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BUILDING CONSTRUCTION
UNDER SEISMIC CONDITIONS
IN THE BALKAN REGION

VOLUME 2

DESIGN AND CONSTRUCTION OF PREFABRICATED REINFORCED CONCRETE BUILDING SYSTEMS

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UNDER SEISMIC CONDITIONS
IN THE BALKAN REGION

UNDP/UNIDO PROJECT RER/79/015

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DESIGN AND CONSTRUCTION OF PREFABRICATED REINFORCED CONCRETE BUILDING SYSTEMS

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION
executing agency for the
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VOLUME 2

DESIGN AND CONSTRUCTION OF PREFABRICATED REINFORCED CONCRETE BUILDING SYSTEMS

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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

The financial support of the United Nations and the Governments of Bulgaria, Greece, Hungary, Rumania, Turkey and Yugoslavia should hereby be acknowledged. Also the cooperation of the National Science Foundation of the USA is noted with appreciation.

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 earth-quake engineering - particularly, objective evaluations of causes and effec in earthquake damage - and in the seismic and geologic characteristics of tl physical environment.

The material reflects the interpretation and opinions of the authors and coi tributors and does not necessarily represent the viewpoint of the United Na- tions Industrial Development Organization, the United Nations Development Programme, the participating Governments and the National Science Foundatioı of the USA. The above mentioned Governments and Organizations - while providing for the presentation of these Volumes in the public interest and for their obvious informational value - assumes no responsibility for any views expressed therein.

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NOTE

Earthquakes in the Balkan region have caused considerable loss of life and property damages. Improved design and construction procedures, proper detailing and strict quality control may lead to improved structural behavior under seismic actions and thus reduce potential losses.

The Manual reflects the experience in the Balkan region in design and construction of earthquake-resistant prefabricated reinforced concrete buildings and is intended to serve as an aid to practicing engineers. In development of this document, analytical and experimental research data as well as recommendations contained in the several national seismic design codes have been taken into account. In addition, economic, social and technological aspects of the region have been considered. The national codes for earthquake resistant design together with the pertinent recommendations of the CEB - Comite Euro-International du Beton - are presented in Manual VII.

The recommendations presented in this manual are principally intended for new structures. However, the basic principles reflected in this manual also offer a guide for strengthening procedures of existing buildings. In connection herewith, it be noted that strengthening procedures for reinforced concrete structures in general are presented in Manual V.

The following contents are devided into two major parts, namely:

Part 1 -- General earthquake resistant design principles for prefabricated reinforced concrete building systems, and

Part 2 -- Design examples of seismic-resistant prefabricated reinforced concrete building systems typical for the Balkan region.

Part 1 presents a summary of general considerations for the design of seismic-resistant prefabricated reinforced concrete building systems. In this part, both technical aspects of the several systems as well as pertinent structural analysis procedures are presented. Detailed information on the seismic behavior of prefabricated members and their connections is also included. Design objectives and criteria as well as detailing principles are reviewed.

Part 2 contains ten design examples of prefabricated reinforced concrete building systems reflecting the state of practice in the Balkan region. Examples of large-panel, package lift-slab and frame-panel systems as well as mixed (monolithic and prefabricated) panel systems are presented. The design of these systems has been based on pertinent national building and seismic codes and have been prepared by delegates from Bulgaria, Greece, Hungary, Rumania, Turkey and Yugoslavia.

The Working Group consisted of National delegates of the participating countries with Dr. Simeon Simeonov, Professor, Higher Institute of Architecture and Civil Engineering, Sofia, Bulgaria, serving as Convenor. Other members were Dr. Peter Sotirov, Higher Institute of Architecture and Civil Engineering, Sofia, Bulgaria; Dr. S.G. Tsoukantas, National Technical University of Athens, Athens, Greece; Dr. Bela Goschy, Consultant, Budapest, Hungary; Dr. Dan Constantinescu, Associate Professor, Faculty of Civil Engineering, University of Bucharest, Bucharest, Rumania; Dr. K. Özden, Professor, Technical University, Istanbul, Turkey and Dr. Drazen Anicic, Structural Research Institute of Kroatia, IGH, Zagreb, Yugoslavia.

Consultant of the Working Group was Dr. Robert Park, Professor, University of Canterbury, Christ-Church, New Zealand.

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

1. INTRODUCTION

1.1 Scope

Part I of this Manual covers earthquake resistant design philosophy and general design principles for prefabricated reinforced concrete building systems.

1.2 General

In Europe after the second world war there was a major demand for the construction of large scale buildings. This demand could not be met by the traditional methods of building construction. As a result, systems of building making use of the advantages of prefabrication were developed and extensive construction of prefabricated reinforced concrete buildings occurred. The massive building program achieved in Europe since the second world war could not have been undertaken without the application of prefabricated concrete construction. The mass production of prefabricated concrete components now ranges from linear members to complete rooms formed from box units. The design and construction of prefabricated concrete buildings has been made possible by developments in building technology, including improvements in transportation and erection equipment. The lack of detailed building codes for prefabricated concrete construction, which is still the case in many countries, has meant that innovation has been necessary. Investigations, both theoretical and experimental, have been conducted with special attention given to the performance of connections between prefabricated components. There are a number of potential problems with prefabricated concrete building systems and these have had to be appreciated and overcome as far as possible. Some problems require further studies in the future.

The major difference between traditional monolithic cast in situ reinforced concrete building structures and prefabricated concrete structures is that prefabricated structures are composed of various members cast in a different place of origin than their final position in the structure. The prefabricated elements are interconnected by looped, lapped, welded or mechanically connected reinforcing bars, or by bars welded or bolted to anchored steel shapes, in the joints. Cast in situ concrete or mortar may or may not be present in the joint region. The elements and connections then constitute the structure. The structural configurations for resisting seismic loads are either structural walls, or frames formed from beams and columns, or dual systems consisting of interconnected structural walls and frames.

The term prefabrication is generally used to denote industrialized casting of concrete elements in a specialised plant. Prefabrication of building systems implies a large initial investment in steel forms, concrete manufacturing equipment, curing technology, means of transportation, and erection equipment. Hence prefabrication is only fully justified when the number of identical or similar structural members to be made is large enough to justify this investment. The successful construction of prefabricated concrete building systems represents the results of careful planning of structural types, factory manufacture of elements, erection and jointing technology, and installation of services.

The overall geometry of prefabricated buildings is modulated both horizontally and vertically. Very careful attention to the pattern of element subdivision and the method of assembling the elements of a prefabricated structure is of crucial importance for their efficient application.
A compromise must be made between the tendency to diversify the basic

parameters (that is, spans, storey heights, allowable live loads, and architectural style) and the need to reduce the number of types of prefabricated elements in order to increase the efficiency of prefabrication.

The main advantages of prefabricated building systems can be summarized as follows:

- The centralization of the main part of the construction process into plants (factories) and the consequential reduction of labour consuming work at the construction site.

- The improvement of the element quality due to the high quality control possible in industrialized (factory) production.

- The use of higher strength concrete due to better conditions of concrete batching and continuous quality control.

- The increase in productivity of the manpower due to well planned industrialized repetitive processes and better working (factory) conditions, and the reduction in inefficient manual effort at the construction site.

- The increase in the effectiveness in the use of the construction equipment due to multiple use.

- The shortening of the time of construction.

- The reduction of influence of season and climate on erection. Erection during winter is possible.

- The reduction in the amount of timber needed for formwork.

- The conversion of building systems into system buildings based on a goal orientated system approach.

Due care should be taken of the possible problems arising from prefabricated construction, which are:

- Proper joints between prefabricated concrete elements cannot be constructed if there are significant deviations from the specified sizes of elements. Hence the allowable tolerances in member sizes as produced should be specified and achieved using steel forms and well controlled factory conditions.

- The effects of differential shrinkage of concrete of different ages and composition between the various parts of the structure need to be controlled by adequate reinforcing details.

- Uncertainties exist which may change the forces between the concrete elements at the connections. For example, deviations during construction may affect the geometry of the structure, the distribution of the forces to be transferred cannot be determined exactly in some types of connections, and the deformations which occur during the life of the structure may alter the position, direction and intensity of the reactions between elements.

- The sealing compounds used for weatherproofing should perform satisfactorily during the life of the structure.

- Architects may prefer more freedom to vary the style of building than is available from most prefabricated building systems.

- The basic problem in the design of earthquake resistant prefabricated concrete buildings is in finding an economical and practical method for connecting the prefabricated elements together which provide a satisfactory structural solution. The special problems of earthquake resistant design are discussed in the next section.

1.3 Special Requirements of Earthquake Resistant Design

Dynamic analyses of structures responding in the elastic range to ground motions recorded during severe earthquakes have shown that the theoretical response inertia loads induced in the structure are generally significantly greater than the equivalent static horizontal design loads recommended by most codes. It has been demonstrated that structures designed for the horizontal loads normally recommended by codes can only survive strong ground shaking if they have sufficient ability to dissipate seismic energy. This energy dissipation is provided mainly by inelastic deformations at critical regions in the structural system, helped by increased viscous damping and by energy dissipation from soil-structure interaction at large deflections of the structure. The energy dissipated by inelastic deformations in the structural system requires the elements of the structure and their connections to possess adequate "ductility". The term "ductility" in earthquake resistant design is used as an abbreviation for "the ability to dissipate a significant amount of energy through inelastic behaviour under large amplitude cyclic deformations without substantial reduction of strength".

Note that there is a major difference between design for just gravity loads and design for both gravity and seismic loads. Design for just gravity loads requires consideration only of the effect of monotonically applied loads. Design for both gravity and seismic loads requires consideration of the effect of cyclic loading and of the resulting effects of large cyclic inelastic deformations which can result in a significant degradation of the strength, stiffness and energy dissipation characteristics of the structural system.

Hence the additional factors which need to be considered in the seismic design of prefabricated concrete structural systems are:

- The best means for achieving ductility in the system is sought. A well diffused number of yield zones is normally the most satisfactory way of assuring protection against collapse. Reliability against other (undesirable) modes of yielding is obtained by amplifying the design actions, associated with those other modes so that they are unlikely to occur. In the case of structural wall systems yielding of coupling beams (lintels) between walls is recognised as an excellent manner in which to dissipate energy. In frame systems plastic hinges should normally form in the beams, rather than in the columns.

- The joints are undoubtedly the region of greatest seismic design difficulty. In moderate earthquakes the displacement of prefabricated concrete systems may be greater than that of monolithic systems with similar geometry and identical structural patterns due to the reduction in stiffness of the joints. Hence particular attention should be given to achieiving adequate stiffness of the joints. In addition, it is most important that the joints have the necessary strength and ductility to enable the structure to survive severe earthquakes. In general, the degree of participation of the joints between the prefabricated concrete elements in the energy dissipating mechanism during severe earthquakes should be limited. The ideal is to avoid yielding in those joint regions which have a potential weakness, by forcing the yielding to occur only within the joints which are ductile and/or within the prefabricated elements.

- The arrangement of the horizontal load resisting elements in a building should be as symmetrical as possible in order to minimize the torsional response of the building during earthquakes. Unsymmetrical structural arrangements, for example, walls enclosing a service core at one end of the building only which is structurally connected to the remainder of the building, can result in significant twisting about the vertical axis of the building and hence lead to greater ductility demand on some parts of the structure than for symmetrical arrangements. Such twisting may become critical for the overall stability of the building. Furthermore, due to numerous uncertainties, the actual behaviour of an unsymmetrical building is difficult to predict, even with elaborate computer models. Fortunately, simple geometry leading to a symmetrical arrangement of horizontal load resisting elements is normally a requirement for efficient prefabricated building systems. Note that the participation of non-structural elements in the response of the structure may result in unexpected and undesirable torsional effects. The designer should endeavour to anticipate the influence of non-structural elements on the response of the structure.

- It is undesirable for discontinuities in stiffness and/or strength of the structural system to exist up the height of the building. For example, the absence of some vertical structural elements in one storey of a building can lead to a dangerous concentration of ductility demand in the remaining elements of the storey. Similarly, sudden variations in building plan dimensions up the height of the building can result in equally dangerous large local deformations.

It is evident, however, that the above special requirements for earthquake resistant design can be achieved satisfactorily by good design. In the 1977 Bucharest, Romania, earthquake a range of prefabricated concrete structures was observed to behave well. In the 1979 Montenegro, Yugoslavia, earthquake large panel systems were shown to have good features. Nevertheless, data on behaviour is not available for an extensive range of severe earthquakes. Large panel structures behaved extremely well in the Bucharest earthquake but the predominance of the relatively long period ground motions in the accelerogram of that earthquake, compared with the smaller fundamental period of vibration of large panel buildings, does mean that in a "more normal" severe earthquake the response of the buildings would have been greater and may have resulted in more significant damage. Some mechanisms for dissipating seismic energy are more effective and less damaging to the structure than others, as will be discussed in subsequent sections. Also, some large panel systems forming a stiff box structure have been observed to dissipate significant energy at foundation level as a result of inelastic deformations of the soil. It is evident that the seismic design of safe and stable prefabricated concrete buildings needs to be based on a careful assessment of the behaviour of the individual elements, the joints, and the structure as a whole.

National codes for the design of earthquake resistant prefabricated concrete structures have been slower in development than have codes for cast in situ (monolithic) concrete structures. Indeed, some codes imply that prefabricated concrete structures must comply with the code provisions pertaining to cast in situ structures. There are obvious difficulties in this approach and codes should cover the special features of prefabricated structures. Some design procedures at present used for proprietry building systems have been based on test results which have proved that particular system. A code for prefabricated concrete structures needs to cover the design of a wide range of possible types of connections between elements, and the design provisions should take into account the degree of participation of the connections in the energy dissipating mechanism. The design provisions should also be closely linked with the method of construction of the structure. It is to be noted that the recently published Appendix to the CEB-FIP Code: "Seismic Design of Concrete Structures" [1] is for monolithic concrete structures and that Appendix states that "For structures having special characteristics, from the point of view of their function or structural type (example ..... structures made of precast elements not monolithically connected ....) this standard must be complemented with ad hoc requirements and provisions".

The aim of this Manual is to bring to designers some of the available international experience, mainly from the Balkan countries, of design, construction, and background developments for earthquake resistant prefabricated reinforced concrete building systems.

2. BASIC PREFABRICATED STRUCTURAL CONCRETE SYSTEMS

2.1 General

Various structural systems incorporating prefabricated reinforced concrete elements have been devised. The predominance of the type of system varies from country to country. The diversity reflects differences in building function, variations in the economy of different types of prefabricated elements, and the design philosophy and ingenuity of the individual designers. The basic prefabricated structural systems can be identified as:

- Large panel systems
- Frame systems
- Slab-column systems with walls
- Mixed systems

The structural system for a particular building can consist of one basic system or of combinations of the basic systems. The system can also be composed either wholly of prefabricated elements, or of prefabricated elements in combination with cast in situ concrete.

A further application of prefabrication is in the production of a range of non-structural elements for use with in situ or prefabricated concrete structural systems.

The configurations of the basic prefabricated structural systems, and the use of non-structural prefabricated elements are discussed below.

2.2 Large Panel Systems

2.2.1 General

Prefabricated large panel systems are used mainly for residential buildings. The designation "large panel system" is applied to multistorey structures composed of large concrete panels which are connected in the vertical and horizontal directions so that the wall panels enclose appropriate size spaces for the rooms of the building. The panels form the structural system. Prefabricated wall panels are usually one storey in height and in general both horizontal and vertical joints exist between the panels. The horizontal floor and roof panels usually consist either of one-way spanning prefabricated slab elements or of two-way spanning elements of the size of the relevant room. When properly connected together the horizontal elements act as diaphragms, transferring the earthquake loads to the walls, in addition to resisting the gravity loads.

2.2.2 Basic Configurations

Three basic configurations are used for large panel buildings:

(a) Cross-Wall System. The walls bearing gravity loads in the cross-wall system are placed perpendicular to the longitudinal axis of the building (see Fig. 1a). These cross-walls provide resistance to horizontal seismic loads in their direction and support the gravity loads from one-way spanning floor or roof elements. Walls not bearing gravity loads are placed parallel to the longitudinal axes of the building to provide resistance to horizontal seismic loads in that direction.

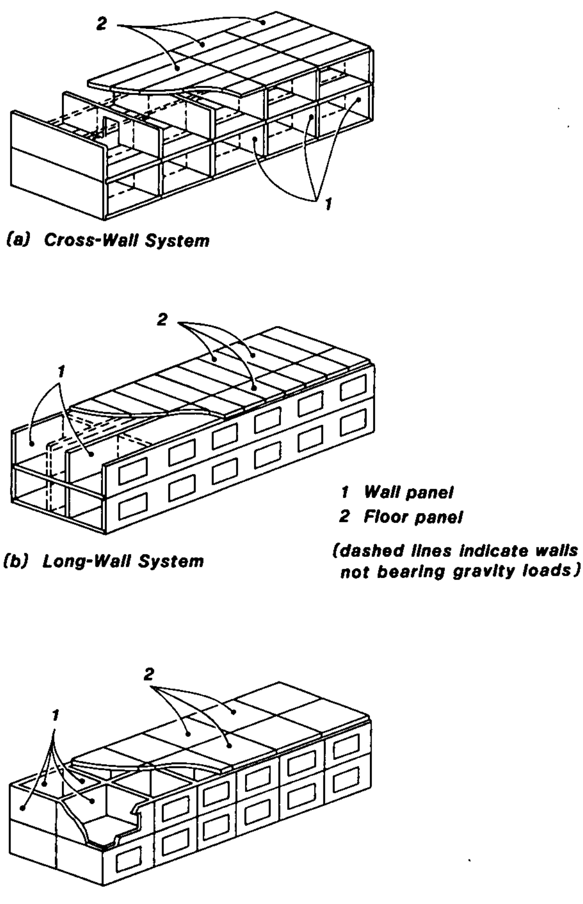


Fig. 1 Basic Structural Configurations of Large Panel Systems

(b) Long-Wall System. The walls bearing gravity loads in the long-wall system are placed parallel to the longitudinal axis of the building (see Fig. 1b). The long-walls provide resistance to horizontal seismic loads in their direction and support the gravity loads from one-way spanning floor or roof elements. Walls not bearing gravity loads are placed perpendicular to the longitudinal axis of the building to provide resistance to horizontal seismic loads in that direction.

(c) Two-Way System. The walls bearing gravity loads in the two-way system are placed both perpendicular and parallel to the longitudinal axis of the building (see Fig. 1c). These walls provide resistance to horizontal seismic loads in both directions and support the gravity loads from two-way spanning floor or roof elements.

Closely related to the large panel system in final form after erection is the box or cellular system. The concrete boxes may be cast as integral units or individual components assembled with connections to provide integral behaviour. In this system room size prefabricated box units are stacked one on top of the other. The walls provide resistance to horizontal seismic loads in both directions and bear the gravity loads. Alternatively, the boxes can be arranged to form moment resisting frames in one direction and to act as structural walls in the other direction.

2.2.3 Connections

Depending on the direction of the joint, two main types of connections may be identified:

- "Vertical joints", which connect the vertical edges of adjoining wall panels and primarily resist vertical shear force due to seismic loading.
- "Horizontal joints", which connect the horizontal edges of adjoining wall and floor panels and primarily resist vertical normal forces due to gravity loads from the upper panels and floors, horizontal shear force due to seismic loads, and bending moments in two directions due to seismic loading acting on the upper panels and gravity loading acting on the adjoining floor panels.

A wide range of details for joints are possible. In general the joint may be either "wet" or "dry". Wet joints are constructed with cast in situ concrete in the joint regions between prefabricated panels. If structural continuity is required through the joint, protruding reinforcing bars from the panels are welded, looped or otherwise connected in the joint region before the cast in situ concrete is placed. Dry joints are constructed by welding or bolting together steel plates or other steel inserts which have been cast into the ends of the prefabricated panels for this purpose. In dry joints the actions between the panels are transferred at discrete points at the panel edges where the steel inserts are connected, and hence stress concentrations occur. Wet joints result in a structure more closely approaching monolithic construction, but dry joints result in speedier erection.

