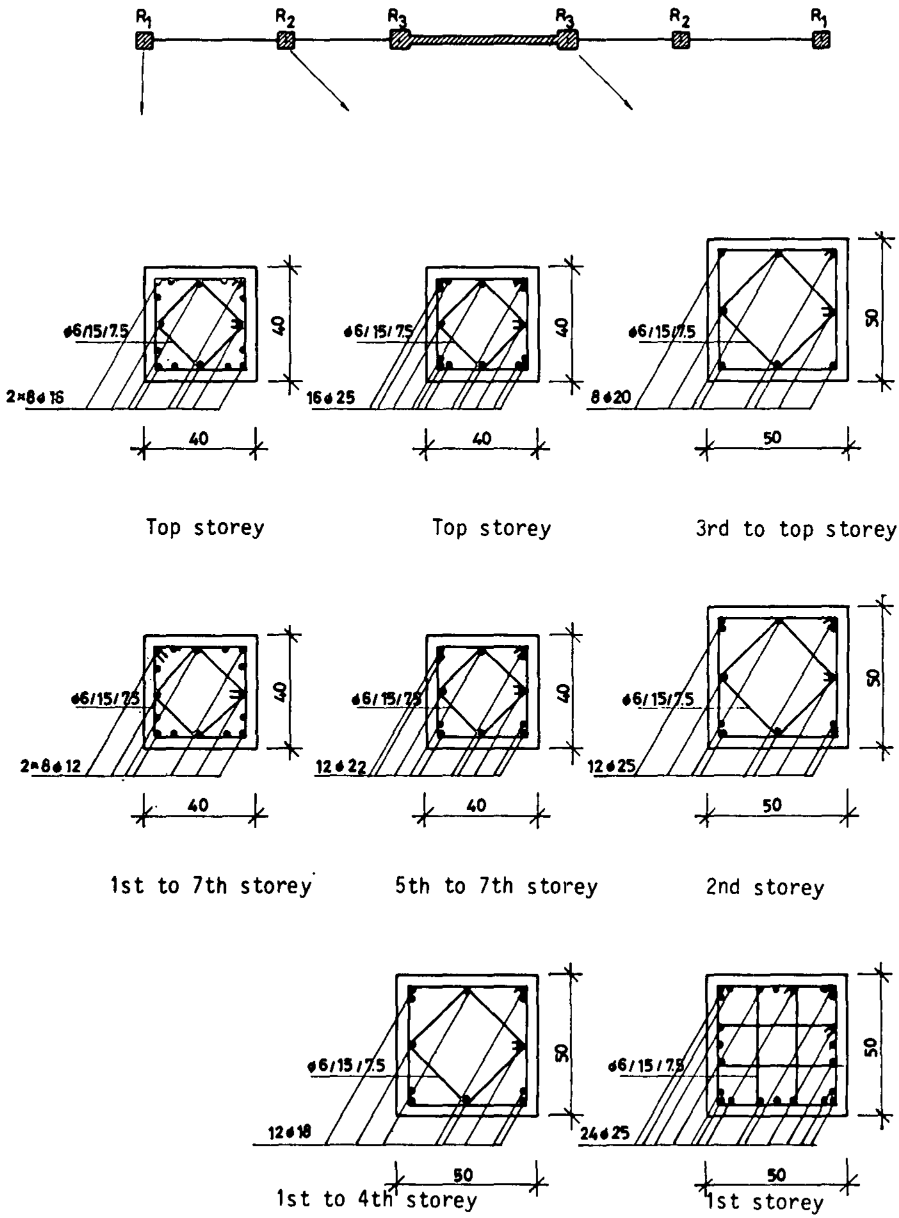


Fig. 13. Reinforcement in columns

STATIC AND DYNAMIC, TIME-HISTORY ANALYSES
OF THE DESIGN EXAMPLES

1. FOREWORD

For comparison of the three design examples, elaborated by the representatives of Bulgaria, Dr. Dimitrov, Rumania, Prof. Dimitrescu and Yugoslavia, Prof. B. Simeonov, a special analysis has been carried out, which contains the following items :

- Applying a three-dimensional static analysis for equivalent static seismic forces and their distribution according to the values determined in each design individually, the static values comparable to those determined in each design example have been obtained.
- Using the new static values, obtained for each structural element, each element of the structure has been proportioned applying Code provisions and design experience from the country representing the design example.
- For each structural element, proportioned with the new static values, the ultimate static values M^y and M^u have beed obtained and the force-displacement relationship has been determined for each element separately, and then the story resultant force-displacement relationship has been determined considering built-in material characteristics for concrete and reinforcement.
- Dynamic analyses have been carried out for the required ductility values of 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5 and 5.0 for two time histories of the seismic effect recorded during the earthquakes in Rumania (1977) and Yugoslavia (1979).

In this way, it is possible to evaluate the safety criteria applied in the individual design examples and to draw conclusions regarding the methodology of the analysis as well as the vulnerability and reliability level of the structures designed in compliance with the current national codes, standards and engineering design practice.

It should be noted that the conclusions and the results cannot be generalized. They are restricted by the type of the considered structures and the basic assumptions made in the applied analysis.

2. ANALYSIS OF THE BULGARIAN DESIGN EXAMPLE

1. Structural System

The selected building, which is analyzed as a representative design example, has 19 floors (garages, a ground floor, 16 stories and an attic). It is assumed to be in a zone of VIII MSK degree seismic intensity.

The structural system consists of structural R.C. walls in both orthogonal directions, and partially of R.C. moment resisting elements.

The built-in material is in compliance with the Bulgarian Code.

According to the simple method of analysis and by equivalent seismic force distribution over the height of the building the shear base coefficient of 2.756 percent is obtained for transverse direction.

2. Structural Analysis

Applying the TABS computer programme (three-dimensional analysis of building systems) a more accurate correlative static analysis of a three-dimensional mathematical model has been carried out. For modelling purposes, the geometrical characteristics of the structure are taken from the plans and the sections of the design, while the individual structural walls are considered as frames inter-connected by structural beams. This is particularly expressed in longitudinal direction.

The horizontal equivalent seismic forces, and their distribution over the height of the model is taken according to the design example. Also, as the design example, the building is considered to be fixed at the foundation level.

The results obtained by analysis of the wall SW6 show that the new seismic equivalent force moments are more than 37 percent lower than those of the design example. For the wall SW1, in x-x direction, the obtained moments are several times lower than those of the design example (4.5 times). This difference imposes that most of the walls and bearing moment-resisting frames in the structural system are considered together, which significantly modifies the distribution of the equivalent static forces in the base and over the height of the structure.

3. Dynamic Response Analysis

3.1. Seismic parameters

The time histories of the following recorded earthquakes have been selected :

- Vrancea Earthquake, 7 March 1977, N-S component
- Montenegro Earthquake, Petrovac record, 15 April 1979, N-S component

The purpose of the analysis was to determine the seismic intensity which induces in the structure the required ductility values of 1, 1.5, 2, 2.5, 3, 3.5, 4, 4.5 and 5.

3.2. Mathematical Modelling of Structure

For the static values determined using the more accurate static analysis, described under Item 2, proportioning of all the structural elements has been carried out applying Bulgarian Code and standards.

In Table 1 through 11 the reinforcement percentage and distribution is presented for each structural element separately. Minimum percentage of reinforcement and its distribution are determined based on the Code and current design practice in Bulgaria.

Applying a special computer programme, the ultimate stress states and deformability have been determined for each structural element proportioned in this way for x-x and y-y direction, respectively.

The resultant story P-Δ bilinear positive diagrams, representing story strength capacity and deformability of the mathematical model of the structure, have been determined for x-x and y-y direction, respectively.

The global shear base coefficients up to the yield point have been found out as follows :

in x-x direction : $Q^y = 4140$ KN ; $C_B = 0.0337$

in y-y direction : $Q^y = 3162$ KN ; $C_B = 0.0257$

3.3. Dynamic Analysis Results

Using the VIBR 4 (IZIIS) computer programme, which provides qualitative evaluation of the obtained results, dynamic analysis has been carried out for the prepared input data on the structural model and the frequency content of the recorded earthquakes.

The results obtained by the dynamic response analysis are shown in Tables 12 and 13 and Figures 1 through 4. According to the results, it can be seen that :

- In transverse direction (y-y), for elastic behaviour of the structure, i.e., ductility D = 1, the ground acceleration is 0.08 g for the Bucharest record, N-S component of the Vrancea earthquake and 0.14 g for the Petrovac record, N-S component of the Montenegro earthquake. In x-x direction, for ductility D=1, the ground acceleration is 0.03 g for the Bucharest record, N-S component of the Vrancea earthquake and 0.07 g for the Petrovac record, N-S component of the Montenegro earthquake.
- For a maximum ductility D = 5, the ground acceleration in y-y direction is 0.45 g for the Vrancea earthquake and 0.65 g for the Petrovac record, N-S component of the Montenegro earthquake. In longitudinal direction x-x the ground acceleration is 0.14 g for the Vrancea earthquake and 0.29 g for the Petrovac record, N-S component of the Montenegro earthquake.
- Considering the structural system and the built-in material, the reasonable required ductility for design purposes should range between 2 and 3. If a required ductility D = 2.5 is taken, the ground acceleration in y-y direction is 0.21 g for the Vrancea earthquake and 0.32 g for the Petrovac record, N-S component of the Montenegro earthquake. In longitudinal direction x-x the ground acceleration is 0.06 g, for the Vrancea earthquake and 0.17 g for the Petrovac record, N-S component of the Montenegro earthquake.

General Conclusions

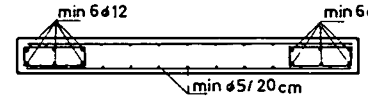
1. The structure has been designed to have a relatively low strength capacity, with a shear base coefficient of only C_B = 2.6%.
2. The stiffness of the structure in longitudinal and transverse direction differs considerably over the height (20 - 250%).
3. The applied approximative method for static analysis gives high force distribution factors from one to another structural element.
4. For a moderate ductility factor D = 2.5 the maximum displacements at the top of the building are high and not allowed by most of the national codes (Table 14).
5. The ground acceleration for a moderate ductility D = 2.5, shows that the structural resistance in x-x direction is considerably lower (3.5 times), Tables 12 and 13.

Direction	Petrovac Earthq.	Vrancea Earthq.
x-x	0.168	0.064 g
y-y	0.318 g	0.211 g

6. For a moderate required ductility D ≈ 2.5, the ultimate deformation capacity is exhausted in some structural elements, thus the capacity of the available ductility is D_{cap} < 2.5 (Fig. 5 and 6).
   Such a state creates conditions for heavy damage to structural elements and/or local failure.

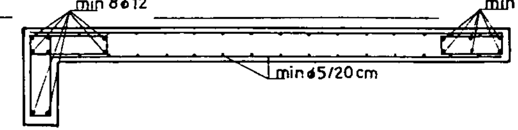
On the basis of the described conclusions it can be questioned whether the building designed according to the current Code meets the required safety criteria for both its orthogonal directions.

Table 1



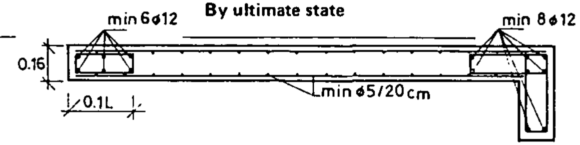
Wall	Storey	By TABS method			By simple method			By ultimate state							By simple method		
		M_E (kN·m)	Q_{g+E} (kN)	N_{g+E} (kN)	M (kN·m)	Q (kN)	N (kN)	Required reinforcement (cm^2)	Required web reinforce. (cm^2/m')	Accepted tensile reinforcement n ϕ	Accepted tensile reinforcement (cm^2)	Accepted compressed reinforcement n ϕ	Accepted compressed reinforcement (cm^2)	Accepted web reinforceme.	n ϕ	Fa = Fa' (cm^2)	Fa^{web} (nϕ/m')
SW 1	Garag.	5182	487	2637	23465	679		6.80 6.80	1.71	6ϕ12	6.79	6ϕ12	6.79	ϕ5/20			
	G.F.	4043	312	2506	21837	675		" "	"	"	"	"	"	"			
	1	3351	238	2376	20016	668		" "	"	"	"	"	"	"			
	2	2766	192	2238	18345	658		" "	"	"	"	"	"	"			
	3	2330	171	2100	16504	643		" "	"	"	"	"	"	"			
	4	1964	161	1963	14704	624		" "	"	"	"	"	"	"			
	5	1641	154	1825	12957	601		" "	"	"	"	"	"	"			
	6	1344	149	1687	11272	575		" "	"	"	"	"	"	"			
	7	1071	144	1548	9664	544		" "	"	"	"	"	"	"			
	8	819	138	1410	8139	510		" "	"	"	"	"	"	"			
	9	590	130	1270	6711	472		" "	"	"	"	"	"	"			
	10	384	122	1130	5390	429		" "	"	"	"	"	"	"			
	11	203	113	989	4188	383		" "	"	"	"	"	"	"			
	12	48	103	847	3115	333		" "	"	"	"	"	"	"			
	13	77	92	704	2182	279		" "	"	"	"	"	"	"			
	14	171	79	561	1401	221		" "	"	"	"	"	"	"			
	15	228	64	417	782	159		" "	"	"	"	"	"	"			
	16	239	43.4	272	336	93		" "	"	"	"	"	"	"			
	Attic	173	106	126	75	39		" "	"	"	"	"	"	"			

Table 2



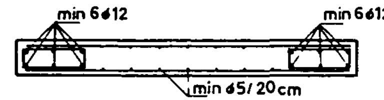
Wall	Storey	By TABS method			By simple method			By ultimate state							By simple method		
		M_E (kN·m)	Q_{g+E} (kN)	N_{g+E} (kN)	M (kN·m)	Q (kN)	N (kN)	Required reinforcement (cm^2)	Required web reinforce. (cm^2/m')	Accepted tensile reinforcement		Accepted compressed reinforcement		Accepted web reinforceme.	$n\phi$	$Fa = Fa'$ (cm^2)	Fa^{web} ($n\phi/m'$)
										$n\phi$	(cm^2)	$n\phi$	(cm^2)				
SW 2	Garag.	2533	292	1488				9.94/9.94	4.71	8ϕ14	12.32	6ϕ16	12.06	ϕ8/20			
	G.F.	1872	168	1367				9.05/6.79	2.70	8ϕ12	9.05	6ϕ12	6.79	ϕ6/20			
	1	1545	131	1258				"	2.11	"	"	"	"	ϕ6/20			
	2	1282	114,5	1153				"	2.0	"	"	"	"	ϕ5/20			
	3	1104	112	1056				"	"	"	"	"	"	"			
	4	950	113	967				"	"	"	"	"	"	"			
	5	811	115	883				"	"	"	"	"	"	"			
	6	683	116	805				"	"	"	"	"	"	"			
	7	563	116	730				"	"	"	"	"	"	"			
	8	452	115	659				"	"	"	"	"	"	"			
	9	350	113	591				"	"	"	"	"	"	"			
	10	1257	110	523				"	"	"	"	"	"	"			
	11	175	106	457				"	"	"	"	"	"	"			
	12	105	102	391				"	"	"	"	"	"	"			
	13	46	96	325				"	"	"	"	"	"	"			
	14	1.7	91	259				"	"	"	"	"	"	"			
	15	29	85	192				"	"	"	"	"	"	"			
	16	46	84	124				"	"	"	"	"	"	"			
	Attic	61.8	80	56				"	"	"	"	"	"	"			

Table 3



Wall	Storey	By TABS method			By simple method			By ultimate state							By simple method		
		M_E (kN·m)	Q_{g+E} (kN)	N_{g+E} (kN)	M (kN·m)	Q (kN)	N (kN)	Required reinforcement (cm^2)	Required web reinforce. (cm^2/m')	Accepted tensile reinforcement		Accepted compressed reinforcement		Accepted web reinforceme.	n ϕ	Fa = Fa' (cm^2)	Fa^{web} (nϕ/m')
										n ϕ	(cm^2)	n ϕ	(cm^2)				
SW 3	Garag.	2300	187	2876				6.8/9.85	2.33	6ϕ12	6.79	8ϕ12	9.05	ϕ5/20			
	G.F.	2353	118	1673				"	2.0	"	"	"	"	"			
	1	2073	93	2431				"	"	"	"	"	"	"			
	2	1818	80	2281				"	"	"	"	"	"	"			
	3	1598	74	2131				"	"	"	"	"	"	"			
	4	1398	70	1981				"	"	"	"	"	"	"			
	5	1212	66	1831				"	"	"	"	"	"	"			
	6	1036	63	1683				"	"	"	"	"	"	"			
	7	870	59	1532				"	"	"	"	"	"	"			
	8	714	56	1382				"	"	"	"	"	"	"			
	9	571	42	1232				"	"	"	"	"	"	"			
	10	438	47	1083				"	"	"	"	"	"	"			
	11	319	42	933				"	"	"	"	"	"	"			
	12	214	37	783				"	"	"	"	"	"	"			
	13	125	31	633				"	"	"	"	"	"	"			
	14	52	25	483				"	"	"	"	"	"	"			
	15	0.9	17	334				"	"	"	"	"	"	"			
	16	31	9	183				"	"	"	"	"	"	"			
	Attic	31	22	34				"	"	"	"	"	"	"			

Table 4



Wall	Storey	By TABS method			By simple method			By ultimate state							By simple method		
		M_E (kN·m)	Q_{g+E} (kN)	N_{g+E} (kN)	M (kN·m)	Q (kN)	N (kN)	Required reinforcement (cm^2)	Required web reinforce. (cm^2/m')	Accepted tensile reinforcement n ø	Accepted tensile reinforcement (cm^2)	Accepted compressed reinforcement n ø	Accepted compressed reinforcement (cm^2)	Accepted web reinforceme.	n ø	Fa = Fa' (cm^2)	Fa^{web} (nø/m')
SW 4	Garag.	41	22	-1299				6.8/6.8	2.41	6ø12	6.79	6ø12	6.79	ø6/20			
	G.F.	45	31	-1243				"	2.62	"	"	"	"	"			
	1	51	31	-1154				"	2.59	"	"	"	"	"			
	2	53	35	-1049				"	2.55	"	"	"	"	"			
	3	51	35	- 937				"	2.52	"	"	"	"	"			
	4	49	35	- 824				"	2.46	"	"	"	"	"			
	5	46	34	- 714				"	2.38	"	"	"	"	"			
	6	43	32	- 607				"	2.28	"	"	"	"	"			
	7	40	31	- 507				"	2.17	"	"	"	"	"			
	8	36	29	- 412				"	2.03	"	"	"	"	"			
	9	32	27	- 326				"	2.0	"	"	"	"	ø5/20			
	10	28	24	- 248				"	"	"	"	"	"	"			
	11	25	22	- 179				"	"	"	"	"	"	"			
	12	21	19	- 120				"	"	"	"	"	"	"			
	13	16	16	- 71				"	"	"	"	"	"	"			
	14	12	13	- 34				"	"	"	"	"	"	"			
	15	8	9	- 10				"	"	"	"	"	"	"			
	16	4.7	6	+ 2				"	"	"	"	"	"	"			
	Attic	2.3	6.51	+ 3				"	"	"	"	"	"	"			

Table 5

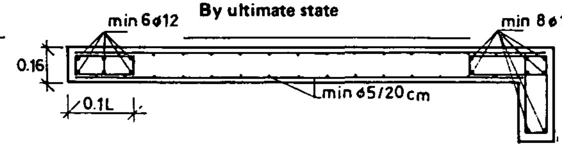
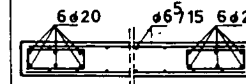
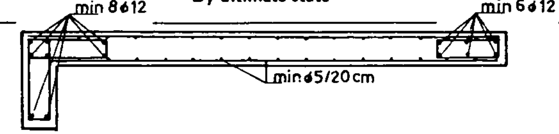
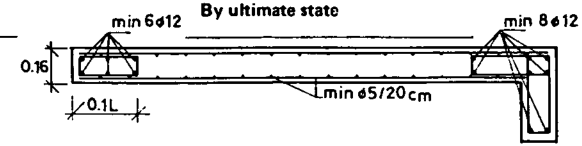
Wall	Storey	By TABS method			By simple method			By ultimate state 							By simple method 		
		M_E (kN·m)	Q_{g+E} (kN)	N_{g+E} (kN)	M (kN·m)	Q (kN)	N (kN)	Required reinforcement (cm^2)	Required web reinforce. (cm^2/m')	Accepted tensile reinforcement n ø	Accepted tensile reinforcement (cm^2)	Accepted compressed reinforcement n ø	Accepted compressed reinforcement (cm^2)	Accepted web reinforceme.	n ø	Fa = Fa' (cm^2)	Fa^{web} (nø/m')
SW 6	Garag.	7443	306	3123	10232	298		11.83	2.7	6ø16	12.06	8ø14	12.32	ø6/20	6ø20	18.85	ø6 5/15
	G.F.	6746	290	2965	9517	296		8.45/9.05	2.55	6ø14	9.24	8ø12	9.05	"	"	"	"
	1	6121	273	2808	8806	293		6.8/9.05	2.4	6ø12	6.79	"	"	"	"	"	"
	2	5441	258	2642	7985	288		"	2.28	"	"	"	"	"	"	"	"
	3	4814	246	2475	7179	281		"	2.17	"	"	"	"	"	"	"	"
	4	4229	236	2309	6392	272		"	2.09	"	"	"	"	"	"	"	"
	5	3678	227	2143	5629	262		"	2.0	"	"	"	"	ø5/20	"	"	"
	6	3158	218	1977	4896	250		"	"	"	"	"	"	"	"	"	"
	7	2668	209	1811	4196	234		"	"	"	"	"	"	"	"	"	"
	8	2209	198	1645	3532	222		"	"	"	"	"	"	"	"	"	"
	9	1782	187	1479	2911	205		"	"	"	"	"	"	"	"	"	"
	10	1391	174	1313	2337	187		"	"	"	"	"	"	"	"	"	"
	11	1038	159	1147	1815	166		"	"	"	"	"	"	"	"	"	"
	12	727	144	980	1349	144		"	"	"	"	"	"	"	"	"	"
	13	461	126	814	945	121		"	"	"	"	"	"	"	"	"	"
	14	245	108	647	607	96		"	"	"	"	"	"	"	"	"	"
	15	84	87	480	339	69		"	"	"	"	"	"	"	"	"	"
	16	15	65	312	146	40		"	"	"	"	"	"	"	"	"	"
	Attic	47	53	145	32	17		"	"	"	"	"	"	"	"	"	"

Table 6



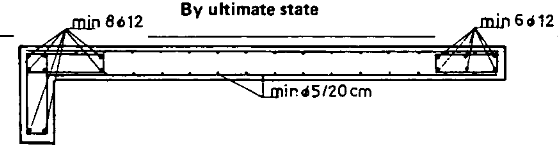
Wall	Storey	By TABS method			By simple method			By ultimate state							By simple method		
		M_E (kN·m)	Q_{g+E} (kN)	N_{g+E} (kN)	M (kN·m)	Q (kN)	N (kN)	Required reinforcement (cm^2)	Required web reinforce. (cm^2/m')	Accepted tensile reinforcement n ϕ	Accepted tensile reinforcement (cm^2)	Accepted compressed reinforcement n ϕ	Accepted compressed reinforcement (cm^2)	Accepted web reinforceme.	n ϕ	Fa = Fa' (cm^2)	Fa^{web} (nϕ/m')
SW 7	Garag.	12614	1015	-1179				86.18	8.17	14ϕ28	86.21	14ϕ28	86.21	ϕ 12/20			
	G.F.	11727	1213	-1040				78.84	9.76	13ϕ28	80.05	13ϕ28	80.05	"			
	1	10963	1282	- 762				69.97	10.32	12ϕ28	73.89	12ϕ28	73.89	"			
	2	9839	1302	- 433				58.31	10.48	12ϕ25	58.90	12ϕ25	58.9	"			
	3	8731	1287	- 107				46.97	10.36	10ϕ25	49.09	10ϕ25	49.09	"			
	4	7653	1253	198				36.32	10.05	8ϕ25	39.27	8ϕ25	39.27	"			
	5	6618	1207	473				26.49	9.72	8ϕ22	30.41	8ϕ22	30.41	"			
	6	5635	1152	714				17.54	9.28	8ϕ18	20.36	8ϕ18	20.36	"			
	7	4709	1089	919				9.50	8.77	8ϕ14	12.32	8ϕ14	12.32	"			
	8	3847	1018	1084				9.05/6.8	8.19	8ϕ12	9.05	6ϕ12	6.79	"			
	9	3052	939	1207				"	7.56	"	"	"	"	ϕ 10/20			
	10	2328	852	1285				"	6.86	"	"	"	"	"			
	11	1683	758	1315				"	6.10	"	"	"	"	"			
	12	1121	656	1294				"	5.29	"	"	"	"	"			
	13	648	547	1219				"	4.41	"	"	"	"	ϕ 8/20			
	14	271	432	1088				"	3.48	"	"	"	"	"			
	15	4	310	897				"	2.50	"	"	"	"	ϕ 6/20			
	16	166	189	643				"	2.0	"	"	"	"	ϕ 5/20			
	Attic	211	78	326				"	2.0	"	"	"	"	"			