2.3 Frame Systems

2.3.1 General

Prefabricated multistorey frames are used for both residential and industrial buildings. They have been more frequently used for industrial buildings than for residential buildings, because fewer partition walls are required in industrial buildings. In those positions where partition walls are needed, they can be appropriately separated and detailed so as not to interfere significantly with the deformability of the frame during an earthquake. Alternatively, some or all of the partition walls can be designed as structural walls, either prefabricated or cast in situ, to resist horizontal seismic loading. Well placed structural walls in a building, because of their greater stiffness, can be used to resist the greater part of the horizontal seismic load and to limit the horizontal deflections, leaving the more flexible frames to carry the majority of the gravity loading. Thus combinations of frames and structural walls can often be used to advantage and indeed are necessary if the beams and columns are not rigidly connected together.

Prefabricated frames can be constructed of linear elements or of spatial beam-column subassemblages comprised of linear elements. Prefabricated beam-column subassemblages have the advantage that the connecting faces between the subassemblages can be placed away from the critical regions of the frames. For example, the connecting faces can be placed at the mid-height of storeys and within the spans of the beams, away from the regions of maximum moment caused by earthquake loading. However, linear elements are generally preferred because of the difficulties associated with forming, handling, and erecting spatial elements. The use of linear elements generally means placing the connecting faces at the beam-column junctions. That is, the beams are generally prefabricated in lengths to occupy the clear spans between the columns, and the columns are either prefabricated, or precast on site, or cast in situ, so as to pass through the junction. The beams are normally seated on corbels at the columns, for ease of construction and to aid the transfer of the vertical shear from the beam reaction due to gravity load.

2.3.2 Rigid Beam-Column Joints

The connections between prefabricated members can be designed to provide the frame with rigid joints when subjected to live load and seismic forces. Continuity of longitudinal reinforcement through the beam-column joint is obtained either by welding the bars together on to steel plates, or by the use of mechanical connectors, or by anchoring the bars in a sufficient length of cast in situ concrete. Cast in situ concrete is required between the ends of the beams and the columns. The columns may be cast in situ. An example of the beam-column connection of such a frame is shown in Fig.2.

2.3.3 Hinged Beam-Column Joints

The connection between the prefabricated beams and columns can be designed to be hinged. This is normally achieved by seating the beams on column corbels and by holding the beam ends in place by welded steel shoes, or by the use of vertical dowels or bolts, so that shear can be transferred between the beam and column but not bending moment. A typical beam-column connection of that type is shown in Fig. 3. Welded shoes have the advantage that torsion due to unsymmetrical loading on the floor can also be transferred from the beam to the column. The frame is designed to carry only gravity loads. The horizontal seismic loading is resisted by structural walls, prefabricated or cast in situ, appropriately positioned in the building. The floor slabs are designed to act as diaphragms to transmit the seismic forces to the structural walls.

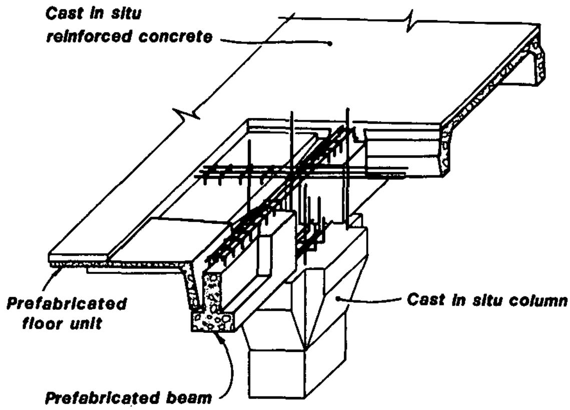


Fig. 2 Example of a "Rigid" Beam-Column Connection in a Frame System.

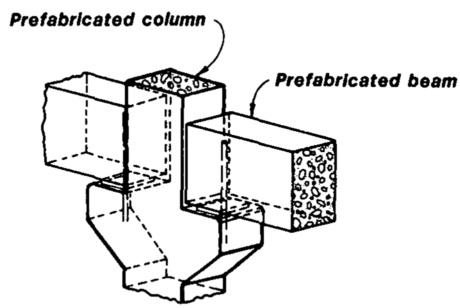


Fig. 3 Example of a "Hinged" Beam-Column Connection in a Frame System.

2.3.4 Floors

Floors for frame systems can be prefabricated as panels occupying the area bounded by the clear spans of the beam grid and acting as two-way slabs, or semi-panels made into continuous two-way slabs by a cast in situ concrete joint between adjacent panels, or one-way spanning elements placed side by side. A topping of cast in situ concrete may be used. Wholly cast in situ concrete slabs are a possible alternative.

The floors need to be designed to act as diaphragms to transfer the seismic forces to the horizontal load resisting elements, as well as to be capable of carrying the gravity loading.

2.4 Slab-Column Systems With Walls

2.4.1 General

Prefabricated slab-column systems with walls have been devised which have as their special feature the method of construction. Two such systems are a lift-slab system involving cast in situ reinforced concrete flat plates and prefabricated reinforced concrete columns, and a system consisting of prefabricated reinforced concrete slabs and columns which are prestressed together after erection to form a continuous structure. Both systems rely on structural walls, either of cast in situ or prefabricated concrete to resist the horizontal seismic loads.

2.4.2 Lift-Slab System With Walls

A lift-slab system which has had extensive application is shown in Fig. 4. The system is used for multistorey residential, office and industrial buildings. The reinforced concrete slabs are cast in situ at ground level, one above the other, and are continuous over the whole area of the building. The reinforced concrete columns are prefabricated in lengths of one clear storey height. The columns are designed to carry only gravity loads. The horizontal seismic loads are resisted by cast in situ concrete structural walls and stair-well cores. The other walls of the building have only partition function. The significant feature of the system is that all slabs are lifted simultaneously as a package. When the required storey level is attained the one storey high prefabricated columns are positioned under the slabs, the bottom slab is then left bearing on the columns, and the lifting procedure continues for the remainder of the slabs. Usually the stair-well cores are constructed before lifting the slabs and act as the guiding and bracing system. The column to slab connection is designed as hinged and the slab to stair-well core connection is rigid. The connections between the slabs, structural walls, and stair-well cores are designed to permit satisfactory transfer of horizontal seismic loading.

2.4.3 Slab-Column System Prestressed for Continuity

A prefabricated slab-column system which uses horizontal prestressing to achieve continuity is shown in Fig. 5. The system has had extensive application in the construction of buildings for a wide range of uses. The reinforced concrete columns are prefabricated in lengths of 1 to 3 storey heights, depending on the building. The reinforced concrete floor slabs are prefabricated generally of a size to fit the clear spans between columns. The slab soffit is either cofferred or flat with the slab voided. After erecting the slabs and columns of a storey the columns and floor

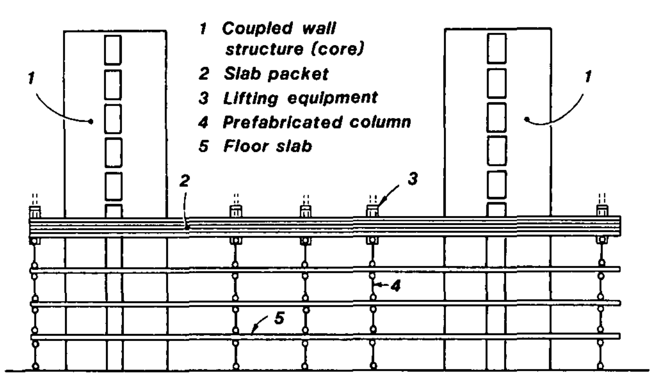
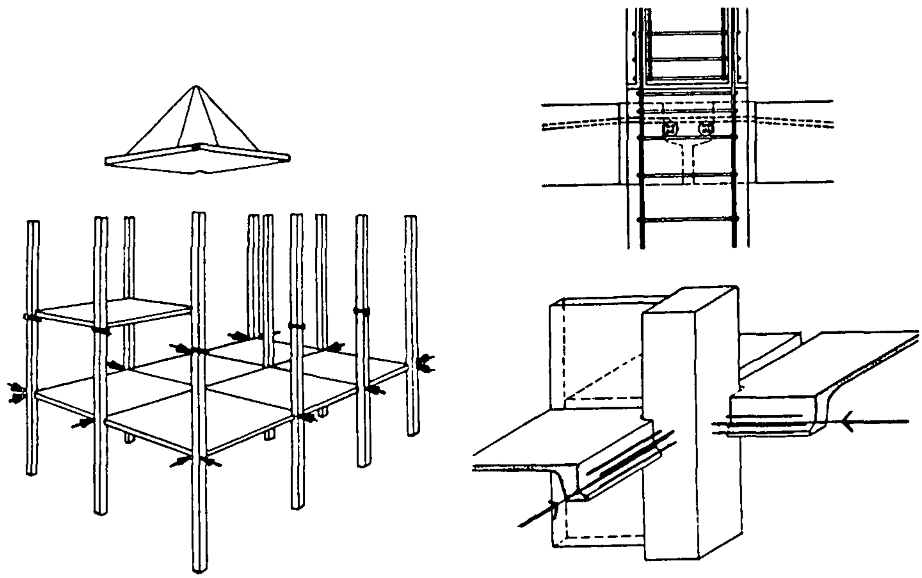


Fig. 4 Slab-Column System Incorporating Lift Slabs and Structural Walls.



(a) Construction Concept (b) Typical Slab-Column Connection

Fig. 5 Slab-Column System Incorporating Prestressing to Achieve Continuity.

slabs are prestressed into a monolithic whole, by prestressing tendons which pass through ducts in the columns at floor slab level and along the gaps left between adjacent slabs. After prestressing, the gaps between the slabs are filled with in situ concrete and the tendons then become bonded within the spans. Horizontal seismic loads are resisted mainly by special prefabricated structural concrete walls which are positioned between columns at appropriate locations. In very tall structures, or in high seismic zones, the structural walls are cast in situ. A range of cladding elements and partition walls are used and the buildings can be given a range of architectural styles.

2.5 Mixed Systems

There remain some structural systems which do not fit specifically into large panel systems or frame systems or slab-column systems with walls. For example, one particular system in use consists of cast in situ structural walls and prefabricated floor slabs. Such systems can be referred to as mixed systems.

2.6 Systems With Non-Structural Prefabricated Elements

A number of structural systems are constructed of cast in situ concrete, for example structural walls and/or floor slabs, but the completed building may utilize prefabricated concrete non-structural units such as staircases, parapets, partition walls, facades, and other wall cladding. There are many possible variations as to which non-structural elements are used and which elements are cast in situ.

2.7 Use of Prefabricated Structural Systems

Prefabricated structural systems have had extensive use in most of the Balkan countries. For example, it is estimated that 60% of the multistorey residential buildings currently constructed in Bulgaria, and more than 50% of the residential buildings with more than four storeys currently constructed in Romania, have structures which are almost totally prefabricated. The experience of the Balkan countries in the use of the various structural systems is reflected by the number of significant prefabricated building systems in current use in those countries. It is to be noted that large panel systems have had the greatest application. Examples of representative prefabricated concrete building systems are given in the design examples presented in Part III of this Manual.

Prefabricated reinforced concrete building systems have also been constructed in many other countries in Europe and Asia which have major building programs. This applies to countries in seismic regions outside the Balkan region, such as the USSR. Other large overseas countries, such as the USA, Canada and Japan, are also interested in the large scale use of prefabricated building systems. A summary of the state of the practice of earthquake resistant design procedures for prefabricated concrete buildings in the USA and Japan is given elsewhere [2,3].

It should be noted that the efficient use of mass production techniques in a centralized factory requires a large market. In less densely populated countries the high initial investment in industrialized plant cannot be justified if the population centres are relatively small and far apart, because the market may be too small and transportation costs too high for efficient production and application of prefabricated systems. Therefore, a careful economic appraisal is required of both the available market and investment in plant when construction by prefabricated concrete systems is being considered.

3. EARTHQUAKE ACTIONS, RESPONSES AND SYSTEM RELIABILITY VERIFICATIONS

3.1 General

Seismic actions result from the vibrations of the soil transmitted to the structure during earthquakes. The seismic actions are the inertia forces induced in the structure when its mass responds to the earthquake, and therefore depend on both the ground motions and on the characteristics of the structure. The ground motions are dependent on many factors involving the seismicity of the area such as source and focal mechanisms, travel path geology and local soil conditions. The main characteristics of the structure influencing its response are its mass, stiffness and damping. The stiffness and damping of the structure in turn are very much dependent on the intensity of the actions induced and on the loading history.

Full means of obtaining descriptions of earthquake motions, likely to be experienced by structures in particular regions, are not yet available. Records of ground motions of severe earthquakes measured in the past form the basis of determining appropriate levels of seismic loading. Dynamic analyses conducted using these strong motion records have shown that structures will not remain in the elastic range if designed to the usual levels of seismic loading recommended by codes. However at the present time the only workable approach for the assessment of an appropriate seismic design action consists of starting from the elastic response spectrum, and then to modify it to account for a number of factors, most notably the actual nonlinear behaviour of the structural type considered, to obtain the design seismic motions. The elastic response spectrum used is determined from the response of a linear one-degree-of-freedom system to the assumed ground shaking (obtained from earthquake records) and gives the maximum acceleration of the mass of the system for a range of periods of vibration (dependent on the stiffness of the system) and for the assumed damping. The recently published Appendix to the CEB-FIP Model Code: "Seismic Design of Concrete Structures" [1] will be used as the model in the following presentation of a general approach.

3.2 Regular Structures

It has already been stressed that in seismic design it is extremely desirable to have structures which are symmetrical in plan and are without vertical discontinuities. Such regular structures behave much more predictably during severe earthquakes, since torsional effects about the vertical axis of the structure are minimized and localized deformations in the members of some (soft) storeys are less likely to occur.

Several codes for earthquake resistant design give definitions of the features of a regular structure. A comprehensive definition is given by the CEB-FIP Seismic Appendix [1] which defines a structure as "regular" when the following conditions regarding both plan and vertical configuration are satisfied:

Plan configuration.

- The building has an approximately symmetrical geometrical shape, with re-entrant corners not exceeding 25 percent of the building external dimensions.

- At any storey the distance (measured in the direction orthogonal to that of the seismic action) between the centre of mass and that of stiffness does not exceed 15 percent of the "resilience radius", defined as the square root of the ratio of the storey torsional and translational stiffnesses. The floor torsional and translational stiffnesses can be computed with reference to the sectional inertias of all vertical structural elements present in each floor. Note that the "resilience radius" in any storey is given by

$$\sqrt{\frac{\text{Torque per unit torsional rotation}}{\text{Shear force per unit deflection}}}$$

Vertical configuration.

- The storey "stiffness ratio" at any storey is not less than 0.70. The storey "stiffness ratio" is defined as the ratio of the inverse of the storey drift (calculated under the static or dynamic design actions), to the average over all the storeys of the inverses of the storey drifts.

- In frame structures, the ratio between actual and design storey shear capacity at each storey (sum of the shear forces contributed by all vertical elements at flexural yielding) does not vary more than 20 percent up the building height.

- In the case of a gradual setback along its height, the setback at any floor is not greater than 10% of the plan dimension in the direction of the setback. This requirement need not be complied with if the setback occurs within the lower 15% of the total height of the building.

The above requirements of the CEB-FIP Seismic Appendix are based on judgement and experience, and cannot be supported at present by vigorous verification. However, the experience of past severe earthquakes has been that many monolithic cast in situ structures with irregular plan or vertical configurations have suffered major damage or collapse due to severe torsional or soft storey effects. The effects of irregular configurations in the case of prefabricated structures is likely to be even worse than for monolithic structures because of the vulnerability of some connections between elements where damage may unexpectedly concentrate. Hence the desirability of regular structures cannot be overemphasized. In any case regularity of structural form generally leads to more efficient and economical structures.

3.3 Design Seismic Actions

3.3.1 Methods of Analysis

The "equivalent static analysis" procedure can be used if the building is classed as "regular". The definition of a regular structure, given by the CEB-FIP Seismic Appendix, is stated in the previous section. In this procedure the actual dynamic (inertia) forces induced in the structure when responding to severe ground shaking are represented by equivalent static forces. The equivalent static forces are found from a design response spectrum with modifications to take into account the ductility of the structure, the importance of the structure, and the effect of soil conditions on the response. The design response spectrum is a suitable elastic response spectrum. The distribution of the equivalent static forces assumed up the height of the structure essentially follows that of the first mode of vibration. The equivalent static forces are the design seismic actions to be applied to the structure to obtain the internal actions. The effects of torsion, due to eccentricity of the design seismic actions, need also to be included. There may be limitations on the use of the equivalent static analysis procedure for very tall buildings due to the greater importance of higher modes of vibration for structures with a long fundamental period of vibration. Hence it is recommended in the CEB-FIP Seismic Appendix that the equivalent static analysis procedure should not be used for buildings taller than 80 m or with a fundamental period of vibration greater than 2 seconds.

A "modal analysis" procedure should be used for irregular structures, or for very tall buildings where the equivalent static analysis procedure does not apply, or for structures of particular importance to the community. The procedure uses dynamic analysis, assuming elastic behaviour, 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 horizontal 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 and then reduced to take into account the ductility of the structure and the resulting design seismic actions applied to the structure to obtain the internal actions. The effects of torsion due to eccentricity of the design seismic actions need also to be included.

In important cases of some unusual structures, or as a research tool, a "time-history nonlinear dynamic analysis" may be justified, in which 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 action-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. Sensitivity studies should be conducted to examine possible margins of error. Modelling the real structural behaviour of complex reinforced concrete systems during large amplitude cyclic loading of variable nature still involves a number of unknown factors.

Only the equivalent static analysis procedure will be discussed in more detail in the material to follow.

3.3.2 Equivalent Static Analysis

The design horizontal seismic action to be applied at each floor level in the direction being analysed is given by

$$F = C_d \gamma_i W_i \qquad (1)$$

where F = design seismic action at floor i
C_d = design seismic coefficient, defined by Eq. 3
W_i = gravity load on floor i
γ_i = distribution factor for floor i, depending on the height of the floor measured from the base of the building and the distribution of gravity load up the height of the structure (see Fig. 6).

The CEB-FIP Seismic Appendix [1] gives the following expression for γ_i

$$\gamma_i = \frac{h_i \Sigma W_i}{\Sigma W_i h_i} \qquad (2)$$

where h_i = height of floor i from the base of the building.
When the gravity load W_i on each floor is the same, Eqs. 1 and 2 indicate that the total design horizontal forces acting on the structure is $C_d \Sigma W_i$ and that the distribution of this total horizontal force up the

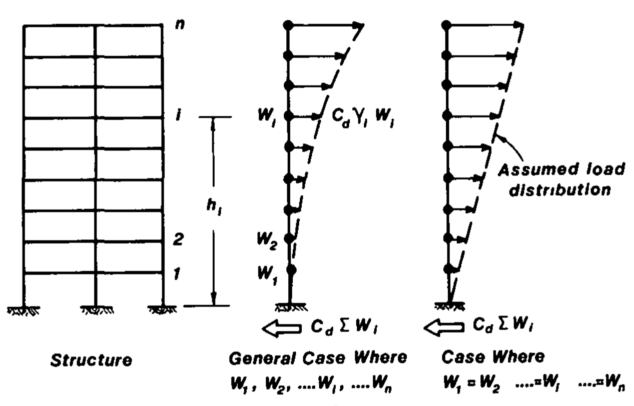


Fig. 6 Assumed Distribution of Design Horizontal Seismic Action.

height of the structure follows the shape of an inverted triangle (see Fig. 6)

It should be noted that in the CEB-FIP Seismic Appendix [1] the design horizontal force is considered to act along each principal axis of the building separately (not concurrently). However the effect of concurrent earthquake actions (oblique earthquake loading which causes actions along both principal axes of the building simultaneously) may need to be considered in the design of columns, joints, walls and foundations (see Section 6.1.5).