Table 7



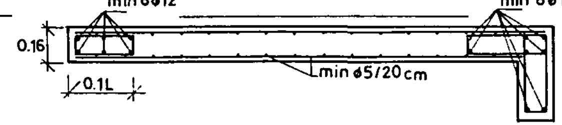
Wall	Storey	By TABS method			By simple method			By ultimate state							By simple method		
		M_E (kN·m)	Q_{g+E} (kN)	N_{g+E} (kN)	M (kN·m)	Q (kN)	N (kN)	Required reinforcement (cm^2)	Required web reinforce. (cm^2/m')	Accepted tensile reinforcement n ϕ	Accepted tensile reinforcement (cm^2)	Accepted compressed reinforcement n ϕ	Accepted compressed reinforcement (cm^2)	Accepted web reinforceme.	n ϕ	Fa = Fa' (cm^2)	Fa^{web} (nϕ/m')
SW 10	Garag.	5259	618	9933				62.59	7.63	13ϕ25	63.81	13ϕ25	63.81	ϕ 10/20			
	G.F.	4801	707	9374				48.78	8.73	13ϕ22	49.43	13ϕ22	49.43	ϕ 12/20			
	1	4509	734	8669				35.43	9.06	12ϕ20	37.70	12ϕ20	37.70	"			
	2	4051	739	7890				21.31	9.13	9ϕ18	22.90	9ϕ18	22.90	"			
	3	3612	727	7108				9.33	8.98	9ϕ12	10.18	9ϕ12	10.18	"			
	4	3186	705	6344				6.8/9.05	8.71	6ϕ12	6.79	8ϕ12	9.05	"			
	5	2776	677	5605				"	8.36	"	"	"	"	"			
	6	2385	644	4899				"	7.95	"	"	"	"	"			
	7	2015	606	4226				"	7.49	"	"	"	"	ϕ 10/20			
	8	1667	564	3592				"	6.97	"	"	"	"	"			
	9	1344	519	2998				"	6.40	"	"	"	"	"			
	10	1046	468	2448				"	5.78	"	"	"	"	"			
	11	277	414	1946				"	4.11	"	"	"	"	ϕ 8/20			
	12	537	354	1494				"	4.38	"	"	"	"	"			
	13	331	292	1096				"	3.6	"	"	"	"	"			
	14	159	224	756				"	2.77	"	"	"	"	ϕ 6/20			
	15	26	153	475				"	2.00	"	"	"	"	ϕ 5/20			
	16	65	81	256				"	"	"	"	"	"	"			
	Attic	97	16	106				"	"	"	"	"	"	"			

Table 8



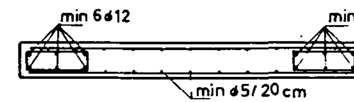
Wall	Storey	By TABS method			By simple method			By ultimate state							By simple method		
		M_E (kN·m)	Q_{g+E} (kN)	N_{g+E} (kN)	M (kN·m)	Q (kN)	N (kN)	Required reinforcement (cm^2)	Required web reinforce. (cm^2/m')	Accepted tensile reinforcement n ø	Accepted tensile reinforcement (cm^2)	Accepted compressed reinforcement n ø	Accepted compressed reinforcement (cm^2)	Accepted web reinforceme.	n ø	Fa = Fa' (cm^2)	Fa^{web} (nø/m')
SW 14	Garag.	5224	569	-279				44.91	6.72	12ø22	45.62	12ø22	45.62	ø 10/20			
	G.F.	4522	576	-207				37.81	6.8	12ø20	37.70	12ø20	37.70	"			
	1	4082	568	- 49				31.72	6.72	10ø20	31.42	10ø20	31.42	"			
	2	3574	556	137				24.71	6.57	10ø18	25.45	10ø18	25.45	"			
	3	3137	542	320				18.45	6.41	10ø16	20.11	10ø16	20.11	"			
	4	2735	525	489				12.79	6.21	9ø14	13.86	9ø14	13.86	"			
	5	2359	506	636				9.05/7.69	5.98	8ø12	9.05	7ø12	7.92	"			
	6	2005	483	762				9.05/6.8	5.71	"	"	6ø12	6.79	"			
	7	1672	458	863				"	5.41	"	"	"	"	"			
	8	1361	429	940				"	5.07	"	"	"	"	"			
	9	1073	397	989				"	4.7	"	"	"	"	ø 8/20			
	10	810	362	1009				"	4.28	"	"	"	"	"			
	11	574	324	1000				"	3.83	"	"	"	"	"			
	12	365	282	959				"	3.34	"	"	"	"	"			
	13	188	237	884				"	2.80	"	"	"	"	ø 6/20			
	14	43	188	774				"	2.23	"	"	"	"	"			
	15	64	138	627				"	2.0	"	"	"	"	ø 5/20			
	16	129	87	442				"	"	"	"	"	"	"			
	Attic	130	49	216				"	"	"	"	"	"	"			

Table 9



Wall	Storey	By TABS method			By simple method			By ultimate state							By simple method		
		M_E (kN·m)	Q_{g+E} (kN)	N_{g+E} (kN)	M (kN·m)	Q (kN)	N (kN)	Required reinforcement (cm^2)	Required web reinforce. (cm^2/m')	Accepted tensile reinforcement n ϕ	Accepted tensile reinforcement (cm^2)	Accepted compressed reinforcement n ϕ	Accepted compressed reinforcement (cm^2)	Accepted web reinforceme.	n ϕ	Fa = Fa′ (cm^2)	Fa^{web} (nϕ/m′)
SW 19	Garag.	2904	395	6677				32.62	6.46	13ϕ18	35.08	13ϕ18	33.08	ϕ 10/20			
	G.F.	2462	395	6312				22.16	6.45	9ϕ18	22.9	9ϕ18	22.9	"			
	1	2239	394	5856				13.87	6.43	9ϕ14	13.86	9ϕ14	13.86	"			
	2	1974	391	5353				6.8/9.05	6.38	6ϕ12	6.79	8ϕ12	9.05	"			
	3	1747	384	4847				"	6.28	"	"	"	"	"			
	4	1535	374	4352				"	6.11	"	"	"	"	"			
	5	1333	361	3873				"	5.9	"	"	"	"	"			
	6	1141	345	3412				"	5.64	"	"	"	"	"			
	7	960	327	2972				"	5.35	"	"	"	"	"			
	8	789	307	2555				"	5.01	"	"	"	"	ϕ 8/20			
	9	629	284	2160				"	4.64	"	"	"	"	"			
	10	483	259	1792				"	4.24	"	"	"	"	"			
	11	349	232	1452				"	3.79	"	"	"	"	"			
	12	231	203	1143				"	3.31	"	"	"	"	"			
	13	127	171	864				"	2.8	"	"	"	"	ϕ 6/20			
	14	40	137	621				"	2.25	"	"	"	"	"			
	15	27	102	413				"	2.0	"	"	"	"	ϕ 5/20			
	16	71	68	243				"	"	"	"	"	"	"			
	Attic	80	74	109				"	"	"	"	"	"	"			

Table 10



Wall	Storey	By TABS method M_E (kN·m)	By TABS method Q_{g+E} (kN)	By TABS method N_{g+E} (kN)	By simple method M (kN·m)	By simple method Q (kN)	By simple method N (kN)	By ultimate state: Required reinforcement (cm²)	Required web reinforce. (cm²/m')	Accepted tensile reinforcement n ϕ	Accepted tensile reinforcement (cm²)	Accepted compressed reinforcement n ϕ	Accepted compressed reinforcement (cm²)	Accepted web reinforceme.	By simple method: n ϕ	Fa = Fa' (cm²)	Fa^{web} (nϕ/m')
SW 12	Garag.	35427	765	10278				14.09	3.27	6ϕ18	15.27	6ϕ18	15.27	ϕ 8/20			
	G.F.	33592	870	9753				9.93	3.72	6ϕ16	12.06	6ϕ16	12.06	"			
	1	31505	965	9228				6.8/6.8	4.12	6ϕ12	6.79	6ϕ12	6.29	"			
	2	28826	1009	8683				"	4.31	"	"	"	"	"			
	3	26035	1018	8137				"	4.35	"	"	"	"	"			
	4	23231	1005	7592				"	4.29	"	"	"	"	"			
	5	20473	977	7046				"	4.17	"	"	"	"	"			
	6	17800	938	6500				"	4.01	"	"	"	"	"			
	7	15243	891	5954				"	3.81	"	"	"	"	"			
	8	12824	836	5409				"	3.57	"	"	"	"	"			
	9	10565	774	4864				"	3.31	"	"	"	"	"			
	10	8482	706	4318				"	3.02	"	"	"	"	"			
	11	6595	631	3772				"	2.70	"	"	"	"	ϕ 6/20			
	12	4919	552	3227				"	2.36	"	"	"	"	"			
	13	3472	467	2681				"	2.0	"	"	"	"	ϕ 5/20			
	14	2265	379	2136				"	"	"	"	"	"	"			
	15	1311	289	1590				"	"	"	"	"	"	"			
	16	615	206	1044				"	"	"	"	"	"	"			
	Attic	165	128	499				"	"	"	"	"	"	"			

Table 11

Wall	Storey	By TABS method			By simple method			By ultimate state							By simple method		
		M_E (kN·m)	Q_{g+E} (kN)	N_{g+E} (kN)	M (kN·m)	Q (kN)	N (kN)	Required reinforcement (cm^2)	Required web reinforce. (cm^2/m')	Accepted tensile reinforcement		Accepted compressed reinforcement		Accepted web reinforceme.	n ϕ	Fa = Fa' (cm^2)	Fa^{web} (nϕ/m')
										n ϕ	(cm^2)	n ϕ	(cm^2)				
SW 21	Garag.	5009	265	3198				6.8/6.8	2.34	6ϕ12	6.79	6ϕ12	6.79	ϕ 6/20			
	G.F.	4373	189	3037				"	2.0	"	"	"	"	ϕ 5/20			
	1	3921	158	2876				"	"	"	"	"	"	"			
	2	3478	141	2706				"	"	"	"	"	"	"			
	3	3085	130	2534				"	"	"	"	"	"	"			
	4	2722	122	2365				"	"	"	"	"	"	"			
	5	2381	116	2194				"	"	"	"	"	"	"			
	6	2058	110	2024				"	"	"	"	"	"	"			
	7	1781	104	1853				"	"	"	"	"	"	"			
	8	1462	97	1683				"	"	"	"	"	"	"			
	9	1192	90	1512				"	"	"	"	"	"	"			
	10	943	82	1342				"	"	"	"	"	"	"			
	11	715	73	1171				"	"	"	"	"	"	"			
	12	512	63	1001				"	"	"	"	"	"	"			
	13	336	53	830				"	"	"	"	"	"	"			
	14	189	41	660				"	"	"	"	"	"	"			
	15	76.0	27	489				"	"	"	"	"	"	"			
	16	2.31	9.8	318				"	"	"	"	"	"	"			
	Attic	22.5	13	148				"	"	"	"	"	"	"			

Direction x–x **Table 12**

Earthquake Petrovac 1979, N–S					Earthquake Vrancea 1977, N–S				
D	Floor	Δ_{max}	Floor	a_o (g)	D	Floor	Δ_{max}	Floor	a_o (g)
1.0	1	1.245	18	0.074	1.0	18	1.303	18	0.030
1.5	1	1.524	18	0.091	1.5	18	2.014	18	0.042
2.0	1	1.711	18	0.104	2.0	18	2.712	18	0.053
2.5	1	2.525	18	0.168	2.5	18	3.344	18	0.064
3.0	1	2.457	17	0.191	3.0	18	4.046	18	0.079
3.5	1	2.473	17	0.212	3.5	18	4.647	18	0.097
4.0	1	2.463	17	0.235	4.0	1	4.938	18	0.116
4.5	1	2.534	17	0.261	4.5	1	5.065	18	0.126
5.0	1	2.633	17	0.287	5.0	1	5.174	18	0.135