The design seismic coefficient C_d is a function of a range of factors. For example, the CEB-FIP Seismic Appendix [1] gives an equation of the following form:

$$C_d = \frac{I S A_{max} \alpha \beta_r}{g K} \qquad (3)$$

where I = factor expressing the importance of the structure, which can be taken as greater than 1.0 for important buildings.

S = site coefficient which is used to modify the design response spectrum to account for the soil conditions at the site. In general the response acceleration for a long period building is greater for soft soils than for hard soils.

A_{max} = peak ground acceleration. Normally a country can be divided into a number of zones of different seismic activity, on the basis of historical records of observed seismicity and/or geologic and tectonic evidence of earthquake occurrence. An A_{max} value for each zone needs to be allocated.

α = spectral amplification factor = ratio of maximum elastic acceleration of structure to peak ground acceleration. A value of α = 2.5 can be assumed in absence of more specific data.

β_r = spectral response factor, equal to or less than 1.0, which depends on the shape of the design response spectrum and the fundamental period of vibration of the structure, T. The variation in β_r is shown diagrammatically in Fig. 7. The viscous damping assumed in the CEB-FIP Seismic Appendix, to obtain the design response spectrum, is 5% of critical. As a simplification, if the period T is not calculated β_r = 1 may be assumed in Eq. 3.

g = acceleration due to gravity.

K = behaviour factor which reflects mainly the ductility properties of the structure and can be expressed as a function of the structural type and the selected ductility level. The CEB-FIP Seismic Appendix defines three ductility levels as follows:

<u>Ductility Level I</u> : Structures proportioned without regard for seismic design provisions, except that longitudinal steel areas in beams and columns are to be within specified limits.

<u>Ductility Level II</u> : Structures proportioned using seismic design provisions so as to be capable of entering the inelastic range of response under repeated reversed loading while avoiding premature-type failures.

<u>Ductility Level III</u> : Structures proportioned using more stringent seismic design provisions so as to ensure the capability to develop selected stable mechanisms

associated with large energy-dissipation capacities.

For monolithic structures the CEB-FIP Seismic Appendix gives the values of the behaviour factor K listed in Table 1.

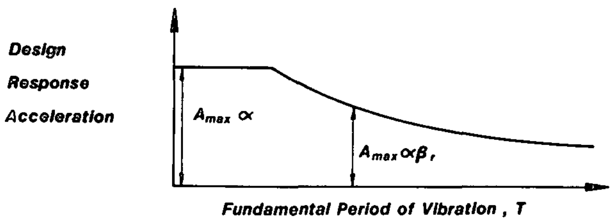


Fig. 7 Typical Shape of Design Response Spectrum for Determining the Design Seismic Coefficient.

Table 1 : Behaviour Factor K for Monolithic Structures [1]

Structural System	Ductility Level		
	I	II	III
Frame	2	3.5	5
Wall or Frame and Wall	2	3	4

Note: The above values for wall or frame and wall systems apply if at least 50% of the horizontal force in each direction is resisted by coupled structural walls. If this condition is not satisfied the above values for wall or frame and wall systems are to be multiplied by 0.7.

The CEB-FIP Seismic Appendix [1] also requires the effect of torsion about the vertical axis of the building to be included in the analysis. When the building is analysed by means of two separate planar models the torsional couples acting at each floor are given by the inertia forces acting at that floor multiplied by the eccentricity

$$e = d \pm 0.1\,a \qquad (4)$$

where d = nominal calculated eccentricity between the centre of mass and the centre of rigidity at the floor measured perpendicular to the direction of seismic action.
a = building plan dimension at right angles to the direction of seismic action.

The term 0.1 a in Eq. 4 is to allow for the very real possibility of an accidental eccentricity of horizontal seismic load (in addition to the nominal calculated eccentricity) due to reasons such as defects in construction and irregularly placed non-structural walls. The term 0.1a is also to account for interaction between torsional and translational modes which can lead to amplification effects, and to torsional ground motions which are a further cause of building torsion. The 0.1a in Eq. 4 has been replaced by a smaller fraction of a, for example 0.05a in some codes. When non-planar models are used for torsion analysis, values for e different from those given by Eq. 4, are recommended in the CEB-FIP Seismic Appendix [1].

The effects of vertical ground accelerations should be taken into account in the design of those structural elements which are particularly sensitive to vibrations in the vertical direction, for example long span structures, cantilevers and balconies. For such elements a design vertical seismic loading may need to be adopted if allowance has not already been made for this effect by the use of appropriate safety factors in design.

3.3.3 Possible Modifications to Seismic Design Actions for Prefabricated Structures

(a) The Fundamental Period of Vibration, T

An important step in calculating the design seismic force F from Eq. 1 is the estimation of the fundamental period of vibration of the structure, T, since the value of the design seismic coefficient C_d given by the design response spectrum depends on T. For prefabricated reinforced concrete structures the actual value for the period T may be larger than for equivalent monolithic structures, due to the less stiff joints and the reduction in stiffness which may occur at the joints during cyclic loading. However, it is important for the designer not to overestimate the increase in the period T of a prefabricated concrete structure due to joint flexibility, since such overestimation will lead to a lower seismic design action than is appropriate. Ideally, forced vibration tests should be conducted on completed prefabricated structures of the type considered to establish experimentally their dynamic characteristics.

For monolithic frame structures the CEB-FIP Seismic Appendix [1] suggests as an approximation for the fundamental period of vibration $T = n/12$ seconds, where n is the number of storeys of the structure above the foundation. The actual value for T for prefabricated reinforced concrete frame structures will generally be larger than the value for monolithic frame structures if beam-column joints designed to be rigid are in fact less stiff than for monolithic construction, or if some beam-column joints are deliberately designed to be hinged.

Wall structures will have a smaller value for the fundamental period of vibration T than frame structures of the same total height, due to the greater stiffness of wall systems. Again, there may be some reduction in stiffness due to joint flexibility in prefabricated wall systems. However a large panel building can act as a relatively stiff box structure and as a result the flexibility of the foundation may have a significant influence on the period of vibration. A number of vibration tests and accompanying theoretical studies have been conducted on full-scale prefabricated large panel building structures. For example, Kollegger and Bouwkamp [4] conducted forced vibration tests on full-scale 12 storey high large panel prefabricated concrete buildings and from the test results and subsequent analytical work it was found that the fundamental period of vibration was approximately $T = n/30$ seconds if the soil was relatively rigid and $T = n/24$ seconds if the soil was relatively flexible, where n is the number of storeys of the structure above the foundation.

It is evident that a relatively flexible soil can significantly lengthen the period of vibration. Other forced vibration studies have been carried out in a number of countries. For example, Jurukovski [5] has conducted forced vibration tests on seven full-scale large panel prefabricated concrete buildings constructed in Yugoslavia, from 5 to 22 storeys in height. Jurukovski found that T = n/25 seconds gave reasonable agreement with the measured periods for buildings founded on relatively poor soils with piled foundations and also for buildings founded on relatively good soil with strip footings. Erdik and Gulkan [6] have conducted forced vibration tests on a number of full-scale buildings in Turkey and concluded that for large panel prefabricated buildings, foundation configurations and soil conditions have a major influence on the period as well as storey height and generalizations were difficult to make. Nevertheless in considering their own results and those of Bouwkamp and Jurukovski (see Fig. 8) they found that the expression T = n/30 seconds gives a fairly representative value for the fundamental period of vibration of large panel prefabricated buildings, where n = number of storeys of structure above the foundation.

It is also possible to calculate analytically, using suitable models for the structural system and soil-structure interaction, the fundamental period of vibration of a particular structure. Approximate empirical formulae for T will not provide sufficient accuracy in all cases.

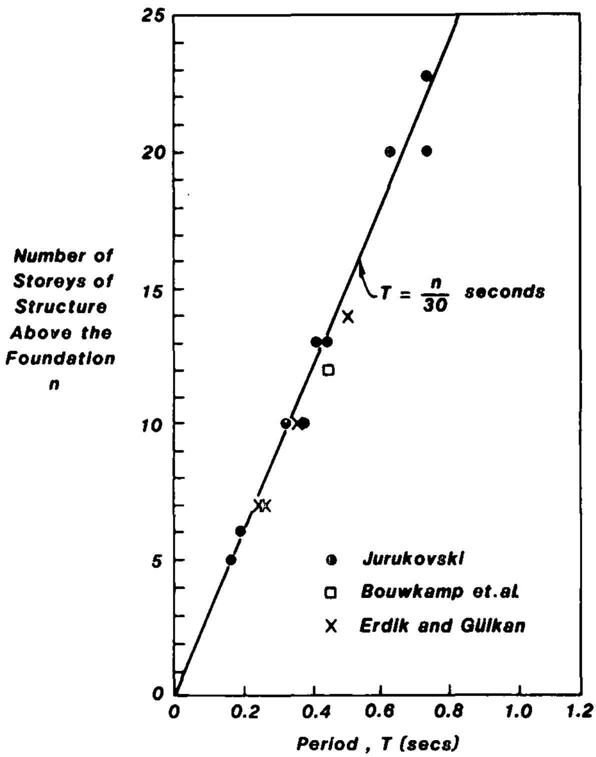


Fig. 8 Measured Fundamental Periods of Vibration from Forced Vibration Tests on Large Panel Prefabricated Concrete Buildings [6].

(b) The Behaviour Factor, K

The values for the behaviour factor K shown in Table 1 are for monolithic structures and assume a good technological level and quality control procedures. They also reflect the ductility properties available from the various monolithic structural systems proportioned according to the CEB-FIP Seismic Appendix [1] for the various ductility levels. Some prefabricated concrete elements and their connections forming part or all of the seismic load resisting system may not be capable of sustaining the inelastic displacements expected of Ductility Level III, or even Ductility Level II, systems. That is, a connection can readily be designed for adequate strength under monotonic loading, but a significant reduction in strength and energy dissipating capacity may result if yielding occurs in the region of the connection when loaded cyclically during a severe earthquake. The higher K values of Table 1 should only be used when prefabricated structural systems form part or all of the seismic load resisting system if it can be shown by analysis or experiment, based on accepted engineering principles, that the assumed level of ductility can be achieved. It may be necessary to use medium values for K, for example those corresponding to Ductility Level II, for many prefabricated concrete structures. If there is doubt about achieving adequate ductility at a particular type of connection, a possible design approach is to make that connection deliberately overstrong, thus forcing the yielding to occur away from the connection in regions (at other more ductile connections and/or within the elements) which can more readily be designed for ductility.

3.4 Design Load Combinations

According to the CEB-FIP Seismic Appendix [1] the design action to be used for verification of the limit states of stiffness, strength and collapse is given by the following combination of load effects

$$S_d = G + P + E + \Sigma\psi_i Q_{ik} \tag{5}$$

where S_d = design action
G = includes all permanent loads at their nominal value
P = prestressing force at long term
E = design seismic action
Q_{ik} = are the fractile values of the extreme distribution of all the variable loads whose duration of application is long enough for the probability of their joint occurrence with earthquake action being not negligible
ψ_i = factors required to pass from Q_{ik} to the average value of Q_i in their instantaneous distribution.

The evaluation of the seismic action E is based on all the gravity loads appearing in Eq. 5, factored by the appropriate ψ values. Note that in Eq. 5 all the partial safety coefficients γ_f are set to unity.

3.5 Limit States to be Verified

The limit states to be verified in seismic design are stiffness, strength and collapse.

3.5.1 Stiffness Verification

The elastic interstorey drift, Δ_{el}, resulting from the application of the design horizontal seismic action obtained from the equivalent static analysis or modal analysis procedures at any storey should not exceed an allowable value. For example, the requirement of the CEB-FIP Seismic Appendix [1] is that

$$\Delta_{el} \leq \frac{0.01h}{K} \tag{6}$$

where h = clear height of storey
K = behaviour factor defined in Table 1 for monolithic structures.

If the allowable value for Δ_{el} is exceeded, separation of the non-structural elements is required of an adequate amount for permitting a displacement of $0.25\Delta_{el}K$ to take place without restraint. In no case should Δ_{el} exceed 2.5 times the allowable value set in Eq. 6.

3.5.2 Strength Verification

The design action effects, allowing for possible second order effects, should nowhere exceed the corresponding design strengths. Design action effects are computed by elastic analysis, possibly modified by redistribution, and amplified where appropriate to ensure that undesirable (brittle) types of failure do not occur. Section 6.2 gives specific recommendations for the values of amplification factors. The design strengths of the sections should be calculated using theory for the ultimate limit states of strength rather than by approximate elastic theory allowable stress approaches.

Note the use of amplification factors for some design actions in order to avoid brittle failures when the structure is responding to a very severe earthquake. The need for amplification factors for some design actions may be demonstrated as follows. The first step of a logical seismic design procedure would be to select suitable regions of the structure where energy dissipation can occur by ductile flexural yielding and to design these regions for adequate flexural strength and ductility. The next step is to recognise that during a severe earthquake the actual moments that develop at those selected plastic hinge positions can be significantly greater than the design values. This greater moment capacity at the plastic hinges is due to the actual areas of reinforcement present being greater than needed, and due to the actual strength of the steel reached during plastic hinge rotation being greater than the characteristic yield strength (due to strain hardening, etc.) This increase in moment will be accompanied by a corresponding increase in the actions throughout the structure. Therefore the design actions associated with brittle modes of failure should be amplified by suitable margins to ensure that the brittle failure strengths are not reached. The amplification factors can be calculated from the expected increases in moments at the plastic hinges. For example, in tall frames plastic hinging in the beams is the desired mode of energy dissipation and can be achieved if the design column moments, and the design shear forces in the beams, columns and joints, are suitably amplified to ensure as far as possible that column yielding or shear failure do not occur.

The stability of the structure when its strength is reached also needs to be verified. The deformability index θ may be defined as

$$\theta = \frac{W\Delta_{el}}{Vh} \tag{7}$$

where W = total gravity load above the considered storey
Δ_{el} = elastic interstorey drift due to the design actions
V = seismic design shear force acting across the storey considered
h = storey height.

For example, according to the CEB-FIP Seismic Appendix [1] the stability verification is considered to be satisfied if $\theta < 0.1$, or if $0.1 \leq \theta \leq 0.2$ if the second order effects are calculated and added to the design forces. In no case is $\theta > 0.2$ permitted.

3.5.3 Collapse Verification

The safety requirement against collapse can be considered to be met if the strength and stability verifications are satisfied and if the primary seismic load resisting elements and their connections are detailed to possess adequate ductility. Codes normally give procedures for detailing for intended ductility levels. For example, the CEB-FIP Seismic Appendix [1] states requirements for detailing monolithic frames and walls for Ductility Levels I, II and III. Note that it is not intended by codes that the designer should calculate from first principles the specific ductility requirements of each region of the structure and then match that demand. Instead the detailing procedures recommended by codes for the ductility levels and the structural types are intended to ensure that the available ductility of the yielding regions will be adequate.

4. STRUCTURAL ANALYSIS PROCEDURES

4.1 General

The commonly used structural analysis procedures for monolithic structures assume linear-elastic behaviour of the structural system. However, procedures which include nonlinear behaviour are also available. The majority of procedures make the assumption that the floors act as rigid diaphragms. In making an overall structural analysis of prefabricated concrete structures it is desirable to include the effects of any reduced stiffness which may occur at the connections. Also, floors constructed from a number of prefabricated elements may not act as rigid diaphragms, and it may be necessary to include the effect of diaphragm flexibility on the distribution of horizontal seismic loading among the vertical elements of the structural system. In general, the structural analysis associated with design is in two stages. One dealing with the analysis of the structure as a whole to assess the action effects, and the other dealing with the analysis of the individual sections of the structural elements and the connections between the elements under the assessed action effects.

The following sections give an outline of structural analysis procedures for a range of structural types. Only the effect of horizontal seismic actions will be considered. The effect of gravity loading is additive.

4.2 Analysis of Wall Systems

4.2.1 Analysis of Wall Systems Assuming Monolithic Behaviour

The assumption of monolithic behaviour is adequate for the determination of the distribution of the internal forces due to seismic actions in many prefabricated structural wall systems. The analysis of monolithic structural wall systems can be carried out by one of a number of methods.

The choice of the method depends on the structural configuration of the particular wall system and the degree of accuracy sought. The methods of analysis for the two basic structural configurations of structural walls (with and without openings) are described below [7]:

(a) Structural Walls Without Openings

An approximate elastic analysis can be conducted assuming that the floors are very flexible in the out of plane direction compared with the flexural stiffness of the walls in their major direction, but that the floors are capable of acting as rigid diaphragms in their plane. This means that the walls can be analysed as separate but linked cantilevers and that the walls will deflect equally at floor level when the building is subjected to horizontal seismic loading (see Fig. 9).

The flexural stiffness of walls with rectangular cross section with respect to their weak axis of bending may be neglected. The assumption of equal horizontal displacement of all cantilever walls at floor level means that the total horizontal seismic load applied at each level is distributed among the individual walls in proportion to their flexural stiffness. Thus the seismic action to be carried by cantilever wall i at a particular floor level when the centres of mass and rigidity of the building coincide and when the structure is in the elastic range is given approximately by

$$F_i = \frac{I_i F}{\Sigma I_i} \tag{8}$$

where F = total horizontal seismic action applied at that floor level
I_i = moment of inertia of section of wall i about axis perpendicular to the direction of the action.

The above expression neglects the effect of shear deformations, but for many walls the effect of such deformations should be included.

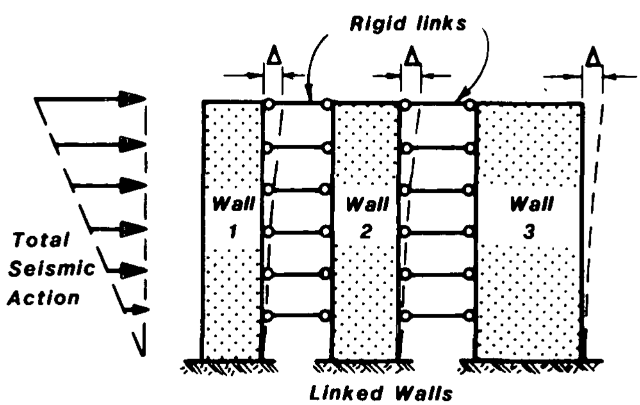


Fig. 9 Mathematical Model for Horizontally Loaded Structural Walls With Rigid Diaphragms.

In design it is generally considered that the centres of mass and rigidity of the building do not coincide (see Eq. 4). If the x and y axes are in the directions of the principal axes of the building plan, the seismic action to be carried in the x and y directions by wall i at a particular floor level when the structure is in the elastic range is given approximately by

$$F_{ix} = \frac{I_{iy}F_x}{\Sigma I_{iy}} + \frac{(F_x e_y - F_y e_x)y_i I_{iy}}{\Sigma(x_i^2 I_{ix} + y_i^2 I_{iy})} \quad (9a)$$

$$F_{iy} = \frac{I_{ix}F_y}{\Sigma I_{ix}} + \frac{(F_y e_x - F_x e_y)x_i I_{ix}}{\Sigma(x_i^2 I_{ix} + y_i^2 I_{iy})} \quad (9b)$$

where F_x, F_y = total horizontal seismic action applied at floor level in the x and y directions, respectively.

I_{ix}, I_{iy} = appropriate moments of inertia of section of wall i about its x and y axis, respectively.

x_i, y_i = coordinates of wall i, with origin taken at the centre of rigidity of building plan.

e_x, e_y = eccentricities resulting from the noncoincidence of the centres of mass and rigidity, in the x and y directions, respectively.

The above expressions ignore the torsional rigidity of the wall sections, as well as neglecting the effects of shear deformations, but the effect of warping torsion is included. For many walls the effect of shear deformations should be included.

If the seismic actions are considered to act along each principal axis of the building separately, then F_y should be put equal to zero in Eq. 9a when F_{ix} is being determined, and F_x should be put equal to zero in Eq. 9b when F_{iy} is being determined.