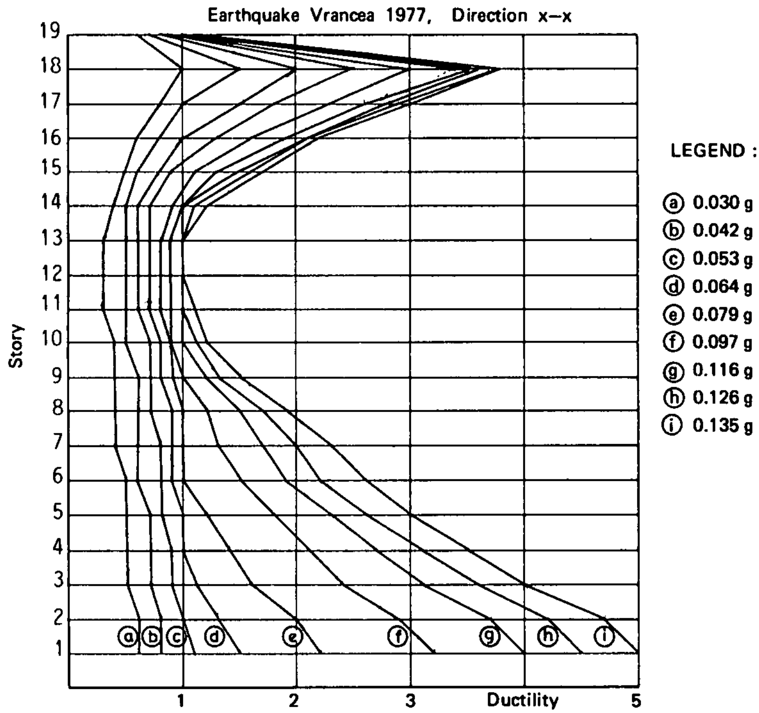
Direction y–y **Table 13**

Earthquake Petrovac 1979, N–S					Earthquake Vrancea 1977, N–S				
D	Floor	Δ_{max}	Floor	a_o (g)	D	Floor	Δ_{max}	Floor	a_o (g)
1.0	1	1.523	17	0.139	1.0	18	2.371	18	0.0813
1.5	1	2.132	17	0.196	1.5	17, 18	3.754	18	0.123
2.0	1	2.744	17	0.261	2.0	17,18	4.912	18	0.161
2.5	1	3.154	17	0.318	2.5	17,18	6.083	18	0.211
3.0	1	3.362	17	0.388	3.0	1,17,18	7.298	18	0.277
3.5	1	3.648	17	0.448	3.5	1	7.84	17	0.316
4.0	1	3.879	17	0.505	4.0	1	8.64	17	0.369
4.5	1	4.058	17	0.575	4.5	1	9.25	17	0.407
5.0	1	4.208	17	0.645	5.0	1	9.87	17	0.446

Maximum displacement at top of building for required ductility D = 2.5

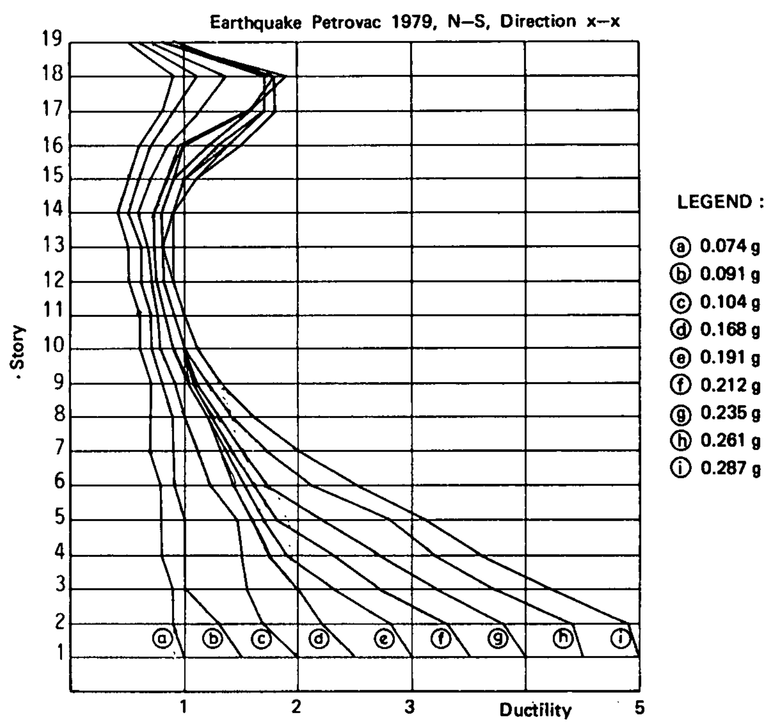
Height of building H = 51.5 m **Table 14**

Direction	x–x (longitudinal)		y–y (transverse)	
Earthquake	Petrovac N–S	Vrancea N–S	Petrovac N–S	Vrancea N–S
Acceleration of g	0.168	0.064	0.32	0.211
Maximum deformation Δ_{max} (cm)	9.02	9.375	20.66	29.639
Time of occurrence Δ_{max} (sec)	8.45	4.78	8.09	4.80
$\frac{H}{\Delta_{max}}$ relationship	571	549	249	174



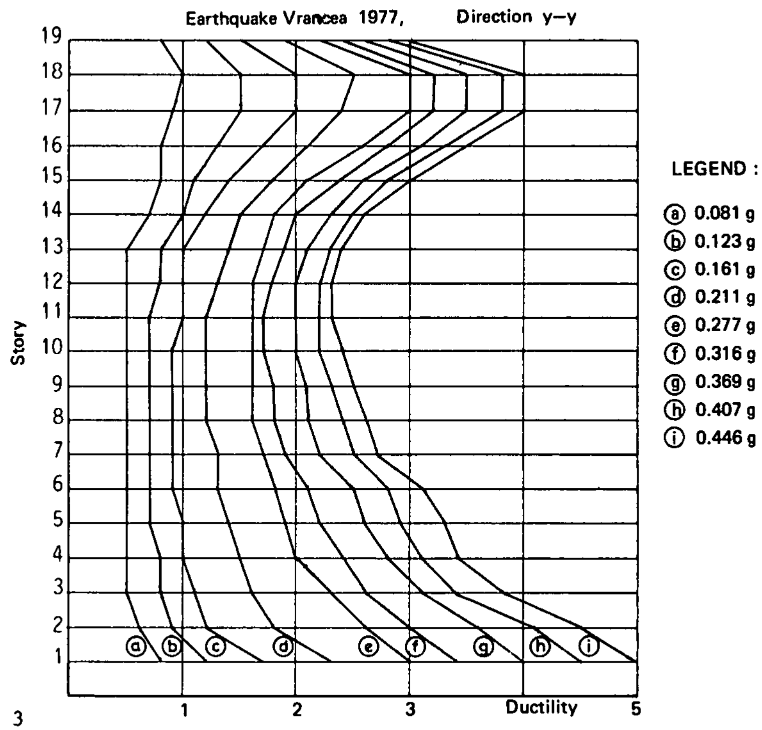
Earthquake Vrancea 1977, Direction x–x
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
Story
a b c d e f g h i
1
2
3
Ductility
5
LEGEND :
a 0.030 g
b 0.042 g
c 0.053 g
d 0.064 g
e 0.079 g
f 0.097 g
g 0.116 g
h 0.126 g
i 0.135 g

Fig. 1



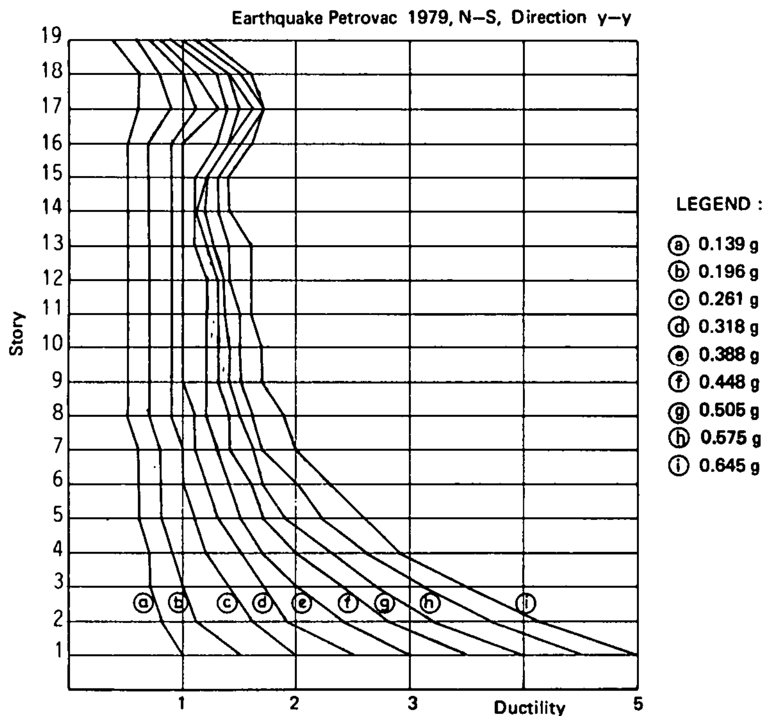
Earthquake Petrovac 1979, N–S, Direction x–x
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
Story
a b c d e f g h i
1
2
3
Ductility
5
LEGEND :
a 0.074 g
b 0.091 g
c 0.104 g
d 0.168 g
e 0.191 g
f 0.212 g
g 0.235 g
h 0.261 g
i 0.287 g

Fig. 2



Earthquake Vrancea 1977, Direction y–y
Story
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
a
b
c
d
e
f
g
h
i
1
2
3
Ductility
5
LEGEND :
a 0.081 g
b 0.123 g
c 0.161 g
d 0.211 g
e 0.277 g
f 0.316 g
g 0.369 g
h 0.407 g
i 0.446 g

Fig. 3



Earthquake Petrovac 1979, N–S, Direction y–y
Story
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
a
b
c
d
e
f
g
h
i
1
2
3
Ductility
5
LEGEND :
a 0.139 g
b 0.196 g
c 0.261 g
d 0.318 g
e 0.388 g
f 0.448 g
g 0.505 g
h 0.575 g
i 0.645 g

Fig. 4

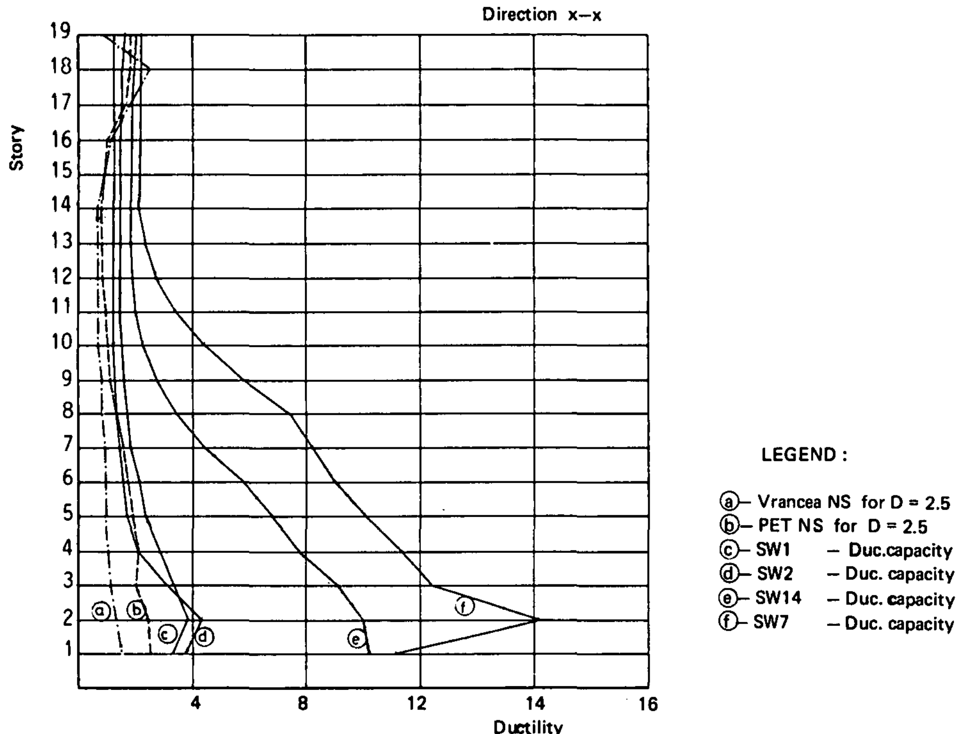

Direction x–x
Story
Ductility
4
8
12
14
16
LEGEND :
ⓐ– Vrancea NS for D = 2.5
ⓑ– PET NS for D = 2.5
ⓒ– SW1 – Duc.capacity
ⓓ– SW2 – Duc. capacity
ⓔ– SW14 – Duc. capacity
ⓕ– SW7 – Duc. capacity