The bending moments and shear forces up the height of the individual cantilevers can be determined from the seismic actions applied to each wall. Cracking will reduce the flexural rigidity of each wall but the use of gross (uncracked) section I values will generally result in sufficient accuracy.

If the building is loaded horizontally to the ultimate limit state, the flexural strength of all walls will be reached at about the same stage if the moments have been determined by the above approximate elastic analysis. Some redistribution of actions between the walls will occur because of differences in the flexural rigidities in the inelastic range, after cracking and inelastic strains commence, but the amount of redistribution required should not be great.

(b) Structural Walls With Openings

If the structural walls are penetrated by vertical rows of openings, typically for doors or windows, the parts of the walls on each side of the openings may be considered to be interconnected by beams referred to as coupling beams or lintels. A wall with one vertical row of openings may be thought of as two separate walls with coupling beams and is known as a coupled structural wall (see Fig. 10a). When horizontal seismic loading is applied to the wall, the resulting deformations will cause shear forces

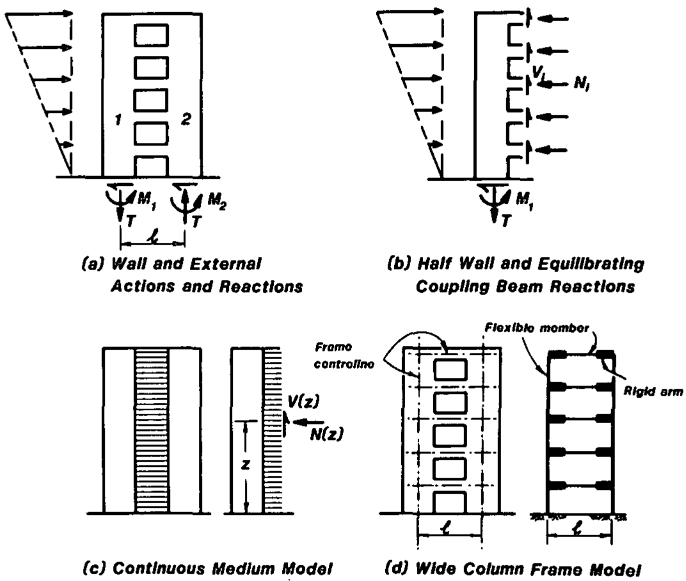


Fig. 10 Mathematical Models for Horizontally Loaded Coupled Structural Walls

to be transferred via the coupling beams from one side of the wall to the other (see Fig. 10b). Thus the coupling beams are subjected to shear force, flexure and axial horizontal force. If the span/depth ratio of these beams is small, the shear deformations become very significant and the effect of axial (vertical) deformations of the separate sides of the wall also become important.

The axial vertical forces T induced at the bases of the wall sides, tension in wall side 1 and compression in wall side 2 (see Fig. 10a), are the result of the accumulation of the shear forces in the coupling beams up the height of the wall. Thus T may be written as

$$T = \Sigma V_i \tag{10}$$

where V_i = vertical shear force in coupling beam at floor i. The overturning moment of the applied horizontal seismic loading (defined as the moment of the seismic actions about the base of the wall) can be written as

$$M_o = M_1 + M_2 + T\ell \tag{11}$$

where M_1, M_2 = moments at the bases of the wall sides 1 and 2, respectively.

T = axial vertical forces induced at the bases of wall sides 1 and 2, given by Eq. 10.

ℓ = distance between the centroids of the sections of wall sides 1 and 2.

There are a number of methods which can be used to determine the shears V_i in the coupling beams up the height of the wall, the reactions T, M_1 and M_2 at the bases of the wall sides, and the resulting displacements. Two approaches are mentioned below.

Continuous medium method

Beck [8], Rosman (for example [9]) and others have treated the vertical row of coupling beams as a continuous elastic medium (see Fig. 10c). The laminae making up this medium are subjected only to shear and axial horizontal forces at their midspan. This results in a highly statically indeterminate situation being reduced to an idealization which is more readily solved. By considering the requirements for compatibility of deformations in the wall, a second order differential equation can be formed and solved to give the shears up the height of the laminae, and the other actions. Coull and Choudhury [10] have published charts which enable the distribution of actions in the elastic range to be readily calculated. The inclusion of the effect of foundation rotation may also be necessary in some cases. The method is most accurate when there is a reasonably large number of coupling beams. The method can also be extended to the case where plastification occurs in the coupling beams and the ultimate horizontal load capacity is calculated for the situation when all the laminae are yielding and plastic hinges have developed at the bases of the two sides of the wall [11].

Wide column frame analysis

The wall may be treated as a rigid jointed frame with wide columns (see Fig. 10d). The idealized frame can be considered to be rigid within the joints. The effects of shear deformations in the beams and axial deformations in the columns should be included in the analysis. Foundation flexibility can also be taken into account. The method is most accurate when the frame members do not have a small length/depth ratio.

4.2.2 Analysis of Walls Assuming Shear Displacements at Vertical and Horizontal Joints

In prefabricated concrete wall systems there may be significant slip (shear displacements) along some vertical and horizontal joints. Fig. 11b and c shows horizontal actions applied to a panelized cantilever wall in which slip has occurred along both vertical and horizontal joints due to shear forces transferred along these joints. The effect of slip along the horizontal joints is to reduce the stiffness of the system. The effect of slip along the vertical joints is also to reduce the stiffness of the system and to cause the behaviour of the wall to be part way between that of a monolithic wall and that of two isolated walls. In the limit, if the vertical joint of a wall has no resistance to slip along it (that is, the vertical joint is frictionless), the wall will function as two separate isolated cantilever walls and the stiffness will be equivalent to the sum of the stiffness of the two separate walls (see Fig. 12). A panelized wall system which, due to slip at horizontal and vertical joints, has a smaller stiffness than a monolithic wall system, is difficult to analyse precisely because models for the shear stress-slip relationships for the various types of joints are needed. The relationship between shear stress along a joint and the corresponding slip is complex, and depends on the normal stress across the joint, the reinforcement across and along the joint, the type of joint face, the material strengths and stiffness, and other factors (see Section 5.2). Given the joint shear stress-slip model, there are analytical techniques which can be used to determine the

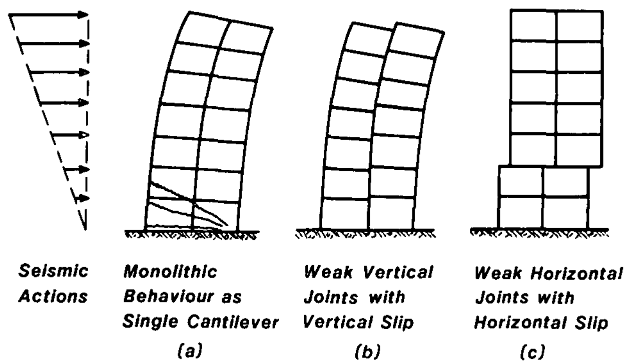


Fig. 11 Panelized Wall With Monolithic Behaviour, Slip Along Vertical Joints and Slip Along Horizontal Joints, When Horizontally Loaded

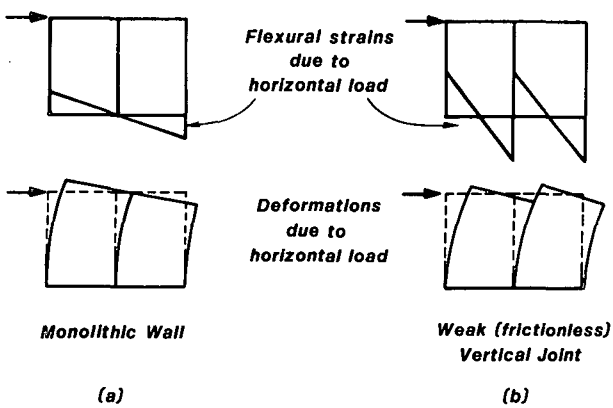


Fig. 12 Panelized Wall With Monolithic and Frictionless Vertical Joint When Horizontally Loaded

forces in and deflections of a panelized wall system which does not behave monolithically. For example, a continuous medium of appropriate shear stiffness can be placed in the joints in the vertical and horizontal directions and the idealized system analysed. Finite element analyses can be used.

4.3 Analysis of Frame Systems

The bending moments and other internal forces in frame structures are generally computed using linear-elastic structural analysis. The bending moments and other actions so found can be modified to allow for some moment redistribution. These actions are used for both the stiffness and strength verifications. The use of the bending moments and actions calculated assuming linear-elastic behaviour to determine the required reinforcing steel to achieve the necessary strength at ultimate load is valid because the bending moment distribution so found is statically admissible. However once cracking and inelastic strains commence the flexural stiffness of the members will change and, unless the bending moments calculated by linear-elastic theory are based on the final complex distribution of stiffnesses, some moment redistribution will be required before all the critical sections achieve their flexural strength. Generally the flexural stiffness values used in structural analysis are based on the gross (uncracked) sections. However in many frames the beams will be cracked but the columns will remain uncracked in the service load range. Joint flexibility, if the joints cannot be considered to be rigid, needs to be taken into account in the analysis.

4.4 Analysis of Wall-Frame Systems

Mixed systems of structural walls and frames are used quite commonly. Many prefabricated concrete frames do not have moment resisting joints at all connections and rely on structural walls to resist a large proportion of the horizontal load on the building, and also to limit the amount of horizontal deflection, leaving the more flexible frames to carry the majority of the gravity loading. Thus combined systems of frames and walls can often be used to advantage. Several methods of structural analysis are available to determine the moments and other internal forces, for example [12]. The walls and frames in one direction are treated as planar systems tied together at each floor level by rigid links which represent the floor diaphragms (see Fig. 13). It is assumed in most analyses that the floor diaphragms are rigid.

There is a basic difference in the behaviour of cantilever walls and frames subjected to horizontal loading that makes invalid the concept of sharing the total horizontal seismic action between them in proportion to their flexural stiffnesses. Frames deform mainly in a shear mode, and the interstorey deflections depend mainly on the shear force applied at the storey level, as illustrated in Fig. 14a. Walls deform predominantly in a flexural mode as in Fig. 14b. Note that because of a different deflected shape a wall can oppose a frame in the upper floors. Only at the lower floors do the wall and frame assist each other in carrying lateral load, as is illustrated in Fig. 14c. Hence tall buildings with walls and frames require careful structural analysis to ensure that the interaction effects are included.

4.5 Effect of Diaphragm Flexibility

It is commonly assumed in the analysis of structural systems responding to horizontal seismic loading that the floors act as rigid diaphragms when transferring the inertia forces from the mass on the floors to the vertical elements of the structural system. In fact the stiffness of floors as diaphragms ranges from rigid to very flexible depending on their geometry, the method of construction and the stiffness of the vertical elements they connect. If the floors act as rigid diaphragms, all the vertical elements connected by the floor will deflect horizontally by the same amount and the

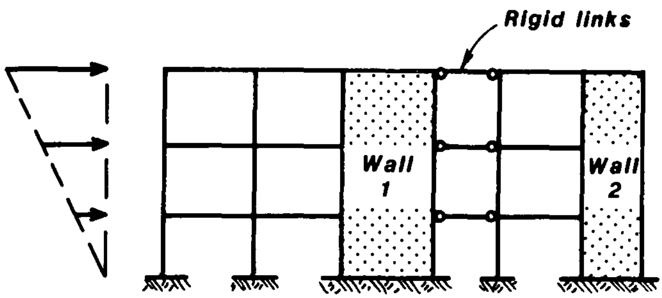


Fig. 13 Two-Dimensional Mathematical Model for Horizontally Loaded Frame and Wall Systems

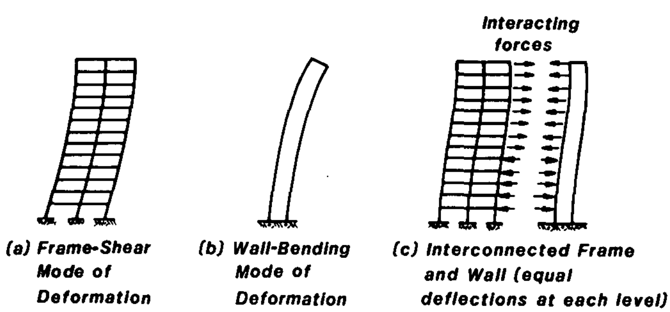


Fig. 14 Deflected Shapes and Interaction of Frames and Walls

seismic force transferred to each vertical element will be dependent on the stiffnesses of the vertical elements and the total mass on the floor. For very flexible diaphragms each vertical element of the building deflects independently and the seismic force transferred to each vertical element will be dependent on the mass on the area of floor adjacent to the vertical element. The behaviour of diaphragms with some flexibility is part way between these two extremes. Methods are available for considering the effect of diaphragm flexibility, for example [13]. Note that floors constructed from a number of prefabricated components may form diaphragms which need to be treated as having some flexibility, unless design procedures are adopted to effectively connect the floor components together and to the remainder of the structure. A topping of cast in situ concrete over prefabricated concrete floor components is an effective method for decreasing diaphragm flexibility.

4.6 Effect of Foundation Flexibility

Studies have shown that the dynamic response of a stiff box structure (for example, a large panel building) is very much influenced by soil-structure interaction, particularly if the structure is supported on relatively soft soil. For example, forced vibration tests conducted on large panel buildings have shown that deformations of the soils due to foundation pressures can result in horizontal displacements of the buildings at foundation level which are of the same order as the interstorey horizontal displacement in the first storey [4]. However, the dynamic response of a frame structure is not significantly modified by soil-structure interaction if supported on a firm soil. The major effects of soil-structure interaction are to lengthen the fundamental period of vibration of the building, and to modify the dynamic response of the structure. This modification may make the dynamic response (for example, maximum acceleration) less than or greater than that for the fixed base structure, and the only reliable way to ascertain the form of the modification is to analyse the structure using a model that includes the effect of the soil stiffness and energy dissipating characteristics. A common method of modelling the foundation properties is to use a translation spring and dashpot and a rotation spring and dashpot. An extra flexible (dummy) storey beneath the structure with properties that model the foundation may also be used as a means of investigating soil-structure interaction effects.

4.7 Computer Programs

The analysis of structural systems involves procedures which in many cases are too lengthy for hand computation. Computer programs in such cases are an essential aid to analysis. The structures are typically idealized as assemblages of one-dimensional linear elements to make up frames and two-dimensional elements for structural walls and floors. Two-dimensional idealizations (planar models) of buildings are used where possible, but three-dimensional idealizations can be used. Torsional motions of buildings are often important and can be accounted for by three-dimensional idealizations.

Although structural analysis by computer has become a basic ingredient of structural design, engineering judgement is still needed when selecting the models to represent accurately the prototype behaviour. Computer programs have now been developed in many countries for linear or non-linear static or dynamic analyses.

Of particular interest are the computer programs developed at the University of California, Berkeley. For example, SAP IV is a structural analysis program for the static and dynamic response of linear-elastic three-dimensional structural systems idealized by various combinations of linear elements representing the beams and columns, finite elements representing the walls, and rigid diaphragms representing the floors [14]. TABS is another structural analysis program for the linear-elastic analysis of three-dimensional frame and shear wall buildings with rigid diaphragms [15]. DRAIN-2D is for the dynamic response of inelastic two-dimensional structures [16] composed of linear elements. Recently developed programs have included the effects of diaphragm flexibility.

5. CONNECTIONS

5.1 General

The word "connection" is used to describe the region where the elements are connected, and the "joint" is the area between the connected elements.

A connection between prefabricated concrete elements should exhibit a service load performance which is equal in quality to the elements it connects, and should possess a strength which corresponds at least with the most adverse ultimate load combinations. In principle, the strength of the connection should not govern the strength of the structure and hence connections should be located where the design actions are small. However, considerations of production, storage, transportation and assembly generally require the subdivision of prefabricated concrete elements into simple forms. Therefore connections often need to be located where the design actions are large, for example at beam-column junctions. Connections in critical regions of the structure should have adequate ductility for seismic load conditions, in line with the preferred mode of energy dissipation (see Section 6.1.4).

In joints involving cast in situ concrete it is desirable to connect the reinforcing steel crossing the joint in as short a distance as possible so as to avoid large quantities of in situ concrete and site formwork. However, if the joint region is too narrow there will be problems associated with the placing of the in situ concrete as well as the detailing of the connecting bars. It is evident that ease of construction and access for depositing in situ concrete are other prominent considerations for joint design.

A wide range of details for joints are possible. In general the joint may either be "wet" or "dry". Wet joints are constructed with cast in situ concrete. If structural continuity is required through the joint, protruding reinforcing bars from the elements are lapped, looped, welded or otherwise connected in the joint region before the in situ concrete is placed. Dry joints are constructed by welding or bolting together steel plates or other steel inserts which were cast into the ends of the elements for this purpose, thus transferring the actions between elements at discrete points and where the steel inserts are connected. Joints between prefabricated elements may be identified as "vertical" or "horizontal", depending on the direction of the edges of the units they connect.

Note that when the welding of reinforcing bars is necessary it must be conducted with proper quality control, otherwise the ductility and strength of the steel will be impaired. The importance of the quality of workmanship in welded connections cannot be overemphasized.

5.2 Wet Joints Between Prefabricated Panels

5.2.1 General

Wet joints are commonly used in large panel reinforced concrete construction, especially in earthquake regions. The design aim is generally to achieve full continuity as far as possible, although in practice it is often difficult to assure monolithic behaviour in some connections during severe earthquake loading (see also Section 6.1.4).

The predominant action transferred by vertical joints between adjacent concrete panels is shear arising mainly from bending due to seismic or wind actions. The predominant actions transferred by horizontal joints between adjacent panels are shear and normal forces (compression or tension) acting simultaneously due to gravity and seismic or wind actions.

The strength and stiffness of a connection is influenced by a number of variables such as the type of surface preparation on the panel ends to be connected, the panel and joint concrete quality, the amount and distribution of reinforcing steel in the joint region, the type of actions (monotonic or cyclic), and the number of cycles in the case of cyclic actions.

Typical panel end preparations are illustrated in Fig. 15. The shear strength and stiffness of joints is significantly improved by the use of shear keys

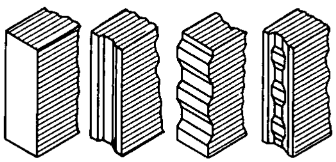


Fig. 15 Typical Panel End Preparations.

with suitable arrangements of connecting reinforcement across the joint. Hence castellated types of joints with distributed connecting reinforcement along the length of the joint are preferred for prefabricated structures in earthquake regions.

5.2.2 Mechanisms of Force Transfer Across Concrete Interfaces

In the general case, shear and normal forces need to be transferred between prefabricated concrete elements and the cast in situ joint concrete. This involves the transfer of actions across the interface of concrete of different ages. Several basic force transfer mechanisms involving both the concrete and the reinforcing steel can generally be identified.

The adhesion that exists at the interface will transfer shear between the two concrete surfaces until it is broken and a crack forms along the interface. The adhesion shear strength will depend on the surface treatment, surface roughness, curing conditions, and material composition. Note that near the interface of the two concrete surfaces the deformations will be increased due to imperfections in the concrete. It may be necessary, for example, when calculating the deformations due to normal forces to use a lower modulus of elasticity E_o within a limited length of concrete near the interface rather than using the value E_c corresponding to the full mass modulus of elasticity.

Once slippage commences along the crack at the interface shear is transferred across the crack by a number of basic mechanisms. Fig. 16 shows the external actions and the mobilised resisting forces acting on an idealized crack. The normal compressive reaction across the crack will result in shear transfer by friction along the crack. The coefficient of friction has been found from tests not to be a constant but is mainly dependent on the roughness of the interface, the type of loading (monotonic or cyclic), the order of the shear slip, and the magnitude of the normal stresses.

When steel reinforcement bars exist across the crack, additional shear force will be transferred by bond action and by dowel action. Bond enables the bars to be anchored in the concrete each side of the crack. The bars will apply a clamping force across the crack when sliding shear displacements occur along the crack and the crack is forced to open slightly as the rough surfaces ride over each other. The resulting clamping force will increase the shear transferred by friction. The steel crossing the crack will reach its yield strength if it is properly anchored each side of the crack. Dowel action also requires sliding shear displacement to occur along the crack before it is mobilised. These shear resisting mechansims are illustrated in Fig. 16.