Fig. 5

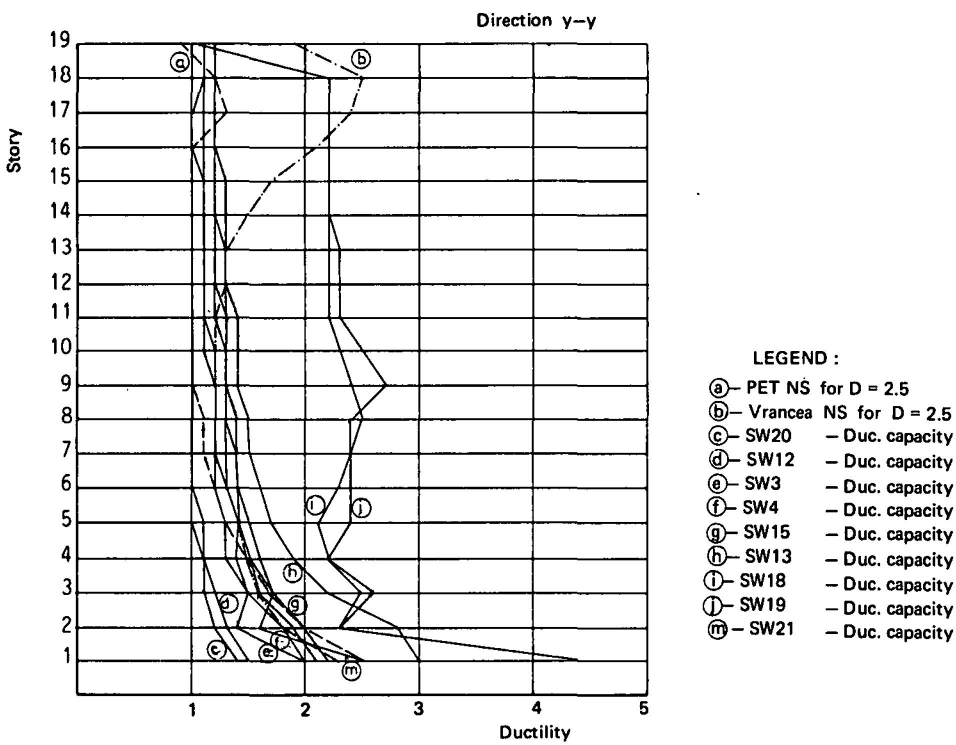

Direction y–y
Story
Ductility
1
2
3
4
5
LEGEND :
ⓐ– PET NS for D = 2.5
ⓑ– Vrancea NS for D = 2.5
ⓒ– SW20 – Duc. capacity
ⓓ– SW12 – Duc. capacity
ⓔ– SW3 – Duc. capacity
ⓕ– SW4 – Duc. capacity
ⓖ– SW15 – Duc. capacity
ⓗ– SW13 – Duc. capacity
ⓘ– SW18 – Duc. capacity
ⓙ– SW19 – Duc. capacity
ⓜ– SW21 – Duc. capacity

Fig. 6

3. ANALYSIS OF THE RUMANIAN DESIGN EXAMPLE

1. Structural System

The considered building has 9 floors. Based on the Rumanian Code for average soil conditions the shear base coefficient is determined to be 7.6 percent and it is equal in both orthogonal directions. The distribution of the equivalent seismic forces is carried out to comply with the Rumanian standards. The seismic forces and their distribution have been applied also in our analysis, as well as the quality of the built-in materials (reinforced concrete and reinforcement).

The structural system consists of structural R.C. walls, so that two facade frames are formed in longitudinal direction.

2. Structural Analysis

Applying the TABS computer programme a more accurate static analysis of a three-dimensional mathematical model has been carried out. For modelling purposes, the geometrical characteristics are taken as in the existing design example and in longitudinal direction the effect of the two facade frames has been taken into consideration.

The horizontal equivalent seismic forces and their distribution over the height of the building is taken according to the design example. Also, as in the design example, the building is considered to be fixed at the above basement level.

The results of the more accurate static analysis of the mathematical model are given in Tables 15 through 20.

On the basis of the results from the analysis of the transverse direction walls (DT1, DT2 and DT3) it can be concluded that the equivalent seismic force moments are 21 percent to 44 percent higher at the fixation with respect to those of the design example. These differences over the height of the building are rather smaller and they differ from wall to wall.

In longitudinal direction x-x, by correlative analysis the static values are determined in both longitudinal walls DL1 and the two peripheral frames.

The global shear base coefficients were found to be :

in x-x direction : $Q^y = 7469$ KN ; $C_B = 0.27$

in y-y direction : $Q^y = 4948$ KN ; $C_B = 0.18$

3. Dynamic Response Analysis

3.1. Seismic Parameters

Two time histories of the following recorded earthquakes have been applied :

- Vrancea Earthquake, 7 March 1977, N-S component
- Montenegro Earthquake, Petrovac Record, N-S component

The analysis was to determine the seismic intensity which induces in the structure the required ductility values of 1, 1.5, 2, 2.5, 3, 3.5, 4, 4.5 and 5.

3.2. Mathematical Modelling of Structure

Based on the obtained results according to Item 2, proportioning of the structural elements in compliance with the Rumanian Code and the current engineering and design practice has been carried out.

Presented in Tables 15 through 20 is the percentage and distribution of reinforcement of each element, separately, applying a minimum percentage of reinforcement according to the Rumanian Code.

For these proportioned structural elements, applying a special computer programme the ultimate stress states and deformations have been determined for each element and each direction, respectively.

The resultant story P-Δ bilinear positive diagrams, representing story strength capacity and deformability of the mathematical model of the structure, have been determined for x-x and y-y direction, respectively.

3.3. Dynamic Analysis Results

Using the VIBR 4 computer programme, which provides qualitative evaluation of the obtained results, dynamic response analysis has been carried out for the prepared input data on the structural model and the frequency content of the recorded earthquakes.

The results obtained by the dynamic response analysis are presented in Tables 21 and 22 and Figures 7 through 10. Based on the results it can be seen that :

- In transverse direction (y-y), for elastic behaviour of the structure, i.e., ductility D = 1, the ground acceleration is 0.123 g for the Bucharest record, N-S component of the Vrancea earthquake, and 0.152 g for the Petrovac record, N-S component of the Montenegro earthquake. In x-x direction for D = 1 the ground acceleration is 0.149 g for the Bucharest record, N-S component of the Vrancea earthquake and 0.067 g for the Petrovac record, N-S component of the Montenegro earthquake.
- For a maximum ductility D = 5, the ground acceleration in y-y direction is 0.379 g for the Vrancea earthquake and 0.797 g for the Petrovac record, N-S component. In longitudinal direction x-x the ground acceleration is 0.427 g for the Vrancea earthquake and 0.401 g for the Petrovac record, N-S component of the Montenegro earthqauke.
- Considering the structural system and the built-in materials, the required ductility for design purposes should range between 2 and 3. If the required ductility D = 2.5 is taken, the ground acceleration for the Vrancea earthquake in y-y direction is 0.246 g, while for the Petrovac record, N-S component it is 0.370 g. In x-x direction the ground acceleration is 0.254 g for the Vrancea earthquake and 0.118 g for the Petrovac record, N-S component for the Montenegro earthquake.

General Conclusions

1. The structure has been designed with sufficient strength capacity.
2. The stiffness of the structure in longitudinal and transverse direction differs considerably (by more than four times).
3. By application of an approximative method of static analysis, force redistribution over some structural elements is carried out.
4. For a moderate ductility factor D = 2.5 the maximum displacements at the top of the building are high (Table 23).
5. For a moderate required ductility D = 2.5 (which corresponds to the average design earthquake conditions), structural walls with ductility capacity lower than 2.5 exist only in a small number of higher floor structural elements (Fig. 11 and 12).

Table 15

Wall	Storey	By TABS method			By simple method			By ultimate state							By simple method		
		M_E (kN·m)	Q_g+E (kN)	N_g+E (kN)	M (kN·m)	Q (kN)	N (kN)	Required reinforcem. (cm^2)	Required web reinforce. (cm^2/m')	Accepted tensile reinforcement n ϕ	Accepted tensile reinforcement (cm^2)	Accepted compressed reinforcement n ϕ	Accepted compressed reinforcement (cm^2)	Accepted web reinforcem.	n ϕ	Fa = Fa' (cm^2)	web Fa ($n\phi/m'$)
DT1	1	2734	180	1260	3945	239.1	16 10	9.43/4.71	2.8	12ø10	9.43	6ø10	4.71	ø6/20			
	2	2239	173	1120				"	"	"	"	"	"	"			
	3	1765	157	980				"	2.0	"	"	"	"	ø5/20			
	4	1334	140	840				"	"	"	"	"	"	"			
	5	949.9	121	699.8				"	"	"	"	"	"	"			
	6	618	99	560				"	"	"	"	"	"	"			
	7	346	73	420				"	"	"	"	"	"	"			
	8	145	44	280				"	"	"	"	"	"	"			
	9	25	9.11	140				"	"	"	"	"	"	"			

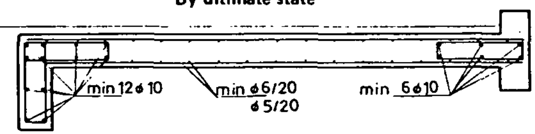


Table 16

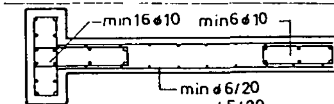
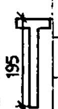
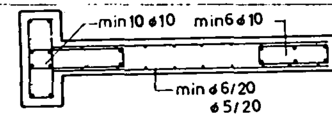
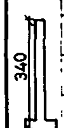
Wall	Storey	By TABS method			By simple method			By ultimate state							By simple method		
																	
										Accepted tensile reinforcement		Accepted compressed reinforcement					
		M_E (kN·m)	Q_g+E (kN)	N_g+E (kN)	M (kN·m)	Q (kN)	N (kN)	Required reinforcem. (cm^2)	Required web reinforce. (cm^2/m')	n ϕ	(cm^2)	n ϕ	(cm^2)	Accepted web reinforcem.	n ϕ	Fa = Fa' (cm^2)	web Fa (nϕ/m')
DT2 	1	161.2	46	124	208			12.57/4.71	2.8	16ϕ10	12.57	6ϕ10	4.71	ϕ6/20			
	2	130	49	130	134			"	"	"	"	"	"	"			
	3	113	52	143	88.7			"	2.0	"	"	"	"	ϕ5/20			
	4	94.5	51	153	51.1			"	"	"	"	"	"	"			
	5	75.2	48	156	22.9			"	"	"	"	"	"	"			
	6	56	42.5	149.6	5.5			"	"	"	"	"	"	"			
	7	37.5	36	132	-14.2			"	"	"	"	"	"	"			
	8	21	28	100	-17.7			"	"	"	"	"	"	"			
	9	6.8	22	54	-12.4			"	"	"	"	"	"	"			

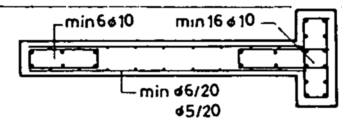
Table 17





Wall	Storey	By TABS method			By simple method			By ultimate state							By simple method		
		M_E (kN·m)	Q_g+E (kN)	N_g+E (kN)	M (kN·m)	Q (kN)	N (kN)	Required reinforcem. (cm^2)	Required web reinforce. (cm^2/m')	Accepted tensile reinforcement n ϕ	Accepted tensile reinforcement (cm^2)	Accepted compressed reinforcement n ϕ	Accepted compressed reinforcement (cm^2)	Accepted web reinforcem.	n ϕ	Fa = Fa' (cm^2)	web Fa (nϕ/m')
DT2	1	465.3	85.4	1234	691			7.85/4.71	2.8	10ϕ10	7.85	6ϕ10	4.71	ϕ6/20			
	2	371.7	89	1077	446			"	"	"	"	"	"	"			
	3	305.5	90.3	914	294			"	2.0	"	"	"	"	ϕ5/20			
	4	245	88	753	170			"	"	"	"	"	"	"			
	5	186.08	81.3	599	76			"	"	"	"	"	"	"			
	6	131.2	72	454	-18			"	"	"	"	"	"	"			
	7	81.6	60	321	-47			"	"	"	"	"	"	"			
	8	39.7	46	202	-59			"	"	"	"	"	"	"			
	9	8.9	31.04	97	-41			"	"	"	"	"	"	"			

Table 18



Wall	Storey	By TABS method			By simple method			By ultimate state							By simple method		
		M_E (kN·m)	Q_g+E (kN)	N_g+E (kN)	M (kN·m)	Q (kN)	N (kN)	Required reinforcem. (cm^2)	Required web reinforce. (cm^2/m')	Accepted tensile reinforcement n ϕ	Accepted tensile reinforcement (cm^2)	Accepted compressed reinforcement n ϕ	Accepted compressed reinforcement (cm^2)	Accepted web reinforcem.	n ϕ	Fa = Fa' (cm^2)	web Fa (nϕ/m')
DT3	1	2497	168	1920	3034			4.71/12.57	2.8	6ϕ10	4.71	16ϕ10	12.57	ϕ6/20			
	2	2036	159	1707	2529			"	"	"	"	"	"	"			
	3	1599	143	1493	2033			"	2.0	"	"	"	"	ϕ5/20			
	4	1206	127	1280	1574			"	"	"	"	"	"	"			
	5	857	110	1067	1143			"	"	"	"	"	"	"			
	6	556	90	854	764			"	"	"	"	"	"	"			
	7	309	66	640	450			"	"	"	"	"	"	"			
	8	127	39	427	209			"	"	"	"	"	"	"			
	9	19.6	7.17	213	54.6			"	"	"	"	"	"	"			

Table 19

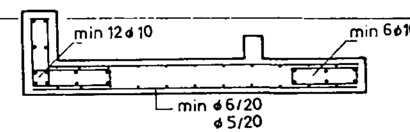
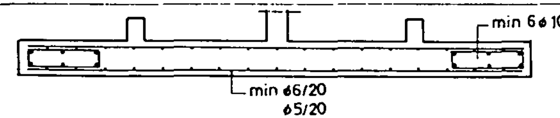
Wall	Storey	By TABS method			By simple method			By ultimate state							By simple method		
																	
								Required reinforcem.	Required web reinforce.	Accepted tensile reinforcement		Accepted compressed reinforcement		Accepted web reinforcem.			
		M_E (kN·m)	Q_g+E (kN)	N_g+E (kN)	M (kN·m)	Q (kN)	N (kN)	(cm^2)	(cm^2/m')	n ϕ	(cm^2)	n ϕ	(cm^2)		n ϕ	Fa = Fa' (cm^2)	web Fa (nϕ/m')
DL1	1	976	232	790				9.43/4.71	2.8	12ø10	9.43	6ø10	4.71	ø6/20			
	2	615	207	713				"	"	"	"	"	"	"			
	3	412	185	666				"	2.0	"	"	"	"	ø5/20			
	4	283	166	621				"	"	"	"	"	"	"			
	5	182	145	567				"	"	"	"	"	"	"			
	6	96.3	120	497				"	"	"	"	"	"	"			
	7	22.5	91	407				"	"	"	"	"	"	"			
	8	34.2	54.6	292				"	"	"	"	"	"	"			
	9	57	9.6	154				"	"	"	"	"	"	"			

Table 20



Wall	Storey	By TABS method			By simple method			By ultimate state							By simple method		
										Accepted tensile reinforcement		Accepted compressed reinforcement					
		M_E (kN·m)	Q_g+E (kN)	N_g+E (kN)	M (kN·m)	Q (kN)	N (kN)	Required reinforcem. (cm^2)	Required web reinforce. (cm^2/m')	n ø	(cm^2)	n ø	(cm^2)	Accepted web reinforcem.	n ø	Fa = Fa' (cm^2)	web Fa (nø/m')
DL1	1	2311	525	2822				4.71/4.71	2.8	6ø10	4.71	6ø10	4.71	ø6/20			
	2	1795	572	2504				"	"	"	"	"	"	"			
	3	1445	561	2188				"	2.0	"	"	"	"	ø5/20			
	4	1158	524	1874				"	"	"	"	"	"	"			
	5	887	465	1560				"	"	"	"	"	"	"			
	6	627	389	1248				"	"	"	"	"	"	"			
	7	382	299	936				"	"	"	"	"	"	"			
	8	165	196	624				"	"	"	"	"	"	"			
	9	94.0	86	312				"	"	"	"	"	"	"			

Direction x–x Table 21

Earthquake Petrovac 1979, N–S					Earthquake Vrancea 1977, N–S				
D	Floor	Δ_{max}	Floor	a_o (g)	D	Floor	Δ_{max}	Floor	a_o (g)
1.0	9	0.381	9	0.067	1.0	9	0.355	9	0.149
1.5	9	0.535	9	0.083	1.5	9	0.532	9	0.184
2.0	9	0.723	9	0.098	2.0	9	0.711	9	0.211
2.5	9	0.883	9	0.118	2.5	9	0.874	9	0.254
3.0	9	1.078	9	0.174	3.0	1	0.973	9	0.337
3.5	9	1.249	9	0.231	3.5	1	1.010	9	0.360
4.0	9	1.433	9	0.283	4.0	1	1.041	9	0.383
4.5	9	1.532	9	0.335	4.5	1	1.065	9	0.404
5.0	1	1.719	9	0.401	5.0	1	1.194	9	0.427

Direction y–y Table 22

Earthquake Petrovac 1979, N–S					Earthquake Vrancea 1977, N–S				
D	Floor	Δ_{max}	Floor	a_o (g)	D	Floor	Δ_{max}	Floor	a_o (g)
1.0	8	1.591	8	0.152	1.0	1,4,5	1.479	8	0.123
1.5	8	2.477	8	0.218	1.5	1	1.741	7	0.154
2.0	8	3.327	8	0.238	2.0	1	2.178	7	0.212
2.5	8	3.927	8	0.370	2.5	1	2.403	7	0.246
3.0	1	4.455	8	0.549	3.0	1	2.603	7	0.274
3.5	1	4.641	8	0.610	3.5	1	2.782	7	0.298
4.0	1	4.842	8	0.671	4.0	1	2.992	7	0.326
4.5	1	5.070	8	0.740	4.5	1	3.404	7	0.351
5.0	1	5.255	8	0.797	5.0	1	3.181	7	0.379

Maximum displacement at top of building for required ductility D = 2.5

Height of building H = 24.75 m

Table 23

Direction	x–x (longitudinal)		y–y (transverse)	
Earthquake	Petrovac N–S	Vrancea N–S	Petrovac N–S	Vrancea N–S
Acceleration of g	0.118	0.254	0.37	0.246
Maximum deformation Δ_{max} (cm)	2.905	3.165	16.58	14.27
Time of occurrence Δ_{max} (sec)	8.58	4.74	8.06	4.48
$\frac{H}{\Delta_{max}}$ relationship	853	782	149	173

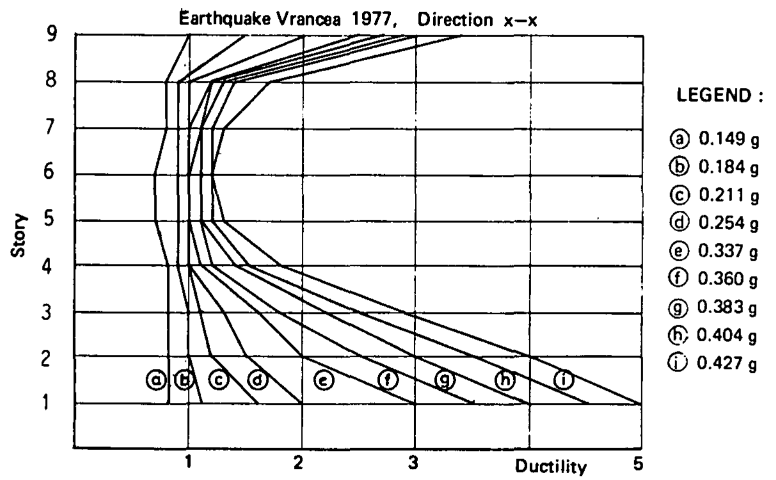

Earthquake Vrancea 1977, Direction x–x
Story
1
2
3
4
5
6
7
8
9
Ductility
1
2
3
5
a b c d e f g h i
LEGEND :
ⓐ 0.149 g
ⓑ 0.184 g
ⓒ 0.211 g
ⓓ 0.254 g
ⓔ 0.337 g
ⓕ 0.360 g
ⓖ 0.383 g
ⓗ 0.404 g
ⓘ 0.427 g

Fig. 7.

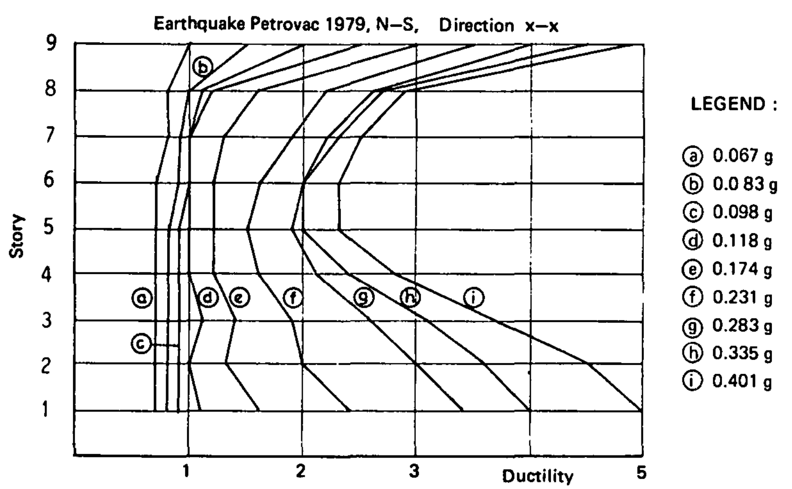
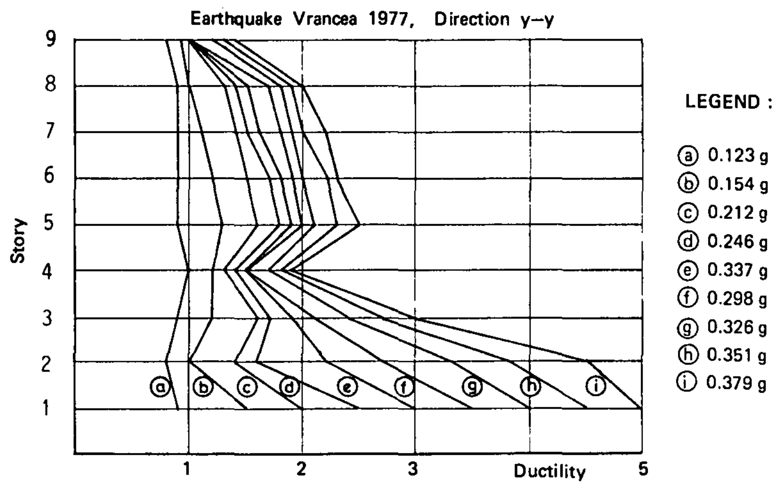
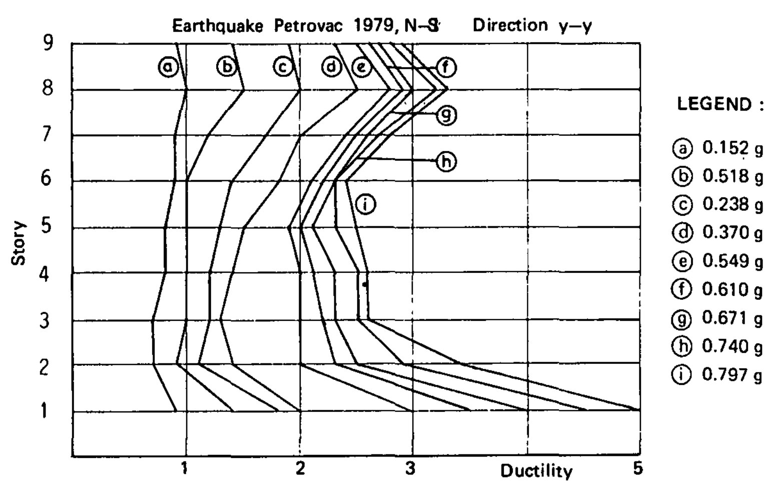

Earthquake Petrovac 1979, N–S, Direction x–x
Story
1
2
3
4
5
6
7
8
9
Ductility
1
2
3
5
a b c d e f g h i
LEGEND :
ⓐ 0.067 g
ⓑ 0.0 83 g
ⓒ 0.098 g
ⓓ 0.118 g
ⓔ 0.174 g
ⓕ 0.231 g
ⓖ 0.283 g
ⓗ 0.335 g
ⓘ 0.401 g

Fig. 8.



Earthquake Vrancea 1977, Direction y–y
Story
Ductility
LEGEND :
ⓐ 0.123 g
ⓑ 0.154 g
ⓒ 0.212 g
ⓓ 0.246 g
ⓔ 0.337 g
ⓕ 0.298 g
ⓖ 0.326 g
ⓗ 0.351 g
ⓘ 0.379 g

Fig. 9.



Earthquake Petrovac 1979, N–S Direction y–y
Story
Ductility
LEGEND :
ⓐ 0.152 g
ⓑ 0.518 g
ⓒ 0.238 g
ⓓ 0.370 g
ⓔ 0.549 g
ⓕ 0.610 g
ⓖ 0.671 g
ⓗ 0.740 g
ⓘ 0.797 g

Fig. 10.