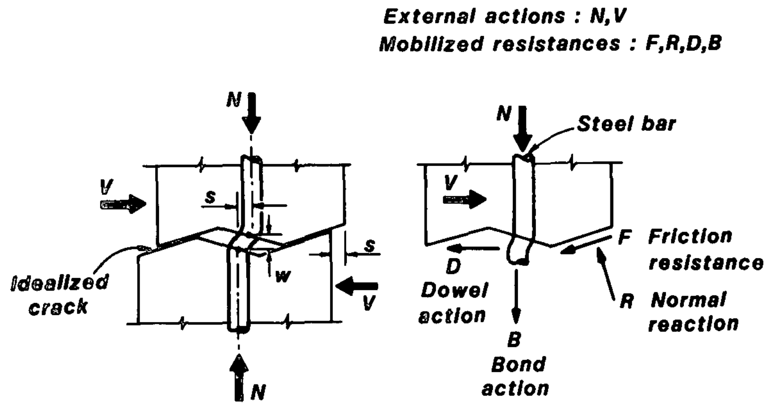


Fig. 16 Shear Slip Along a Crack and Resisting Mechanisms.

In Fig. 16 an imposed shear slip of s along the idealised crack causes an opening (separation) of the crack of w at the transverse reinforcement which is resisted by tensile force in the reinforcement developed by bond. The transverse reinforcement is subjected to a kinking displacement of s and a local extension of w. The shear force resisted along the crack may be expressed in terms of the frictional force F (which is a function of the normal reaction R which in turn is dependent on the external normal force N and the force in the reinforcing bar B) and the dowel force D. Taking into account the likely values for w, the steel forces B and D can be determined from suitable B versus s and D versus s constitutive relations, and used together with the friction coefficient to determine the shear force V versus shear slip s relation. Such an approach is discussed in more detail by Tassios [17].

Note that during cyclic normal loading (tension-compression) the concrete may be precracked by prior tensile loading. When compression is applied to concrete which has been precracked, and when some shear displacement has occurred along crack, the compressive force is transferred through the crack by the contact of the protruding parts of the rough interface of the concrete. As a result the compression stiffness of the connection is lower than for the uncracked situation until local crushing allows the crack to mainly close.

5.2.3 Shear Joints Under Monotonic Loading

Figs. 17 and 18 show typical shear stress - shear slip curves measured for wet joints between prefabricated reinforced concrete panels with plain end surfaces and castellated end surfaces, respectively, during monotonic loading. In both cases reinforcing bars either cross the joint, or are connected across the joint, perpendicularly. The shear strength of a joint with shear keys, such as a castellated joint, may be many times that of a plain joint, mainly because inclined diagonal struts can form as illustrated in Fig. 18 which increases the shear resistance. Tests have shown that the shear strength (maximum shear capacity) of a castellated joint is reached at a shear slip of about 1 mm followed by a reduction in shear capacity at higher slips, whereas for plain joints the shear strength is reached at a slip of several mm and the subsequent reduction in shear capacity at higher slips is small.

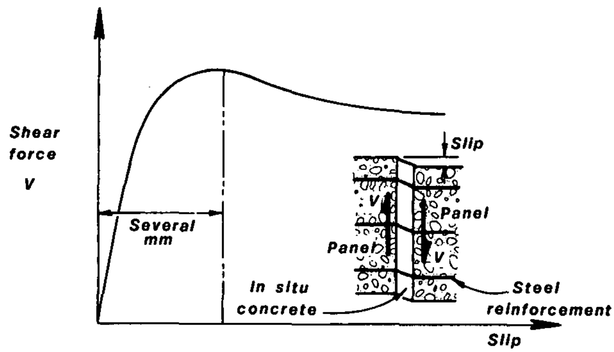


Fig. 17 Shear Stress - Shear Slip Relationship for a Joint Between Panels With Plain End Surfaces and With Reinforcing Steel Crossing the Joint.

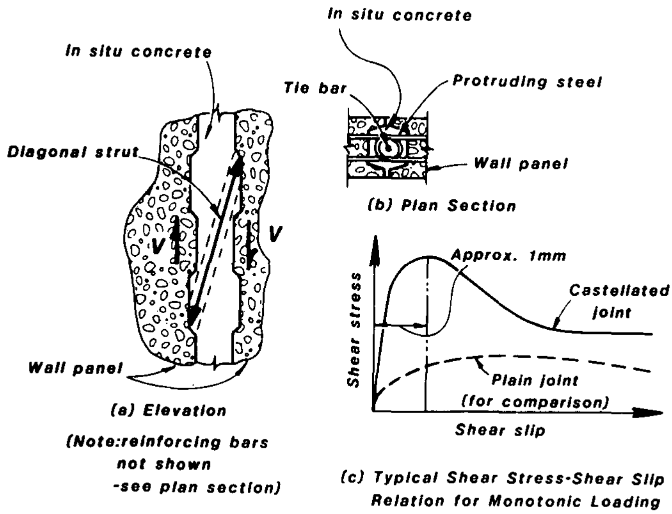


Fig. 18 Typical Castellated Joint Between Prefabricated Large Panels.

A significant number of tests have been conducted in the past on joints transferring shear force between prefabricated reinforced concrete panels when subjected to monotonic loading. Examples of such tests are those of Pommeret [18], Hansen and Olesen [19], Cholewicki [20], Tassios and Tsoukantas [21], and others. Some general conclusions from tests on joints transferring shear along their length are as follows:

- The preparation of the surfaces at the ends of prefabricated concrete panels to be connected is of major importance when considering the shear strength and possible shear slip at the interface. The shear strength of connections between panels with castellated end surfaces can be many times higher, and the slip at the shear strength significantly lower, than for connections between panels with plain end surfaces.

- Reinforcement perpendicular to the joint, adequately anchored inside the body of the prefabricated panels and connected in the joint in situ concrete by welding or looping, should cross the joint. The presence of such reinforcement increases the shear strength of the joint (due to the clamping effect) as well as its ductility, especially when the connecting bars are well distributed along the joint.

- Reinforcement parallel to the joint, placed in the joint in situ concrete, also increases the shear strength of the joint, particularly when the perpendicular reinforcement is connected using loops and mechanical devices.

- The shear strength of the joint is increased if a compressive normal stress exists across the joint, since such stress increases the clamping force across the joint. The permanent action of the normal stress should be ensured, however, if its benefit is to be taken into account in design.

A simplified model for the shear transfer in a keyed wet joint after cracking, according to Tassios and Tsoukantas [21,22] is shown in Fig. 19. Direct diagonal compression transfer between keys, friction and dowel action, are mobilised due to shear displacement at the interface between the prefabricated concrete panel and the joint in situ concrete. Taking into account a suitable reduction of the compressive strength of the concrete in the diagonal compression struts within the joint (due to compressive stress acting simultaneously with high shear stresses in the struts), and using

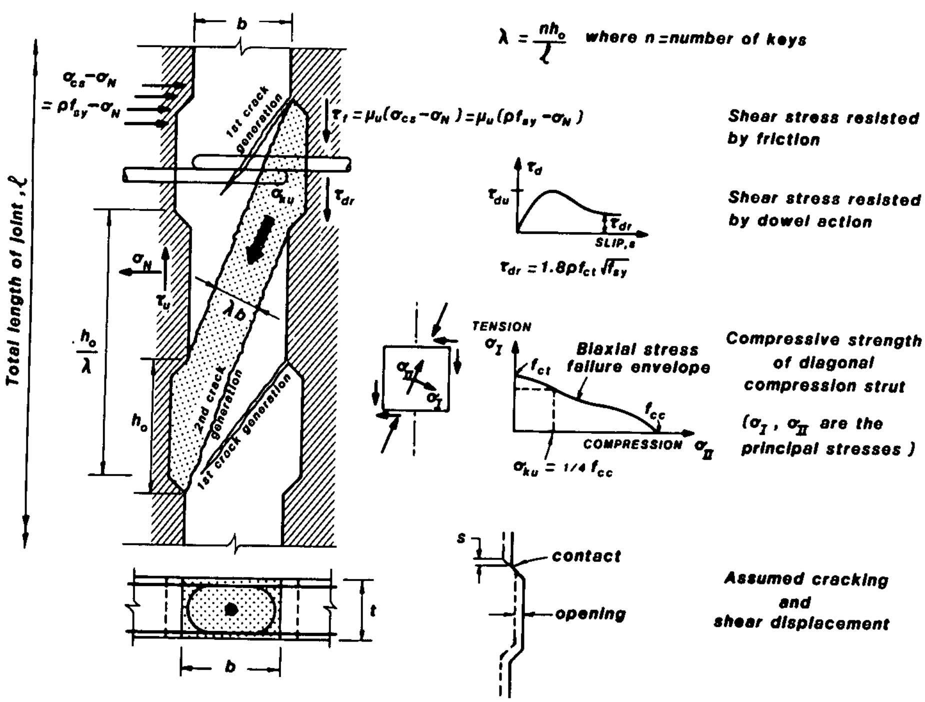


Fig. 19 Model for the Shear Transfer Within a Large Panel Reinforced Concrete Connection [21,22].

suitable models for the estimation of frictional resistance and dowel action, the following equation was proposed for the ultimate shear strength (stress) of shear joints provided with transverse reinforcement connected across the joint by means of loops [21,22]:

$$\tau_u = 0.25 \frac{b}{h_o} \lambda^2 f_{cc} + \mu_u (\rho f_{sy} - \sigma_N) + 1.8 \rho f_{ct} \sqrt{f_{sy}} \qquad (12a)$$

but not to exceed

$$\tau_u = 0.15 \lambda f_{cc} + \mu_u (\rho f_{sy} - \sigma_N) + 1.8 \rho f_{ct} \sqrt{f_{sy}} \qquad (12b)$$

where the notation (see also Fig. 19) is:

b = maximum joint width (in plane of panels), mm
h_o = length of each key at outside edge of panel, mm
λ = density of the keys = $n h_o / \ell$
n = number of keys in joint length
ℓ = length of joint, mm
f_{cc} = compressive strength of joint concrete, MPa
f_{ct} = tensile strength of joint concrete, MPa
f_{sy} = yield stress of the connecting transverse steel bars, MPa
μ_u = friction coefficient, taken as follows:

ρf_{sy}	1.0	1.5
μ_u	1.0 - 0.8	0.8 - 0.6

ρ = ratio of transverse reinforcement crossing the joint = $A_{st}/\ell t$
A_{st} = total area of transverse reinforcement crossing the joint, mm^2
t = joint thickness (perpendicular to plane of panels), mm
σ_N = normal stress due to external force acting on the joint in the plane of the panels, positive when tensile and negative when compressive, MPa.

In the right hand side of Eqs. 12a and b, the first term is the contribution from the diagonal compression struts within the joint, the second term is from the friction mobilised from the clamping action of the external forces and the transverse reinforcement, and the third term is the contribution from dowel action. Eq. 12b places a limitation on the shear transferred by the diagonal compression struts to take into account the possibility of shear failure of the keys in the joint.

Eqs. 12a and b have been presented as an example of the manner in which the mechanisms of shear transfer which contribute to the strength of the joint may be taken into account. Several equations for the shear strength of shear joints in the ultimate limit state, suitable for Code use, have been developed on the basis of test results in many countries. Part III of the Manual contains some specific design examples and equations in use in the Balkan region.

Some test data is also available which gives an indication of the level of shear transfer when diagonal tension cracking first commences in the joint. Assessment of that data [19,20,21] shows that first diagonal tension cracking may be expected to occur at a load of about one half to two thirds of the ultimate shear strength of the joint, and that the shear slip associated with that cracking load is in the order of up to 0.1 mm.

It should be noted, however, that this cracking load is very dependent on the treatment of the surfaces of the edges of panels to be connected.

5.2.4 Shear Joints Under Cyclic Loading

The behaviour of shear joints subject to cyclic loading is less explored than for monotonic loading. Nevertheless a number of tests have been carried out in Europe, mostly in the Balkan countries, as well as in North America.

Typical measured shear force - shear displacement hysteretic behaviour of shear joints is shown in Fig. 20. The degradation of shear strength, stiffness and energy dissipation with increasing number of load cycles is evident. The degradation of shear strength means, for example, that a reduced ultimate shear resistance V_3 should be considered after three postelastic load reversals, rather than the monotonic ultimate shear resistance V_1. Figs. 21 and 22 show some experimental data measured by Tassios and Tsoukantas [22] on large scale vertical joints under cyclic shear deformations, illustrating typical shear stress - shear slip characteristics and the degradation of ultimate shear resistance.

It is evident that in the design of earthquake resistant structures, the dependable shear strength of the joint found from monotonic tests should be reduced by means of suitable correction factors to take into account the effect of strength degradation due to cyclic loading [22].

5.2.5 Compression Joints

Compression joints are usually the horizontal joints in large panel structures and transfer normal forces and shear forces between adjacent panels. The stress state in a horizontal joint is extremely complex and experiments have been necessary to provide an insight into the joint strength and stiffness. According to the state of stresses a horizontal compression joint may be under normal compression along the whole of the length or under compression for only part of its length with tension over the remainder.

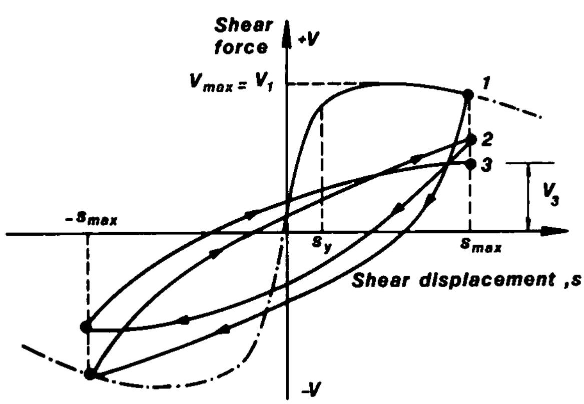


Fig. 20 Typical Shear Force - Shear Displacement Behaviour of a Large Panel Reinforced Concrete Connection Under Cyclic Shear Loading.

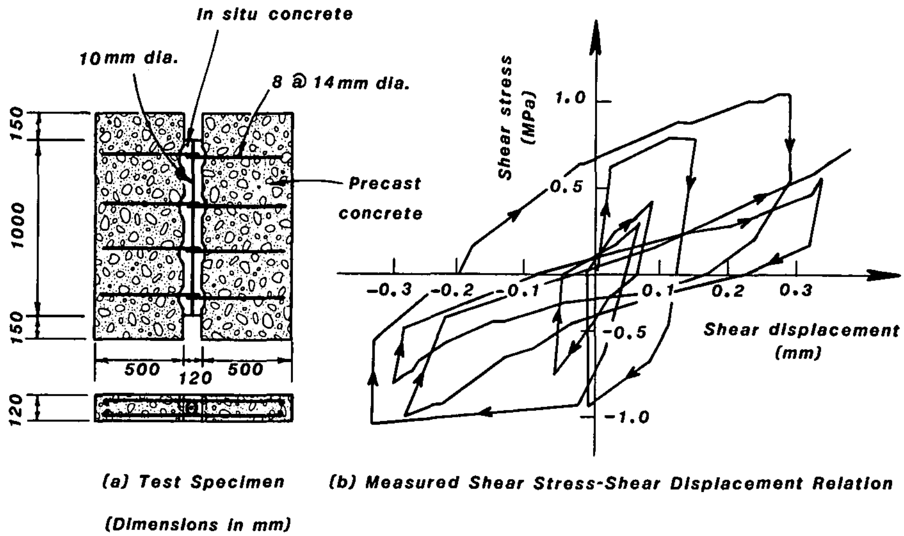


Fig. 21 Connections Between Large Panels and Measured Behaviour During Cyclic Shear Loading [22].

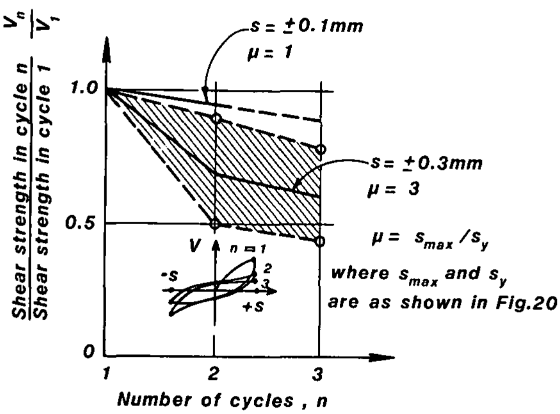


Fig. 22 Measured Degradation in Shear Strength of Large Panel Reinforced Concrete Connections During Cyclic Shear Loading to Shear Displacements of ± s [22].

An agreed rational method for the dimensioning of compression joints between large panels has not yet been developed at an international level. However, a CEB/CIB Task Group [23] is considering design approaches for joints in precast wall structures. Although still under discussion, draft proposals have been made concerning the dimensioning of horizontal joints. The draft proposals [23] are summarized below for the case where the joint is transferring ultimate design actions of moment M_d, shear force V_d and normal compressive force N_d.

(a) Horizontal Joint Under Compression Along the Whole of Its Length

Verification against shear:

(i) Plane joints, without keys and without reinforcement.

$$\text{If } \frac{N_d}{A_j f_{cd}} \leq 0.2, \quad \text{then } \frac{V_d}{A_j f_{cd}} \leq \frac{\beta_3}{\gamma_n} \text{ is required} \tag{13a}$$

or if

$$\frac{N_d}{A_j f_{cd}} > 0.2, \quad \text{then } \frac{V_d}{A_j f_{cd}} \leq \frac{\beta_3}{\gamma_n} + \frac{\beta_4}{\gamma_n}\left(\frac{N_d}{A_j f_{cd}} - 0.2\right) \not> 0.3$$

is required (13b)

where for smooth joints $\beta_3 = 0.13$, $\beta_4 = 0.34$
or for rough joints $\beta_3 = 0.18$, $\beta_4 = 0.47$
and N_d = design normal compressive action
A_j = area of joint in horizontal plane
f_{cd} = design compressive strength of concrete
V_d = design shear action
γ_n = correction coefficient > 1.0.

(ii) Reinforced joints, formed with keys.
If the conditions of Eqs. 13a or b are not fulfilled, fully anchored vertical reinforcement should be provided, connected across the joint, to satisfy the following relationship

$$V_d \leq \frac{1}{\gamma_n}\left(\beta_4 A_k \sigma_{cc,G} + \beta_2 A_s f_{yd}\right) \not> 0.3\,\frac{A_j f_{cd}}{\gamma_n} \tag{14}$$

where β_2 = 1.0, β_4 = 0.47
A_k = area of keys in joint
$\sigma_{cc,G}$ = mean compressive stress under N_{dG} due only to dead loads
A_s = total area of vertical reinforcement crossing the joint
f_{yd} = design yield strength of steel reinforcement.

(b) Horizontal Joint Under Compression Along Only Part of Its Length and Tension Along the Remainder

The distribution of normal stress in the concrete due to the design normal compressive action N_d and the design moment M_d may be found assuming linear elastic behaviour and an uncracked joint (see Fig. 23). The joint should always be keyed.

Verification against tension:
Vertical reinforcement of area $A_{s,t}$ made continuous through the joint by welding or some other appropriate method, should be provided near the most tensioned edge and accommodated in

appropriate keys. The area provided should satisfy the following relationship

$$A_{s,t} f_{yd} \geq 0.5 b_j \ell_t \sigma_{ct,u} \tag{15}$$

where $A_{s,t}$ = area of vertical reinforcement in tension region of the joint
b_j = joint thickness (perpendicular to plane of panels)
ℓ_t = length of joint in tension
$\sigma_{ct,u}$ = maximum normal tensile stress in the concrete.

Verification against shear:
In the tensioned region of the joint of length ℓ_t the area of vertical reinforcement connected through the joint should satisfy the following relationship

$$\frac{1}{\gamma_n} \left(\frac{10}{\pi} f_{cd} A_{s,t} \right) \geq \frac{\ell_t}{\ell_j} V_d \tag{16}$$

where ℓ_j = length of joint.

In the compression region of the joint of length ℓ_c the requirements of Eq. 14 should be respected for a design shear force of

$$V'_d = \frac{\ell_c}{\ell_j} V_d \tag{17}$$

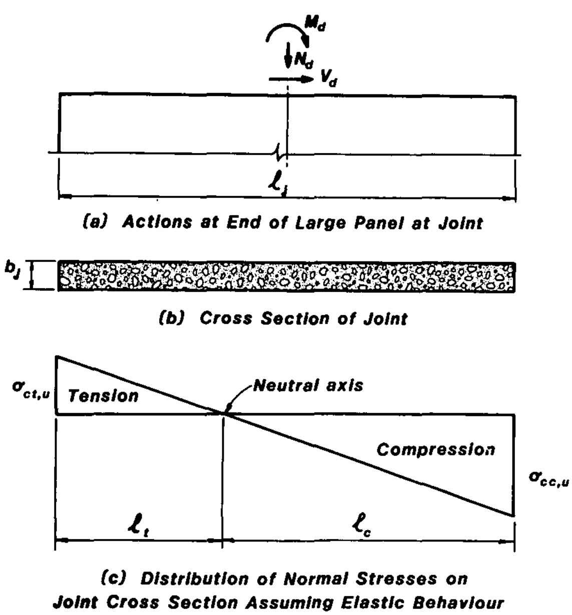


Fig. 23 Actions at End of Large Panel at Joint and Assumed Distribution of Normal Stress on Joint Cross Section for the Dimensioning of the Joint [23].

5.2.6 Typical Vertical and Horizontal Joints

A range of vertical and horizontal joints used between prefabricated reinforced concrete panel structures in the Balkan countries are illustrated in Part III of this Manual. For example, Fig. 24 shows two typical vertical joints. In Fig. 24a the reinforcing bars perpendicular to the joint are connected by overlapping loops; in Fig. 24b the reinforcing bars crossing the joint are connected by welding. In both cases vertical steel also exists in the in situ concrete.

Fig. 25 shows two typical horizontal joints. In Fig. 25a the horizontal bars from the floor panels are connected by overlapping loops; in Fig. 25b the horizontal bars are connected by welding. Fig. 25c illustrates a possible method of connecting the vertical bars by welding the bars together in recesses or pockets in the ends of the prefabricated elements.

An alternative to the types of horizontal joints shown in Fig. 25 is the "platform joint" shown in Fig. 26 which is commonly used in the USA and some other countries. In this joint the upper wall panel is seated on the edges of the floor panels below which in turn are seated on the lower wall panel. This requires the placing of in situ concrete or grout into a smaller joint cavity. The cast steel sleeve shown in Fig. 26a (available commercially) is apparently able to develop the strength of the bar without loss of ductility. This sleeve provides an excellent means of connecting bars and allows speedy construction. Bolted connections as shown in Fig. 26b have also been utilized. Platform joints have been widely used but appear to be best suited for countries without seismic design requirements.

Post tensioned bars connected by threaded couplers have been used in the USA and elsewhere as vertical ties. Prefabricated wall panels can be post tensioned together on the site to form continuous structures. Mortar joints between the panel ends can be used. This form of construction makes it easier to achieve continuity of steel through the connection.

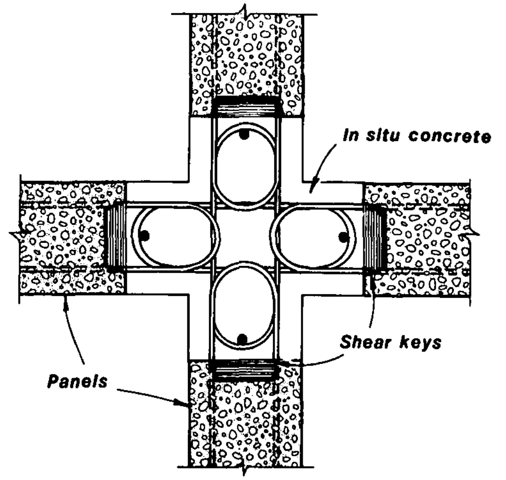
5.3 Dry Joints Between Prefabricated Panels

Some dry joints which have been used in the USA and other countries for vertical joints are shown in Fig. 27. The connection is made by welding or bolting steel shapes together which are anchored to the panels. Intensive quality control is necessary when site welding is used to ensure that the steel retains its strength and ductility, and that the adjacent concrete is not damaged by the high localized temperatures during welding. Care needs to be taken in the design of dry joints of this type since the forces are transferred from panel to panel at the discrete points where the connections are made and stress concentrations arise. Hence in the design of such joints it is necessary to ensure that localized failure does not occur in the concrete around the connection.

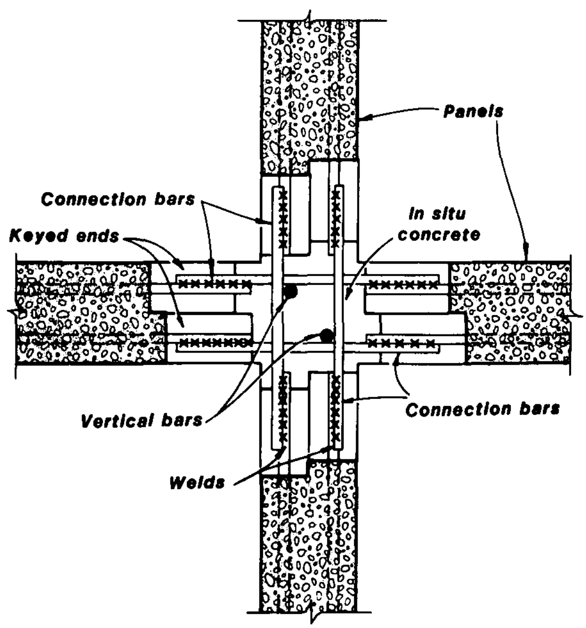
Many types of dry joints are adequate for gravity load design, but must be carefully assessed before being adopted for use in earthquake resistant design. It is difficult to ensure ductile behaviour with many dry joints.

5.4 Beam-Column Joints

Beam-column joints forming rigid connections have been designed and constructed using various arrangements of prefabricated concrete members and cast in situ concrete. One such concept is shown in Fig. 28. The shear stress at the interface of the prefabricated and cast in situ concrete in the beam needs to be checked in design and sufficient stirrups provided to ensure satisfactory shear transfer.



(a) Horizontal Bars Connected by Overlapping Loops



(b) Horizontal Bars Connected by Welding

Fig. 24 Typical Vertical Joints Between Prefabricated Large Panels.

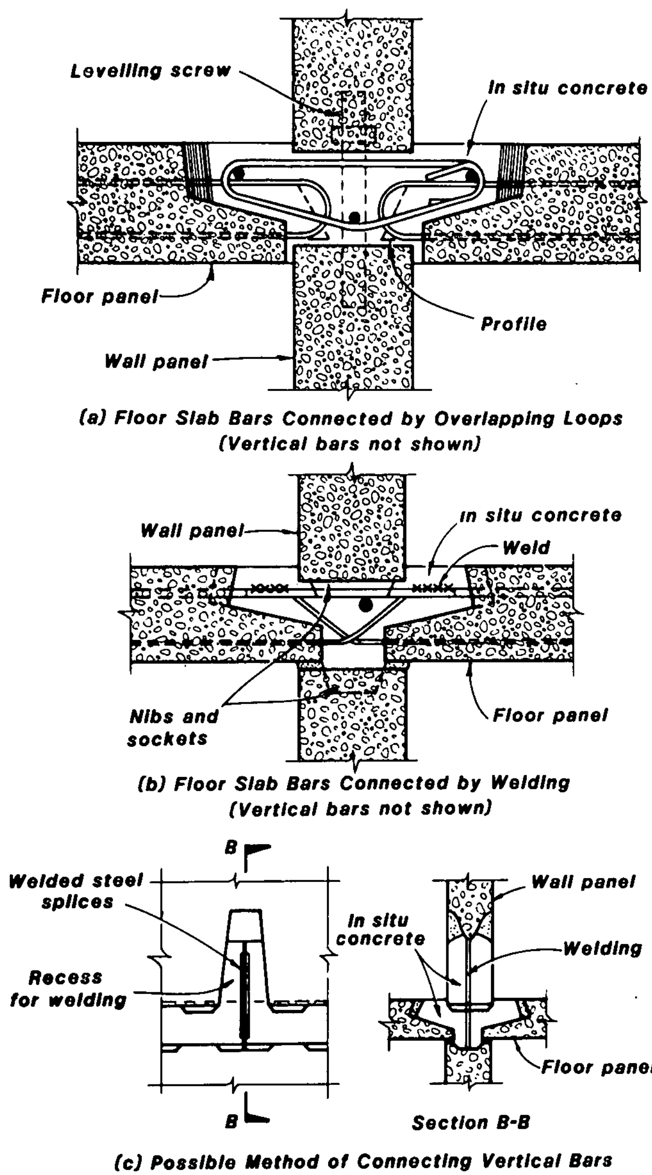


Fig. 25 Typical Horizontal Joints Between Prefabricated Large Panels.

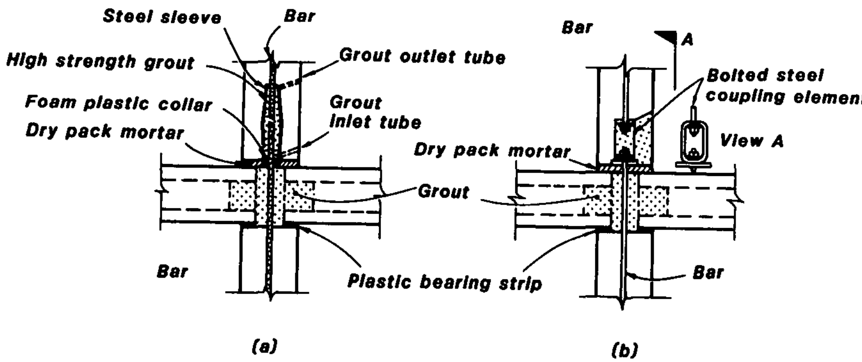


Fig. 26 Further Horizontal Joints for Prefabricated Wall Panels [24].

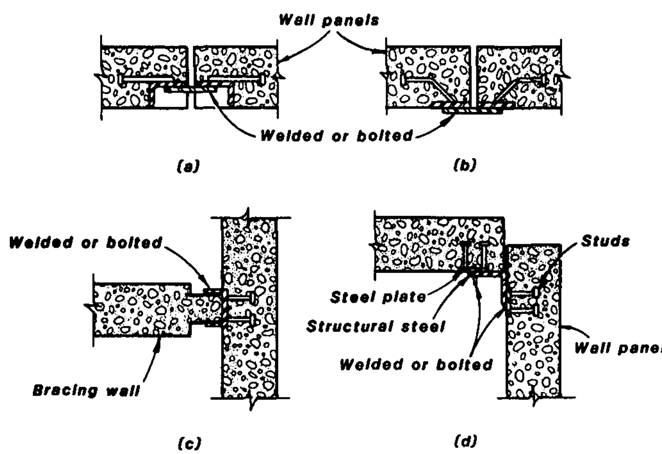


Fig. 27 A Range of Dry Vertical Joints for Prefabricated Wall Panels.

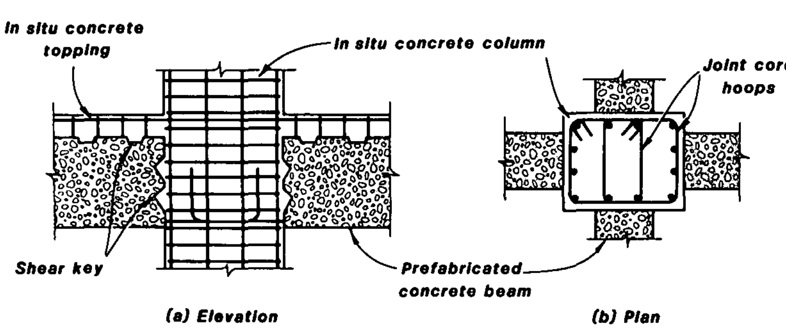
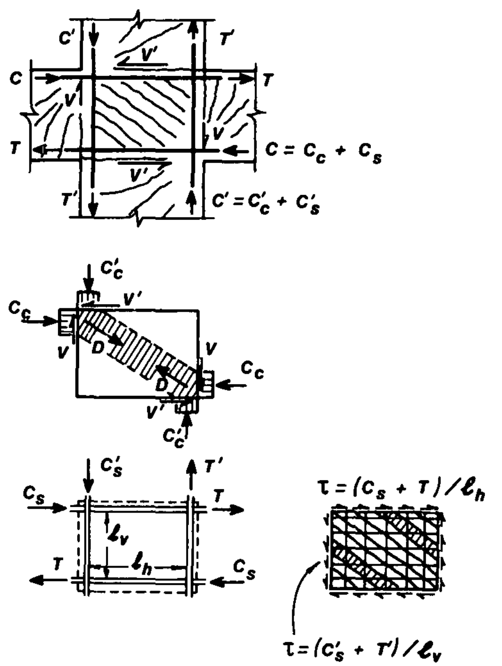


Fig. 28 Interior Beam-Column Joint.

The potential plastic hinge regions in the members need to be designed for adequate strength and ductility. Also, the joint core regions of beam-column connections need special attention because of the critical shear and bond stresses that can develop there during seismic loading. Fig. 29a shows a beam-column joint and forces to be transferred during seismic loading. The shear forces acting in the joint are resisted partly by a concrete compressive strut which acts between diagonally opposite corners of the joint (see Fig. 29b) and partly by a truss mechanism formed by the horizontal and vertical reinforcement and the concrete in the joint (see Fig. 29c). It has been observed during tests that when plastic hinges form in the beams adjacent to the joint during cyclic loading the contribution of the concrete diagonal compression strut to the joint shear resistance diminishes, because full depth cracks exist in the beams at the column faces [7]. Hence shear reinforcement for truss action is required in the joint to carry a significant proportion of the total shear force to be transferred. Note that truss action requires both the horizontal and vertical reinforcement to be present. That is, as well as horizontal hoops in the joint, vertical column bars are required between the corner bars to cross the joint region. Also, the diameter of the longitudinal beam and column bars should be limited to ensure that slip of those bars does not occur across the joint during cyclic loading.

Prestressing can also be used to achieve continuity between prefabricated concrete frame members. Beams and columns can be post-tensioned together on the site to form continuous seismic resistant frames. Combinations of prefabricated and cast in situ concrete members can be used. Draped (curved) post-tensioned tendons can be used in the beams to load balance a



(a) Joint and Forces to be Transferred

(b) Shear Transfer by Diagonal Compression Strut Action

(c) Shear Transfer by Truss Action of Shear Reinforcement

Fig. 29 Mechanisms of Shear Transfer in a Reinforced Concrete Beam-Column Joint [7].

substantial proportion of the gravity loads. At the joints at the ends of the beams the prestress may be close to concentric, with the tendons spread down the depth of the section in order to resist seismic load reversals. Prestressed connections generally perform well if care is taken in their detailing. The ductility and energy dissipation of a plastic hinge region in a prestressed concrete member can be significantly improved by the presence of non-prestressed longitudinal reinforcement [25].

5.5 Slab-Column Joints

Slab-column connections in prefabricated concrete structures have as their main function the transfer of vertical shear from the floors to the columns, because generally in such building systems the horizontal seismic loading is resisted mainly by frames or structural walls. However, it is advisable that the slab-column connections are made ductile, since some unbalanced bending moment may need to be transferred by the connections. The ductility of the connection is improved if the slab contains both top and bottom reinforcing steel passing through the column.

Some multi-storey slab-column building structures have been built without the presence of some frames or structural walls. However, without such stiffening elements considerable interstorey deflections may occur during earthquakes, resulting in serious non-structural damage. Also, although it is possible to design a slab-column connection to transfer bending moment, tests have demonstrated that during cyclic loading the connection undergoes a large reduction in strength and stiffness due to diagonal tension cracking in the slab. Hence the connection would not contribute significantly to energy dissipation during a severe earthquake. Therefore multistorey slab-column structures should not in general be used as seismic resistant structures without the presence of some frames or walls.

6. STRUCTURAL DESIGN CONCEPTS

6.1 Design Strength and Ductility

6.1.1 Ductility Requirements

Nonlinear dynamic analyses of code-designed multistorey 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 required of a structure is the displacement ductility factor μ defined as

$$\mu = \Delta_{max}/\Delta_y \qquad (18)$$

where Δ_{max} = maximum horizontal deflection of the structure during severe earthquake shaking, 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 at first yield.

In the case of an elasto-plastic system the deflection at first yield, Δ_y, is well defined. However, in reality, most structural systems do not behave elasto-plastically. Typically, in a multistorey structure the horizontal load-deflection relation is curved due to the gradual development of yielding throughout the structure as the horizontal load is increased. Therefore, a definition for the deflection at first yield which applies to the general case needs to be established. One possibility is to define the deflection at first yield as that deflection calculated assuming elastic behaviour up to the ultimate horizontal load of the structure.

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. 30a. Dynamic analyses have also indicated that for structures with a short fundamental period of vibration a better approximation is given by the "equal maximum potential energy" response illustrated in Fig. 30b, which requires the area OCD to 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 Figs. 30a and b) is in fact the behaviour factor K used in Eq. 3. Table 1 shows the values for the behaviour factor K for monolithic structures recommended by the CEB-FIP Seismic Appendix [1]. It is evident that the equal maximum deflection assumption of Fig. 30a means that

$$K = \mu \qquad (19)$$

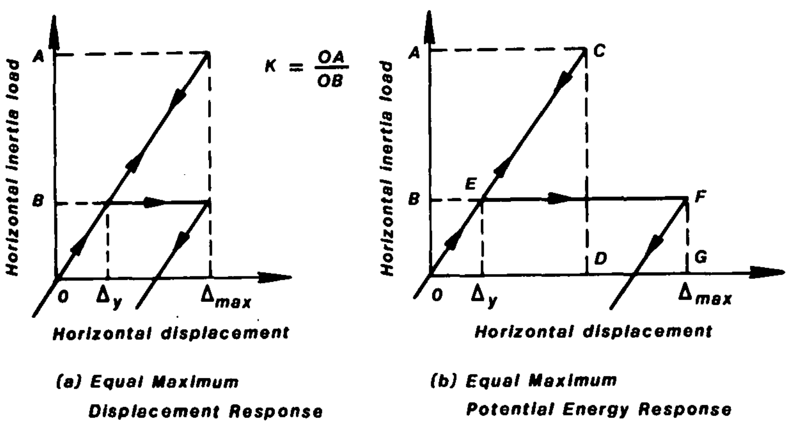


Fig. 30 Assumed Responses of Elastic and Elasto-Plastic Systems to Earthquake Ground Shaking.

The equal maximum potential energy assumption of Fig. 30b may be shown [7] to mean that

$$K = \sqrt{2\mu - 1} \tag{20}$$

A comparison between the values of μ obtained from Eqs. 19 and 20 for the various values of K used in Table 1 are shown in Table 2.

Table 2 : Relationship Between K and μ

$K = \dfrac{\text{Elastic Response Load}}{\text{Design Load}}$	2	3	3.5	4	5
μ from Eq. 19	2	3	3.5	4	5
μ from Eq. 20	2.5	5	6.6	8.5	13

The higher value for μ required for each K value given by Eq. 20 is only expected to apply to short period 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 required at a plastic hinge in a yielding structure may be expressed by the curvature ductility factor ϕ_{max}/ϕ_y, where ϕ_{max} 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_{max}/ϕ_y at plastic hinge sections will generally be much greater than the required Δ_{max}/Δ_y value for the structure, since once yielding commences further displacement occurs mainly by rotation at the plastic hinges. This aspect of behaviour in the yield range is discussed further below.