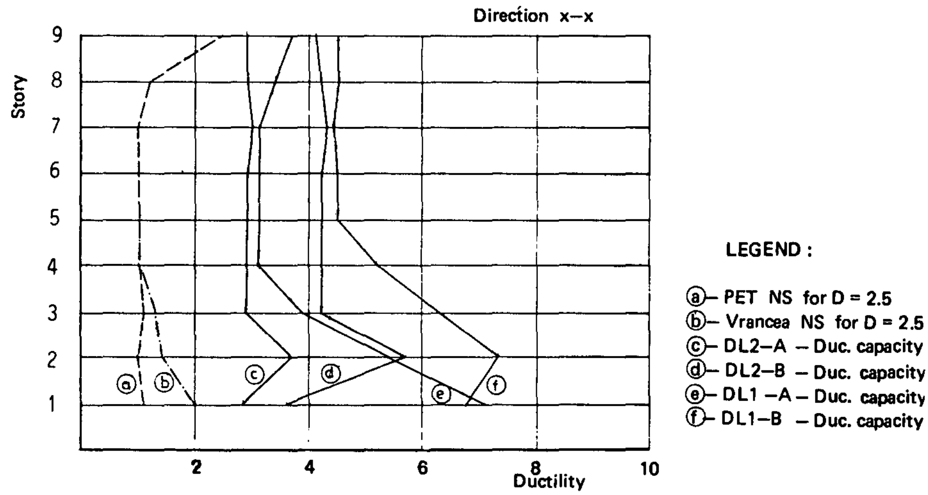

Direction x–x
Story
9
8
7
6
5
4
3
2
1
2
4
6
8
10
Ductility
a
b
c
d
e
f
LEGEND :
a– PET NS for D = 2.5
b– Vrancea NS for D = 2.5
c– DL2–A – Duc. capacity
d– DL2–B – Duc. capacity
e– DL1 –A – Duc. capacity
f– DL1–B – Duc. capacity

Fig. 11

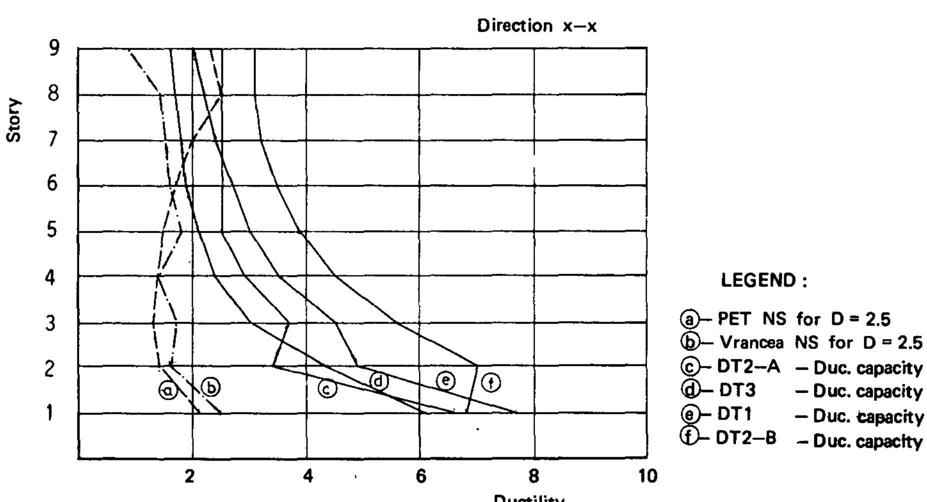

Direction x–x
Story
9
8
7
6
5
4
3
2
1
2
4
6
8
10
Ductility
a
b
c
d
e
f
LEGEND :
a– PET NS for D = 2.5
b– Vrancea NS for D = 2.5
c– DT2–A – Duc. capacity
d– DT3 – Duc. capacity
e– DT1 – Duc. capacity
f– DT2–B – Duc. capacity

Fig. 12

4. ANALYSIS OF THE YUGOSLAV DESIGN EXAMPLE

1. Structural System

The design example represented by Yugoslavia is an eight-story building constructed in a zone of high seismicity.

The structural system is a R.C. moment-resisting frame structure with a small number of structural walls in the central part of the building.

The built-in material is in compliance with the Yugoslav Code.

2. Structural Analysis

Prof. B. Simeonov analyzed the building applying two methods : the very simple Muto method and the more sophisticated static analysis by the TABS computer programme. Presented on Page 16* and Figs. 4 and 5c* are the analyses of the obtained results with estimation of the differences observed in the static values, which are rather high, and therefore a more accurate method such as application of the TABS programme is recommended.(*see design example)

A shear base coefficient of 0.10 has been accepted.

Tables 24 through 26 show the results obtained by our additional performed analysis, between the Muto and the TABS method, which was applied to some structural elements, i.e., column S1 of R6, column S2 of R6 and the structural wall in R6. The obtained results are in good correlation with those obtained by Prof. B. Simeonov.

The global shear base coefficients were found to be :

in x-x direction : $Q^y = 11029$ KN ; $C_B = 0.25$

in y-y direction : $Q^y = 9004$ KN ; $C_B = 0.206$

3. Dynamic Response Analysis

3.1. Seismic Parameters

The design example, shown on pages 5 and 6 in Item 2.2.2, presents the results obtained by the nonlinear dynamic analysis carried out by Prof. B. Simeonov of the selected El Centro and Parkfield earthquakes and for a ground acceleration of 0.23 g, which represents a design earthquake for the site of the considered building.

As in the previous two examples, in our case, the following seismic effects have been applied :

- Vrancea Earthquake, 7 March 1977, N-S component
- Montenegro Earthquake, Petrovac record, 15 April 1979, N-S component

The purpose of the analysis is to determine the intensity level which induces in the structure ductilities of 1, 1.5, 2, 2.5, 3, 3.5, 4, 4.5 and 5.

3.2. Mathematical Modelling of Structure

Considering the fact that in addition to the two simplified methods the design example has been also analyzed by using the TABS computer programme, the results from the latter more accurate analysis have been applied in our example, as well.

Presented in Tables 24 through 26 are data on the reinforcement in the two characteristic columns and the structural wall.

For these structural elements, proportioned by the special computer programme, the ultimate stress states and deformations have been determined for the structural elements of each floor, separately, and thus the P-Δ bilinear positive diagrams have been obtained.

These diagrams represent the story strength capacity and deformability of the mathematical model of the structure in x-x and y-y direction.

3.3. Dynamic Analysis Results

Applying the same VIBR 4 computer programme dynamic response analysis has been carried out for the prepared input data. Tables 27 and 28 and Figures 13 through 16 show the results obtained by the dynamic response analysis. Based on the results it can be concluded that :

- In x-x direction, for elastic behaviour of the structure, i.e., D = 1.0, the ground acceleration is 0.067 g for the Vrancea earthquake and 0.039 g for the Petrovac record, N-S component of the Montenegro earthquake. In y-y direction, for D = 1.0, the ground acceleration is 0.076 g for the Vrancea earthquake and 0.139 g for the Petrovac record, N-S component of the Montenegro earthquake.
- For the maximum ductility of D = 5, the ground acceleration in x-x direction is 0.312 g for the Vrancea earthquake and 0.275 g for the Petrovac record, N-S component of the Montenegro earthquake.
  In y-y direction the ground acceleration is 0.395 g for the Vrancea earthquake and 0.762 g for the Petrovac record, N-S component of the Montenegro earthquake.

- Considering the applied structural system and the built-in material the required ductility for design purposes should range between 2.5 and 3.5, depending upon the inter-story drift, which according to the Yugoslav Code is :

- For elastic behaviour D = 1 ; $\Delta_i \leq \frac{h_i}{350}$

- for nonlinear behaviour (level I) $\Delta_i \leq \frac{h_i}{150}$

(h_i - story height ; Δ_i - interstory drift)

If the required ductility is taken to be D = 3, the ground acceleration for the Vrancea earthquake in x-x direction is 0.17 g and for the Petrovac record, N-S component it is 0.131 g. In y-y direction, for ductility D = 3 the ground acceleration is 0.249 g for the Vrancea earthquake and 0.305 g for the Petrovac record, N-S component of the Montenegro earthquake.

General Conclusions

1. The structure has been designed with sufficient strength capacity.
2. The stiffness of the structure differs in longitudinal and transverse direction due to the structural walls in x-x direction,therefore, the difference in stiffness is around 1.3 to 3.0 times.
3. If an approximative method of static analysis is applied (Muto's method), force redistribution over the structural elements should be carried out.
4. For a moderate required ductility factor D = 3, the maximum displacement at the top of the building is within the allowable ones (Table 29).
5. For a moderate required ductility D = 3.0, (which corresponds to the average design earthquake conditions) the structural elements have the required ductility capacity (D_{cap} > 3.0). (Figs. 17 and 18).

It is also possible to analyze the obtained results with respect to the site seismicity and to consider the level of vulnerability and reliability of the structure as well as the applied design Code.

Table 24

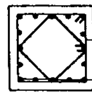
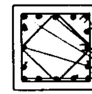
Wall	Storey	By TABS method			By simple method			By ultimate state							By simple method		
								 ϕ6/15/7.5, 5ϕ12							 ϕ6/15/7.5, 2×8ϕ16(8ϕ12)		
		M_E (kN·m)	Q_g+E (kN)	N_g+E (kN)	M (kN·m)	Q (kN)	N (kN)	Required reinforcem. (cm²)	Required web reinforce. (cm²/m')	Accepted tensile reinforcement n ϕ	Accepted tensile reinforcement (cm²)	Accepted compressed reinforcement n ϕ	Accepted compressed reinforcement (cm²)	Accepted web reinforcem.	n ϕ	Fa = Fa' (cm²)	web Fa (nϕ/m')
R6-S1	1	30	29	443	17			4.8	0.0	5ϕ12	5.66	5ϕ12	5.66	ϕ6/15/7⁵	5ϕ12	5.66	ϕ6/15/7⁵
	2	31	36	394	21			"	"	"	"	"	"	"	"	"	"
	3	37	37	341	28			"	"	"	"	"	"	"	"	"	"
	4	37	38	307	32			"	"	"	"	"	"	"	"	"	"
	5	35	36	260	35			"	"	"	"	"	"	"	"	"	"
	6	30	34	198	43			"	"	"	"	"	"	"	"	"	"
	7	29	33	133	32			"	"	"	"	"	"	"	"	"	"
	8	28	37	65	25			"	"	"	"	"	"	"	5ϕ16	10.05	"

Table 25

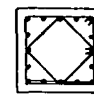
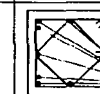
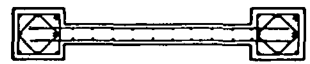
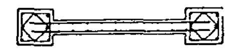
Wall	Storey	By TABS method			By simple method			By ultimate state							By simple method		
																	
		M_E (kN·m)	Q_g+E (kN)	N_g+E (kN)	M (kN·m)	Q (kN)	N (kN)	Required reinforcem. (cm^2)	Required web reinforce. (cm^2/m')	Accepted tensile reinforcement n ϕ	Accepted tensile reinforcement (cm^2)	Accepted compressed reinforcement n ϕ	Accepted compressed reinforcement (cm^2)	Accepted web reinforcem.	n ϕ	Fa = Fa' (cm^2)	web Fa (nϕ/m')
R6-S2	1	73	66	1044	44			7.5	0.0	5ϕ14	7.7	5ϕ14	7.7	ϕ6/15/7[5]	5ϕ18	12.72	ϕ6/15/7[5]
	2	82	90	865	63			"	"	"	"	"	"	"	"	"	"
	3	100	96	722	89			"	"	"	"	"	"	"	"	"	"
	4	116	114	589	97			"	"	"	"	"	"	"	"	"	"
	5	72	69	460	76			"	"	"	"	"	"	"	5ϕ22	19.01	"
	6	83	75	334	96			"	"	"	"	"	"	"	"	"	"
	7	75	67	209	74			"	"	"	"	"	"	"	"	"	"
	8	80	78	84	65			"	"	"	"	"	"	"	7ϕ25	34.36	"