6.1.2 Achieving Adequate Ductility in Structural Systems

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 behaviour of complete structures is still open to question. Hence it is impossible to evaluate all aspects of the complete behaviour 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 behaviour. 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 chose 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 developes. This procedure will ensure (as far as possible) that yielding will occur only in the chosen manner during a severe earthquake.

6.1.3 Preferred Mechanisms of Energy Dissipation for Monolithic Structural Systems

The seismic response of a monolithic cantilever structural wall should be dominated by flexure if the greatest energy dissipation is to be achieved. That is, shear, bond, instability or any other type of failure should not occur before the flexural strength is reached at the base of the wall. Thus the ideal mechanism of inelastic deformation involves a plastic hinge at the base (see Fig. 31a) and the required curvature ductility factor, ϕ_{max}/ϕ_y, for a given displacement ductility factor, μ, depends very much on the plastic hinge length as a proportion of the wall height. The value of the required curvature ductility factor, ϕ_{max}/ϕ_y, can be much higher than the required displacement ductility factor, μ, particularly when the plastic hinge length is a small proportion of the wall height [7]. If the equivalent plastic hinge length is equal to or greater than 0.1 of the height of the wall, the value of the available ϕ_u/ϕ_y provided at the base of the wall should be at least 3μ, where ϕ_u is the available ultimate curvature and ϕ_y is the curvature at first yield at the critical section.

For monolithic coupled structural walls, the best energy dissipating mechanism is when flexure dominates and plastic hinges form as shown in Fig. 31b. Ideally the coupling beams should yield before the walls at the bases so as not to impair the gravity load carrying function of the wall. Other types of failure should be avoided. The mechanism illustrated in Fig. 31b is probably the best method for dissipating energy of all structural types. The value of the available ϕ_u/ϕ_y provided by the coupling beams should be at least 3μ, where ϕ_u is the available ultimate curvature and ϕ_y is the curvature at first yield at the critical section.

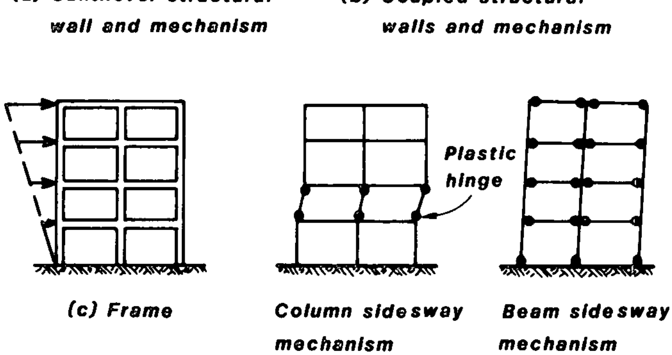


Fig. 31 Building Structures With Horizontal Seismic Loading and Possible Mechanisms.

For frames, mechanisms which involve flexural yielding at plastic hinges are shown in Fig. 31c. If yielding commences in the columns of a frame before the beams, a column sidesway mechanism can form. In the worst case the plastic hinges may form in the columns in only one storey, since the columns of the other storeys are stronger. Such a mechanism can make very large curvature ductility demands on the plastic hinges of the critical storey [7], particularly for tall buildings. The curvature ductility required at the plastic hinges of a column sidesway mechanism may be so large that it cannot be met and in that case collapse of the structure will occur. On the other hand if yielding commences in the beams before in the columns, a beam sidesway mechanism, as illustrated in Fig. 31c, will develop [7], which makes more moderate demands on the curvature ductility required at the plastic hinges in the beams and at the column bases. The curvature ductility demands at the plastic hinges of a beam sidesway mechanism can be met by careful detailing. Therefore for tall frames a beam sidesway mechanism is the preferred mode of inelastic deformation and a strong column-weak beam concept is advocated to ensure beam hinging. For frames with less than about three storeys, and for the top storey of tall frames, the curvature ductility required at the plastic hinges if a column sidesway mechanism develops is not particularly high. Hence for one and two storey frames, and in the top storey of taller frames, a strong beam-weak column concept can be permitted. The required ϕ_{max}/ϕ_y values at the plastic hinge regions in a strong column-weak beam design of a framed structure will depend on the geometry of the members. However it would seem that an available ϕ_u/ϕ_y of at least 3μ should be provided in the potential plastic hinge regions. Where a strong beam-weak column design is permitted (in one or two storey frames or the top storey of taller frames) an available ϕ_u/ϕ_y of at least 3μ (or possibly greater) should be provided in the plastic hinge regions of columns. Shear and bond failure should always be avoided.

The static collapse mechanisms of Fig. 31 are idealized in that they involve behaviour under code type equivalent static loading. The actual dynamic situation is different, due mainly to the effects of higher modes of vibration, but nevertheless considerations such as in Fig. 31 give the designer a reasonable feel for the situation.

The above recommended values for the required curvature ductility factor ϕ_{max}/ϕ_y which should be available are only reasonable approximations. More accurate static collapse mechanism analyses to determine the required values can be carried out taking into account the many variables involved, such as the actual member geometry and relative flexural strengths of sections. In important cases of some unusual structures it may be necessary to use time-history nonlinear dynamic analysis of the structure responding to severe earthquakes to obtain a better indication of the required curvature ductility at the critical plastic hinge sections.

6.1.4 Preferred Mechanisms of Energy Dissipation for Prefabricated Structural Systems

Connections between prefabricated elements can constitute the weakest regions in prefabricated concrete structures and if so will be where the demand for ductility will concentrate during a severe earthquake. If the strength of a connection is reached during a severe earthquake its behaviour under cyclic loading in the inelastic range may be suspect. Ideally, if there is doubt about achieving adequate ductility in the connection, the connection detail should be made intentionally overstrong thus forcing the inelastic strains to occur away from the connection in a region of the element which can be more readily detailed for ductility.

However, in practice it is often difficult to avoid yielding in some connection regions during severe earthquake loading. Connection yielding and slippage can produce a mechanism for energy dissipation but care should be taken in such a case that adequate ductility can be achieved.

Where structural walls are constructed from prefabricated panels the design of the vertical and horizontal reinforcement for the flexural and shear strength of the panels and joints should be such as to ensure that the desired sequence of plastification occurs during a severe earthquake. There are at least three ways of approaching the seismic design of wall panel systems [24]. These three approaches will be discussed with reference to cantilever structural walls without openings.

Equivalent monolithic design: The wall panels and connections can be detailed so as to achieve monolithic (or near monolithic) behaviour so that flexural yielding occurs as illustrated in Figs. 11a and 31a with a plastic hinge at the wall base as the primary mode of energy dissipation. This approach requires vertical and horizontal joints strong enough to resist without significant shear displacements the forces associated with the formation of a plastic hinge near the base of the wall. Then detailing principles for ductile cast in situ cantilever walls can be followed.

Weak vertical joints and strong horizontal joints: The vertical joints can be designed to provide a mechanism to dissipate energy by vertical (sliding) shear displacements (see Fig. 11b). This mechanism is analogous to the yielding of coupling beams in coupled structural walls. Yielding of vertical joints does not endanger the stability of the structures, since such joints do not resist actions due to gravity loading. Also, when the horizontal seismic action is increased, yielding spreads from the first point of yield to other parts of the vertical joint and finally to the walls, rather than just concentrating at the first point of yield. Therefore the spread of damage caused by a severe earthquake is gradual as the intensity of loading increases. The vertical joint should have stable vertical shear stress-shear slip hysteresis characteristics in order to function as an effective energy dissipating medium. Thus the vertical joints should be suitably detailed. The horizontal joints and the panels themselves should be made suitably strong to ensure that yielding commences first in the vertical joints. Limited-slip bolted joints, involving anchored steel inserts which are connected by friction bolted steel plates with slotted holes, have shown substantial promise as a means for improving the energy dissipating characteristics of vertical joints [26]. A vertical joint detail which could incorporate friction grip bolts is shown in Fig. 27a.

Weak horizontal joints and strong vertical joints: A further option is to provide energy dissipation by shear (sliding) displacements along horizontal joints (see Fig. 11c). This mechanism of energy dissipation is not considered desirable. It results in severe degradation of strength and stiffness of the structure and the damage will occur to a joint which also has the role of carrying the gravity loads. Note that sliding along a horizontal joint in effect is an independent mechanism in that the subsequent yielding of vertical joints or other horizontal joints, or the walls themselves, cannot occur because the actions in the structure will be limited to the capacity of the critical horizontal joint. (In the case of weak vertical joints, further horizontal load can be carried by the structure as yielding spreads along the full height of the vertical joint and into the panel base). Sliding along softened horizontal joints can result in large displacements and can be an unconstrained mechanism.

Of these design approaches, the weak horizontal joint-strong vertical joint approach is definitely undesirable and should not be used. The weak vertical joint-strong horizontal joint approach should only be considered if vertical joints with proven stable shear stress-shear slip hysteresis characteristics can be designed. Generally the best design approach is to seek monolithic behaviour.

Where structural walls are constructed from prefabricated panels with openings, the coupling beams or lintels connecting the parts of the walls separated by openings can be detailed as in monolithic walls to be the major energy dissipating regions of the structure during severe earthquakes.

For prefabricated frames the preferred mechanisms of energy dissipation are as for monolithic frames. That is, a strong column-weak beam concept should be used in tall frames. Column sidesway mechanisms with plastic hinges forming in the columns of just one storey should only be permitted in frames with less than about three storeys. Connections between prefabricated members should be placed away from the critical potential hinge regions of the frames if possible. If the connections between members are placed in critical regions it should be ensured that continuity of flexural reinforcement exists through the joint by properly tested details involving welding, mechanical connectors or the anchorage of bars in in situ concrete, so that the behaviour of the connection approaches that of a monolithic structure.

6.1.5 Concurrent (Oblique) Earthquake Loading

Earthquake ground motions occur in random directions, but it has been the practice in seismic design to consider the seismic loading to act only in the directions of the principal axes of the building and only in one direction at a time. In fact a general angle of seismic loading can produce a very severe condition in a building structure. For example, it may make it extremely difficult to prevent the formation of plastic hinges in columns of building frames in the general case of loading [7].

The effects of concurrent (oblique) earthquake loading can be illustrated with respect to the symmetrical framed building structure with plan as in Fig. 32a subjected to horizontal seismic loading in a general direction. Let a floor of the building deflect horizontally in the direction of the earthquake action as in Fig. 32b. It is evident that the angle θ need not be very large before yielding will be enforced in the beams in both directions. For example, if a displacement ductility factor $\mu = \Delta_{max}/\Delta_y$ of 4 is reached by the symmetrical structure of Fig. 32 in direction 2, it only requires $\Delta_1 = \Delta_2/4$ to cause yielding in direction 1 as well. Thus for a displacement ductility factor of 4, the earthquake action need only be inclined at an angle $\theta = \tan^{-1} 0.25 = 14°$ to one principal axis of the building to cause yielding in the beams in both directions of a symmetrical building. Therefore yielding in both directions may occur simultaneously for much of the loading.

The simultaneous yielding of beams in both directions of frames has the following effects:
(a) The resultant bending moment from the beam system applied to the columns is increased and the flexural strength of the columns is reduced. For example, if the beams in Fig. 32a have equal flexural strength (ΣM_u total) in each direction, the resultant moment from the beams applied about the column diagonal ($\sqrt{2}\Sigma M_u$) will be 41% higher than for beams with moment in one principal direction only. In addition the flexural strength of the column when bending about a diagonal is less than when bending about one principal

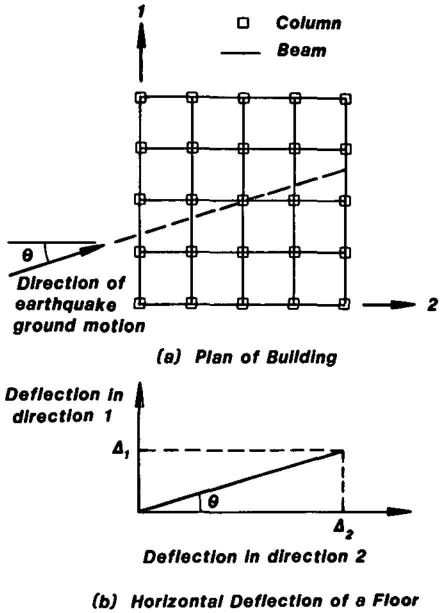


Fig. 32 General Direction of Earthquake Loading on a Building.

axis only (about 15% less). This increased column moment combined with the reduction in column flexural strength may result in plastic hinges forming in the columns before forming in the beams, leading to a column sidesway mechanism.

(b) The resultant shear force acting on the columns and joint cores will be higher than for frames with beams yielding in one direction only.

Similarly, walls and foundations may be affected adversely by the results of concurrent earthquake loading.

Designers should be mindful of the possible effects of concurrent earthquake loading when considering the relative strengths of members necessary to achieve the preferred mechanism of energy dissipation.

6.2 General Structural Design and Detailing

6.2.1 The Aim of Detailing Reinforcement

The importance of good detailing of steel reinforcement in earthquake resistant concrete structures cannot be overemphasized. Significant protection against damage will be provided by carefully detailed reinforcement. As well as serving the usual function of providing resistance to tensile and compressive forces in concrete elements and their connections arising from bending, shear and normal forces, steel reinforcement is necessary to prevent compressed bars from buckling and to provide confinement to concrete in highly stressed areas of compression. Detailing should be based on a thorough understanding of the behaviour of reinforced concrete, thus ensuring that the requirements of all the internal forces in the structure are considered from the point of view of serviceability, strength and ductility of the structure.

6.2.2 Structural Design and Detailing of Wall Systems

In eastern European countries the traditional practice is to detail large panel wall systems so as to achieve as nearly as possible, monolithic behaviour. In general, walls may have various sections such as rectangular or flanged. It is necessary to define in advance where the development of nonlinear deformations is allowed. In some cases it may be possible to design a wall structure with the lower part, where inelastic deformations are expected, constructed of cast in situ concrete.

A critical consideration in design is the choice of the types of connection to be used, where they are to be placed, and what is to be expected from them. One of the factors to be considered is that wide joint regions of in situ concrete, and/or large numbers of joints in the structure, will lead to increased flexibility of the structure. Also, only those connections providing the desired strength and ductility should be used, as found from experimental tests conducted on the connection details. All connections should be capable of withstanding cyclic loading in the inelastic range. The extent of inelastic deformations imposed on a connection will depend on the extent to which the connection is part of the energy dissipating mechanism of the structure. For stability, it is recommended that at vertical joints between wall panels at least one panel must be at right angles to the other panel or panels.

In general, the design of sections of earthquake resistant wall structures should be carried out using theory for the ultimate limit states of flexural and shear resistance rather than approximate elastic theory allowable stress approaches. Several codes contain design provisions for earthquake resistant wall structures. As one source, the CEB-FIP Seismic Appendix [1] contains specific requirements and design principles for monolithic walls which can be used for many aspects of the design of prefabricated concrete walls. These requirements include the design actions, geometrical constraints limits for longitudinal and transverse reinforcement contents, including special transverse (confining) reinforcement, shear strength requirements, coupling beam details, and bar anchorage, for the various ductility levels. That Appendix also recommends values for the dynamic amplification factor, ω, by which the design horizontal shear force obtained from equivalent static analysis should be multiplied to take into account the dynamic magnification due to higher modes of vibration, which are of particular importance for tall structures.

For walls without openings, where monolithic behaviour is sought, and detailing is to Ductility Level III, the design actions for horizontal and vertical shear should be compatible with the actual flexural strength that develops at the wall base when plastic hinging occurs there. Hence the vertical and horizontal design shear force due to code loading should be multiplied by an amplification factor γ_n (in additional to multiplying by the ω factor) given by

$$\gamma_n = \frac{M^+_{ud}}{M_d} \qquad (21)$$

where M^+_{ud} = flexural strength of section based on the actual area of reinforcement present and the probable strength of that reinforcement.

M_d = design moment obtained from code loading.

Where yielding at vertical joints is sought as part of the energy dissipating mechanism only the design actions for horizontal shear should be multiplied by γ_n.

Note that M^+_{ud} in Eq. 21 should be found using theory for the ultimate limit state of flexural resistance. M^+_{ud} cannot be found accurately assuming elastic theory for section behaviour.

6.2.3 Structural Design and Detailing of Frames

In general the design of member sections of earthquake resistant frame structures should be carried out using theory for the ultimate limit states of flexural and shear resistance rather than approximate elastic theory allowable stress approaches. Several codes contain design provisions for earthquake resistant frames. As one source, the CEB-FIP Seismic Appendix [1] contains specific requirements and design principles for monolithic frames which can be used for most aspects of the design of prefabricated frames. The requirements include design actions, geometrical constraints, limits for longitudinal and transverse reinforcement contents, including special transverse (confining) reinforcement, shear strength requirements, beam-column joints, and bar anchorage, for the various ductility levels.

One of the basic requirements in the design of ductile frames is that the possibility of plastic hinges forming in the columns, leading to a column sidesway mechanism in one storey of tall frames, is minimized. Hence, ideally, the column moments obtained from code loading need to be amplified to take into account the combined effect of: (a) the probable flexural strength of the beams at the plastic hinges (which will generally be greater than the design moment since the actual area of reinforcement present is usually larger than necessary and the actual strength of the reinforcement is normally higher than the characteristic yield strength), (b) concurrent (oblique) earthquake loading which may cause yielding in beams in both directions simultaneously in two-way frames, and (c) higher modes of vibration which can cause the points of contraflexure to shift well away from the locations indicated by analysis for static code loading [7]. Thus the design column moments due to code loading should be multiplied by $\omega\gamma_n$, where ω is the dynamic amplification factor which takes into account the effect of higher modes of vibration and concurrent earthquake loading, and γ_n is an amplification factor given by

$$\gamma_n = \frac{M^+_{ud}}{M_d} \tag{22}$$

where M^+_{ud} = flexural strength of the beam plastic hinge section based on the actual area of reinforcement present and the probable strength of that reinforcement.

M_d = design moment at beam plastic hinge section obtained from code loading.

Also, for the design of ductile frames the design actions for shear in beams should be those associated with the development of the probable flexural strength at the plastic hinges. Hence the shear forces in beams obtained from code loading should be multiplied by an amplification factor γ_n which is intended to ensure that brittle shear failures do not occur before the flexural strength at the chosen plastic hinge sections is reached. The values of γ_n, used to amplify moments when calculating shear forces from those moments, are given by Eq. 22. For columns the design actions for shear should be based on the moment gradient associated with the development of plastic hinges in the adjoining beams and columns where plastic hinges are expected. Similarly, the design shear force on beam-column joints should be based on the actual area of flexural steel in the beams and the probable strength of that steel.

The CEB-FIP Seismic Appendix [1] recommends values for the dynamic amplification factor, ω, by which the column moments, for regular frames three storeys or higher, found from equivalent static analysis should be multiplied to take into account the dynamic magnification of column moments due to higher modes of vibration and concurrent (oblique) earthquake loading. That Appendix also lists values for the amplification factor, γ_n, for frames designed for Ductility Level III, by which the design actions should be multiplied when calculating the design shear forces. The recommended values for γ_n are used to amplify the bending moments when calculating the shear forces from those moments. The recommended γ_n values are 1.25 for shear force in beams, 1.10 for shear force in columns (increased to 1.25 if column hinging is permitted to occur), and 1.15 for shear force in beam-column joints. The likelihood of plastic hinges occurring in the columns is eliminated as far as possible by designing the columns for the bending moments amplified by the dynamic amplification factor and also be ensuring that at any beam-column joint the sum of the column flexural strengths is at least 1.15 times the sum of the flexural strengths of the beams framing into that joint. The above recommended γ_n values from the CEB-FIP Seismic Appendix should be regarded as minimum values since analysis from first principles will often indicate that higher values may be necessary.

6.2.4 Diaphragms

A horizontal element of a building which transfers the horizontal seismic forces to the walls and/or columns of the structure is referred to as a diaphragm. Generally the floor slabs and roof are expected to act as diaphragms, distributing the horizontal seismic effects of the gravity loads on their own surface to the vertical seismic load resisting elements (walls and/or columns) of the structure. It is usually considered in design that diaphragms have infinite stiffness when transferring actions in their own plane. In some cases the effect of diaphragm flexibility may need to be taken into account.