Table 26

Wall	Storey	By TABS method			By simple method			By ultimate state 							By simple method 		
		M_E (kN·m)	Q_g+E (kN)	N_g+E (kN)	M (kN·m)	Q (kN)	N (kN)	Required reinforcem. (cm^2)	Required web reinforce. (cm^2/m')	Accepted tensile reinforcement		Accepted compressed reinforcement		Accepted web reinforcem.	n ϕ	Fa = Fa' (cm^2)	Fa web (nϕ/m')
										n ϕ	(cm^2)	n ϕ	(cm^2)				
R6	1	20815	2007	2859.6	25220			56.12	6.34	12ø25	58.9	12ø25	58.9	ø10/20	24ø25	117.84	8ø8/m'
	2	14485	1653	2520	18640			17.4	5.42	6ø20	18.85	6ø20	18.85	"	12ø25	58.90	"
	3	9300	1414	2178	12665			"	4.0	"	"	"	"	ø8/20	8ø20	25.13	8ø6/m'
	4	5500	1010	1834	7885			"	"	"	"	"	"	"	"	"	"
	5	2970	1011	1487	4350			"	"	"	"	"	"	"	"	"	"
	6	520	640	1157	1545			"	"	"	"	"	"	"	"	"	"
	7	815	293	784	205			"	"	"	"	"	"	"	"	"	"
	8	1125	246	428	725			"	"	"	"	"	"	"	"	"	"

Direction x–x Table 27

Earthquake Petrovac 1979, N–S					Earthquake Vrancea 1977, N–S				
D	Floor	Δ_{max}	Floor	a_o (g)	D	Floor	Δ_{max}	Floor	a_o (g)
1.0	5	0.380	5	0.039	1.0	5	0.371	5	0.067
1.5	5	0.553	5	0.053	1.5	5	0.548	5	0.087
2.0	5	0.756	5	0.070	2.0	5	0.742	5	0.108
2.5	5	0.916	5	0.096	2.5	5	0.919	5	0.130
3.0	5	1.100	5	0.131	3.0	5	1.100	5	0.170
3.5	5	1.291	5	0.168	3.5	5	1.283	5	0.211
4.0	5	1.463	5	0.200	4.0	5	1.504	5	0.254
4.5	5	1.665	5	0.240	4.5	5	1.671	5	0.281
5.0	5	1.835	5	0.275	5.0	5	1.859	5	0.312

Direction y–y Table 28

Earthquake Petrovac 1979, N–S					Earthquake Vrancea 1977, N–S				
D	Floor	Δ_{max}	Floor	a_o (g)	D	Floor	Δ_{max}	Floor	a_o (g)
1.0	3,5	0.792	5	0.139	1	3	0.676	5	0.0763
1.5	5	1.107	5	0.168	1.5	3,5	1.093	5	0.105
2.0	5	1.498	5	0.196	2.0	3,5	1.510	5	0.175
2.5	5	1.852	5	0.231	2.5	3	1.774	3	0.219
3.0	5	2.178	5	0.305	3.0	3	2.034	3	0.249
3.5	3,5	2.583	5	0.501	3.5	3	2.452	3	0.295
4.0	3	2.807	3	0.597	4.0	3	2.787	5	0.330
4.5	5	3.247	5	0.697	4.5	3	3.226	6	0.365
5.0	5	3.629	5	0.762	5.0	5	3.612	5	0.395

Maximum displacement at top of building for required ductility D = 3.0

Height of building H = 23.12 m Table 29

Direction	x–x (longitudinal)		y–y (transverse)	
Earthquake	Petrovac N–S	Vrancea N–S	Petrovac N–S	Vrancea N–S
Acceleration of g	0.131	0.17	0.305	0.249
Maximum deformation Δ_{max} (cm)	4.514	4.70	9.69	9.03
Time of occurrence Δ_{max} (sec)	7.98	5.08	8.04	3.91
$\frac{H}{\Delta_{max}}$ relationship	513	492	239	256

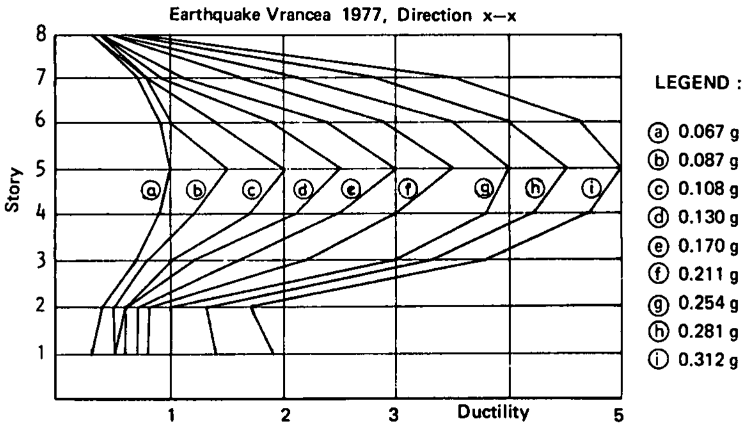

Earthquake Vrancea 1977, Direction x–x
Story
8
7
6
5
4
3
2
1
a b c d e f g h i
1
2
3
Ductility
5
LEGEND :
a 0.067 g
b 0.087 g
c 0.108 g
d 0.130 g
e 0.170 g
f 0.211 g
g 0.254 g
h 0.281 g
i 0.312 g

Fig. 13.

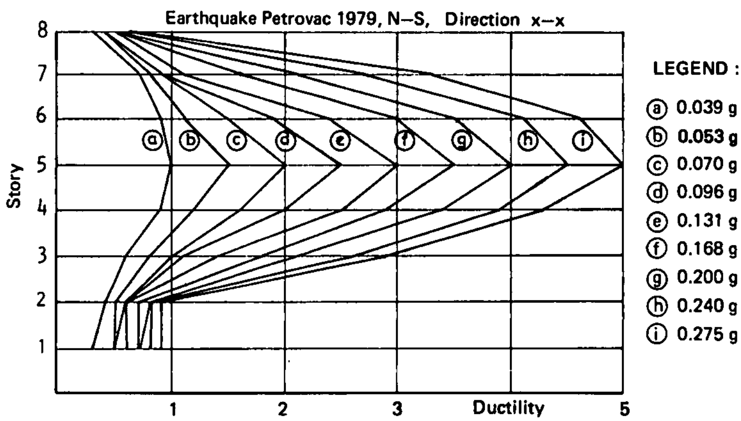

Earthquake Petrovac 1979, N–S, Direction x–x
Story
8
7
6
5
4
3
2
1
a b c d e f g h i
1
2
3
Ductility
5
LEGEND :
a 0.039 g
b 0.053 g
c 0.070 g
d 0.096 g
e 0.131 g
f 0.168 g
g 0.200 g
h 0.240 g
i 0.275 g

Fig. 14.

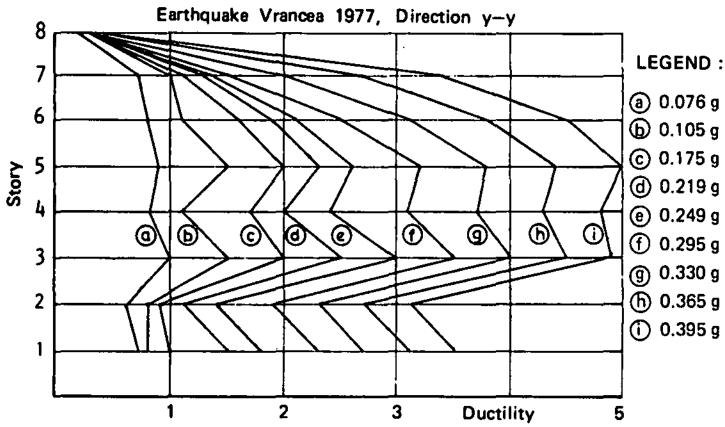

Earthquake Vrancea 1977, Direction y–y
Story
8
7
6
5
4
3
2
1
a
b
c
d
e
f
g
h
i
1
2
3
Ductility
5
LEGEND :
a 0.076 g
b 0.105 g
c 0.175 g
d 0.219 g
e 0.249 g
f 0.295 g
g 0.330 g
h 0.365 g
i 0.395 g

Fig. 15.

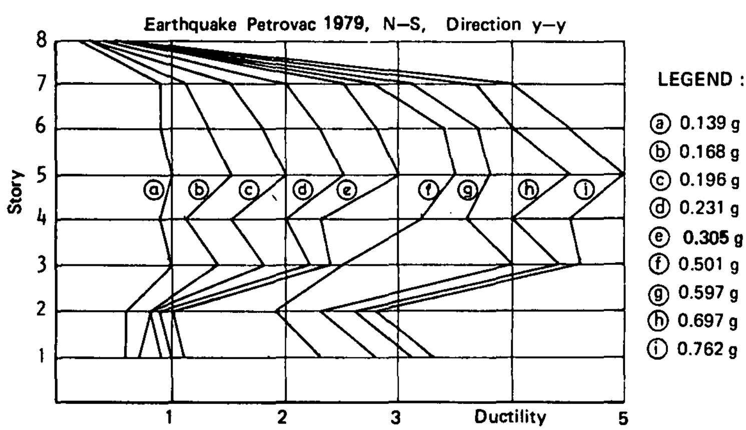

Earthquake Petrovac 1979, N–S, Direction y–y
Story
8
7
6
5
4
3
2
1
a
b
c
d
e
f
g
h
i
1
2
3
Ductility
5
LEGEND :
a 0.139 g
b 0.168 g
c 0.196 g
d 0.231 g
e 0.305 g
f 0.501 g
g 0.597 g
h 0.697 g
i 0.762 g

Fig. 16.

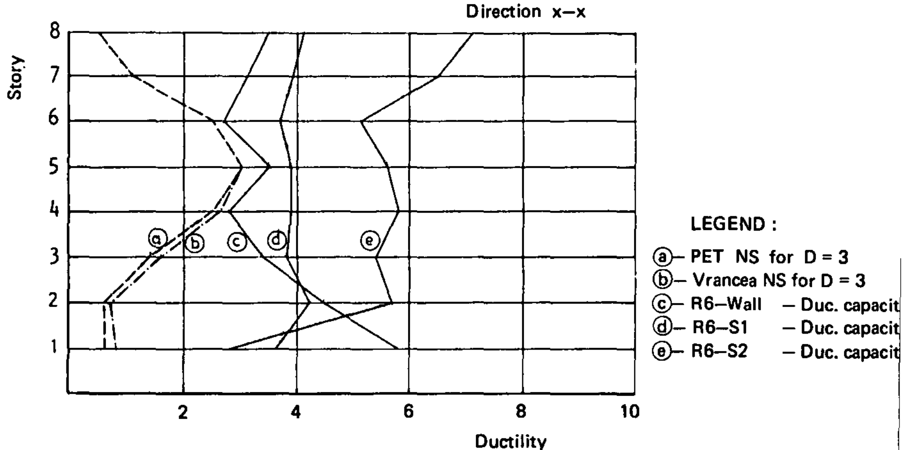

Direction x–x
Story
8
7
6
5
4
3
2
1
2
4
6
8
10
Ductility
a
b
c
d
e
LEGEND :
a– PET NS for D = 3
b– Vrancea NS for D = 3
c– R6–Wall – Duc. capacit
d– R6–S1 – Duc. capacit
e– R6–S2 – Duc. capacit

Fig. 17

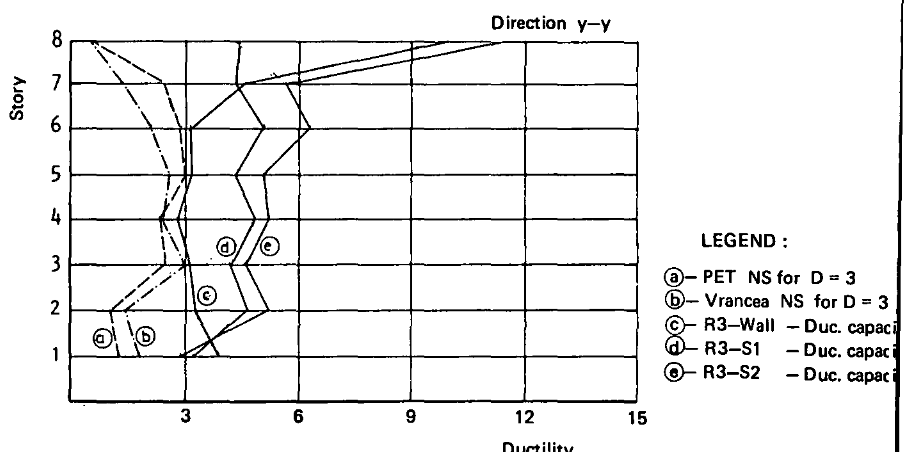

Direction y–y
Story
8
7
6
5
4
3
2
1
3
6
9
12
15
Ductility
a
b
c
d
e
LEGEND :
a– PET NS for D = 3
b– Vrancea NS for D = 3
c– R3–Wall – Duc. capaci
d– R3–S1 – Duc. capaci
e– R3–S2 – Duc. capaci

Fig. 18