The following conditions should be satisfied by a floor formed from prefabricated elements when providing diaphragm action:

- The floor should constitute a plane.
- The connections between adjacent prefabricated floor elements should be capable of providing the shear transfer necessary between elements from the point of view of both strength and stiffness.
- The connection between the prefabricated floor elements and the vertical elements to which the horizontal loads are transferred should have adequate strength and stiffness. Shear keys or mechanical connections can be used with appropriate reinforcement to form such connections.
- Tensile forces due to deep beam deformations of the diaphragm between the vertical structural elements should be taken by specially designed tensile reinforcement placed at the perimeter of the floor in each direction.

6.3 Foundations

The foundation system should be capable of supporting the design gravity loads while maintaining the chosen seismic energy dissipating mechanisms of the structure above. Foundations should generally be of cast in situ reinforced concrete construction in the form of continuous strip footings in two directions, or slabs. The load bearing areas should be determined in such a way that the soil strains remain essentially within the elastic range; that is, without appreciable residual deformations. For a relatively rigid structure, as for example wall systems, the effect of soil deformations should be taken into account in the assessment of the stiffness of cantilever walls when resisting horizontal seismic actions. In the case of pile foundations the piles should have adequate strength to sustain the forces transmitted from the structure above and adequate ductility to withstand the curvatures resulting from the horizontal seismic actions.

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PART II

REGIONAL REPRESENTATIVE DESIGN EXAMPLES

1. REPRESENTATIVE EXAMPLE OF LARGE PANEL SYSTEM

BULGARIA

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1. UNIFIED INDUSTRIALIZED LARGE PANEL SYSTEM

1.1 SCOPE

The present example deals with earthquake resistant design of a prefabricated large panel system developed in Bulgaria. Structural analysis of an eight storey building is presented. Computation of earthquake forces, internal forces and proportioning of a selected shear wall are done in accordance with the National Design Codes.

1.2 DESCRIPTION OF THE SYSTEM

The unified industrialized large panel system is to replace the undesired variety of the panel systems used in Bulgaria. (The abbreviation for this system in Bulgarian is ESS and will be used further in the text).

The unified building system is designed for construction of residential and public buildings in seismic areas with intensity up to IX. The building height is up to 9 storeys.

The ESS is a cell type structure designed for small wall distances. The floor panels are designed as one-way or two-way spanning elements.

The ESS is an open type system, i.e. it is not designed for a particular type of a large panel building. This system consists of generalized prefabricated elements with a wide range of specific parameters (fig.1.1).

The catalogue includes all types of elements. The designer chooses a different set of elements from the catalogue for each particular type of building. Thus, each plant operates with a given element set. When a new type of building is designed the plant produces the new set of elements without additional equipment. All elements of a given family can be produced in one cast form.

The system is designed for square modulus of 60 cm. Investigations have shown that the most reasonable spans are 3.00 m and 3.60 m in one of the main directions and from 1.20 m to 7.20 m by 60 cm in the other direction. The maximum dimensions of the elements are 3.60 m x 6.00 m. The storey height of the building is 2.80 m.

The floor structure is composed of individual floor panels with dowel connections of two ϕ 10 bars at intervals of 60 cm (fig. 1.2).

The floor panels are supported on the wall panels only by their studs. The rigidity of the floor diaphragms is provided by welding of the steel connections and cast-in-place concrete in the dowels. The floor panels are 14 cm thick and are designed as one-way or two-way spanning elements.

The interior bearing wall panels are 14 cm thick (fig.1.3). The horizontal forces are borne mainly by shear walls composed of interior shear-bearing wall panels (fig. 1.4).

In these panels the horizontal forces are transferred through special key connections of embedded steel I profiles located at 60 cm intervals at the modulus axes. Fig. 1.5 shows an isometric view of an interior and a facade panel as well as a floor panel.

Shear walls can be formed by one or more panels. The vertical joints between them are realized by welding of the reinforcing bars,and cast-in-place concrete in the dowels.

Reinforcement bars throughout the total height of the interior panels are provided to carry the tension forces in the vertical shear walls (fig. 1.4). The reinforcement area is proportioned for the maximum internal forces due to vertical and horizontal loadings.

The facade panels in the system are designed either as three-layered (sandwich) panels composed of two layers of C 20 concrete and a thermoinsulation of penopolystirol or as one-layered panels of light-weight concrete C 10. The facade panels with openings are designed for vertical load only.

The facade panels without openings serve as shear walls and are designed to carry both vertical and horizontal loads (fig. 1.6 and 1.7). Therefore, they are reinforced similarly to the interior panels.

1.3 STRUCTURAL ANALYSIS

An eight storey residential building is presented as a design example. It is situated in a zone of design intensity of VII according to MSK scale. The local soil conditions are classified in group 4 in accordance with the Bulgarian Code for Buildings in Earthquake Regions - 1964 (BCBER) |1| . The corresponding seismic design coefficient to the given intensity and soil conditions is $K_c = 0.033$.

The foundation and the walls of the basement are designed to be constructed by cast-in-place reinforced concrete. A typical floor panel of the building is shown on fig. 1.6. The plan of the first storey is different from that of the other storeys, but the vertical bearing elements (shear walls) are located at the same place and are not interrupted. The roof system is designed with two slabs at a distance of 1.50 m. The arrangement of the shear walls is shown on fig. 1.7. The shear walls are with a constant cross-section along the height and their geometric characteristics are given in table 1.1.

The dynamic model of the building is assumed to be a fixed-base cantilever beam with lumped masses at the level of the floor slabs.

1.3.1 Design Loads

According to the BCBER the design gravity loads considered in the computation of the horizontal earthquake forces include the specified total weight of the structure and the permanent equipment, 50 per cent of the live load and the effective weight of the snow load. The load factors are:

- dead load of all permanent structural and nonstructural components - $n = 1.0$.
- live load on the floor slabs - $n = 0.5$.
- snow load - $n = 0.8$.

The computed design load forces concentrated at the storey levels are given in table 1.2.

Geometric Characteristics of the Shear Walls

Transversal Direction — Table 1.1

SHEAR WALLS	LENGTH |m|	THICKNESS |m|	X |m|	Y |m|	STOREYS
SW 1.1	4.040	0.14	0.00	-	8
SW 1.2	4.040	0.14	12.00	-	8
SW 2.1	4.970	0.14	3.00	-	8
SW 3.1	5.540	0.14	6.60	-	8
SW 4.1	3.740	0.14	9.00	-	8
SW 5.1	5.090	0.14	9.00	-	8
SW 5.2	5.090	0.14	16.20	-	8
SW 5.3	5.090	0.14	18.00	-	8
SW 6.1	6.140	0.14	13.20	-	8

Longitudinal Direction

SHEAR WALLS	LENGTH |m|	THICKNESS |m|	X |m|	Y |m|	STOREYS
SW 7.1	4.640	0.14	-	0.00	8
SW 7.2	4.640	0.14	-	19.20	8
SW 7.3	4.640	0.14	-	21.00	8
SW 8.1	5.540	0.14	-	3.00	8
SW 8.2	5.540	0.14	-	9.60	8
SW 8.3	5.540	0.14	-	11.40	8
SW 8.4	5.540	0.14	-	16.20	8
SW 8.5	5.540	0.14	-	18.00	8
SW 9.1	4.490	0.14	-	6.00	8
SW 9.2	4.490	0.14	-	13.20	8
SW10.1	3.960	0.14	-	9.60	8
SW11.1	3.740	0.14	-	9.60	8

1.3.2 Computation Techniques

The lateral seismic forces are computed in accordance with the following formula (BCBER).

$$S_{ik} = \beta_i \eta_{ik} K_c Q_k \tag{1.1}$$

where: S_{ik} - the design seismic force, applied at point k for the i-th mode of vibration; β_i - the spectral dynamic coefficient

$$2.40 \geq \beta_i = \frac{0.7}{T_i} \geq 0.8 \tag{1.2}$$

T_i - the natural period of the i-th mode of vibration;
K_c - the seismic design coefficient, Q_k - the effective gravity load at k-th level; η_{ik} - the modal response coefficient of the i-th mode for the k-th level.

The analysis is carried out by means of a computer programme. The following assumptions are made:

- the earthquake bearing structure consists of vertical shear walls connected with horizontal diaphragms;
- the horizontal diaphragms at floor levels are not deformable.
- distribution of the total lateral forces due to translational and torsional vibration is to be done on the basis of compatibility of the deformation in the space structure.

The input data for the given example are: design intensity - VII, soil conditions - group 4, corresponding seismic design coefficient - K_c = 0,033, concrete of grade C 20 with modulus of elasticity E_c = 2.40 x 10^4 MPa.
The computation is performed by means of a computer progrem for wall systems. The modal response coefficients are computed in accordance with the following formula

$$\eta_k = \frac{X_k \sum_{j=1} Q_j X_j}{\sum_{j=1} Q_j X_j^2} \tag{1.3}$$

where: X_k and X_j are the horizontal displacements at points k and j of the first mode of vibration.

The additional magnification factor for the spectral dynamic coefficient, according to the Amendment of 1972 to the BCBER, for large panel buildings is

$$A = 1.07 + 0.05\ (n-5), \tag{1.4}$$

but not greater than 1.20. In formula(1.4)n is the number of the storeys.

The computed results only for the transversal direction, of excitation are presented here. The natural period for the first mode of vibration is T = 0,45 s, and the corresponding specral dynamic coefficient is

$$\beta = \frac{0.7}{T} A = \frac{0.7}{0.45} \times 1.20 = 1.867$$

The values of the modal response coefficients and the computed seismic forces at each floor level are listed in table 1.2. The total shear forces and bending moments for the accepted model of cantilerer beam are presented in the same table.

Table 1.2

STOREY LEVEL k	DESIGN LOADS UNIFORMLY DISTRIBUT-ED $\|kN/m^2\|$	TOTAL $\|kN\|$	η_k	SEISMIC FORCES $\|kN\|$	TOTAL SHEAR FORCES $\|kN\|$	TOTAL BENDING MOMENTS $\|kNm\|$
8	17.25	7910	1.363	663.42	663.42	0.0
7	10.75	4930	1.128	341.52	1004.94	1857.58
6	10.75	4930	0.895	271.09	1276.03	4671.41
5	10.75	4930	0.671	203.37	1479.40	8244.30
4	10.75	4930	0.464	140.72	1620.12	12386.62
3	10.75	4930	0.284	86.00	1706.12	16922.95
2	10.75	4930	0.139	42.16	1748.28	21700.09
1	15.05	6890	0.041	17.32	1765.60	26295.27
0	-	-	0.0	0.0	1765.60	31538.95

The storey torsional moment due to eccentricity between the centres of mass and stiffness is computed as a storey shear force times the eccentricity perpendicular to the considered direction of excitation.

1.3.3 Internal Forces

The computed values of the distributed to the shear walls shear forces and bending moments at the bottom of shear walls are given in table 1.3.

Table 1.3

SHEAR WALL	SHEAR FORCE $\|kN\|$	BENDING MOMENT $\|kNm\|$	AXIAL FORCE $\|kN\|$
SW 1.1	112.01	1538.36	752.56
SW 1.2	143.46	2080.61	752.56
SW 2.1	168.89	2943.12	1333.41
SW 3.1	223.33	4344.42	1455.89
SW 4.1	117.44	1574.37	657.83
SW 5.1	205.72	3662.20	1006.65
SW 5.2	242.03	4289.90	974.98
SW 5.3	251.11	4446.83	974.98
SW 6.1	320.86	6715.04	1375.28

The vertical distribution of shear forces, bending moments and axial forces for SW 6.1 is presented in table 1.4.

Table 1.4

STOREY LEVEL	SHEAR FORCE |kN|	BENDING MOMENT |kNm|	AXIAL FORCE |kN|
8	160.38	0.0	238.00
7	225.39	449.06	238.00
6	281.67	1080.16	400.40
5	325.09	1868.83	562.80
4	354.32	2779.08	725.20
3	368.21	3771.18	887.59
2	362.78	4802.17	1049.99
1	320.39	5817.95	1212.39
0	320.39	6715.04	1375.28

1.4 PROPORTIONING

Proportioning of the shear wall SW 6.1 is presented here as an example. The maximum values of the internal forces at 0 level are: bending moment M = 6715.04 kNm, shear force Q = 320.39 kN and axial force N = 1375.28 kN. The dimensions of the cross-section of SW 6.1 are a = 6.140 m and b = 0.140 m.

The shear walls are designed with the following materials:

- concrete of grade C 20 with modulus of elasticity E_c = 24000 MPa and design compression strength R_c = 9 MPa.
- reinforcement steel of class A-III (according to Bulgarian standards) with design strength R_s = 360 MPa and yield strength of 400 MPa.

For earthquaue loading the design strength of both concrete and reinforcement should be increased by a factor n = 1.2 (BCBER). Then, $R'_c = 9.0 \times 1.2 =$ $= 10.8$ MPa and $R'_s = 360 \times 1.2 = 432$, but not greater than the yield strength - R'_s = 400 MPa.

The vertical reinforcement of the panels is symmetric and the distance from the extreme fibers of the concrete to the centroids of the reinforcement is $a_o = a'_o = 0.57$ m.

The depth of the compression face x is

$$x = \frac{N}{R'_c b} = \frac{1375.28 \times 10^{-3}}{10.8 \times 0.14} = 0.909 \text{ m} < 2a'_o$$

then: the area of the longitudinal reinforcement is computed in accordance with the following formula:

$$F_s = F'_s = \frac{N}{R'_s} \left(\frac{e}{h_o - a'_o} - 1\right) \qquad (1.5)$$

where

$$e = e_o + 0.5\ h - a_o \qquad (1.6)$$

$$e_o = \frac{M}{N} \eta \qquad (1.7)$$

$$\eta = \frac{1}{1-\frac{N}{N_c}} \tag{1.8}$$

The critical value of the axial force N_c in the plane of the shear wall is very high in comparison with the applied axial force N. Therefore, $\eta \approx 1$.

$$e_o = \frac{6715.04}{1375.28} \times 1 = 4.883$$

$$h_o = 6.14 - 0.57 = 5.57 \text{ m}$$

$$e = 4.883 + \frac{5.57 - 0.57}{2} = 7.383$$

$$F_s = F'_s = \frac{1375.28 \times 10^{-3}}{400} \left(\frac{7.383}{5.000} - 1\right) = 16.38 \times 10^{-4} \text{ m}^2$$

$$F_s = F'_s = 16.38 \text{ cm}^2$$

The reinforcement bars can be

$2 \times$ No 22 + $2 \times$ No 25, $F_s = 7.60 + 9.82 = 17.42 \text{ cm}^2$

The shear forces in the horizontal connections between the shear wall panels are transferred by embedded steel I 10 profiles. The minimum embeddment into the panels is:

$$l_{e\ min} = \frac{Q}{1.65\ b\ R'_c} \tag{1.9}$$

where: Q - shear force applied to one key connection; b - width of the steel profile; R'_c - design compressive strength of the concrete. The ratio of embeddment l_e to the free length - l should be $l_e/l \geq 2$.

The ultimate shear capacity for one key can be determined from:

$$Q' = \mu \gamma R'_c F' \tag{1.10}$$

where: μ - coefficient for nonuniform distribution of the compressive stress between steel profile and concrete; F' - contact effective compression area; γ - amplification factor for compressive strength of confindd concrete:

$$\gamma = \sqrt[3]{\frac{F}{F_o}} \tag{1.11}$$

where: F - effective cross-section area of the concrete in the connection; F_o - flange area of the steel profile of the connection.

For the given example: $l_e = 16$ cm, $l = 7$ cm, $\mu = 0.75$, $F_o = 7 \times 5.5 = 38.5 \text{cm}^2$, $F = 7 \times 10 = 70 \text{ cm}^2$, $F' \cong 7 \times (5.5 + 5.05) = 74.0 \text{ cm}^2$,

$$\gamma = \sqrt[3]{\frac{70.0}{38.5}} = 1.22$$

$$Q' = 0.75 \times 1.22 \times 10.8 \times 10^{-1} \times 74.0 = 73.1 \text{ kN}$$

For shear wall SW 6.1 with maximum shear force 320.86 kN six key connections are necessary -

$$6 \times 73.1 = 438.6 > 1.25 \times 320.86 = 401.01.$$

The coefficient 1.25 takes into account nonuniform distribution of the shear force between the connections.

1.5 SELECTED DETAILS

Some typical details for the ESS are presented on figs. 1.8 to 1.11.

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NOTATIONS

<u>Report</u>		<u>CEB</u>
A	- reduction factor of the spectral dynamic coefficient	-
E_c	- modulus of elasticity of concrete	E_c
E_s	- modulus of elasticity of reinforcement	E_s
F_s	- area of tension reinforcement	A_s
F'_s	- area of compression reinforcement	A'_s
G	- shear modulus of concrete	-
H_k	- height from foundation level to k-th level	h_k
I	- moment of inertia	I
K	- lateral stiffness	K
K_ϕ	- torsional stiffness	K_ϕ
K_c	- seismic design coefficient	-
M	- flexural moment	M
M_t	- torsional moment	M_t
N	- axial force	N
Q	- shear force	V
Q_c	- shear strength of concrete	V_c
Q_k	- effective gravity load at k-th level	W_k
R_c	- design compressive strength of concrete	f_c
R_s	- design strength of reinforcement	f_s
R_t	- design tension strength of concrete	f_{ct}
S	- design seismic force	F
T	- natural period of vibration	T
a	- distance from shear wall axis to centre of stiffness	-
e	- eccentricity	e
h_o	- effective depth	-
x	- height of compressive zone	-
x	- coordinates	x
y	- coordinates	y
β	- spectral dynamic coefficient	R_a
δ	- unit deflection coefficient	δ
η	- modal response coefficient, slenderness coefficient	-
μ	- distribution coefficient	-
ξ	- relative height of compression zone	-
σ_A	- effective stress in reinforcement	-

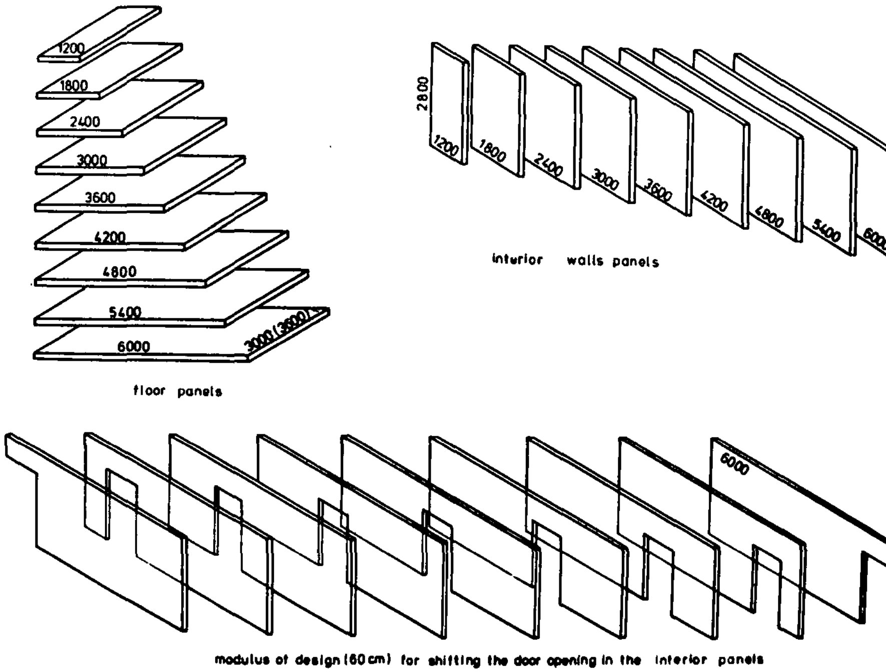


fig. 1.1 Standard prefabricated elements

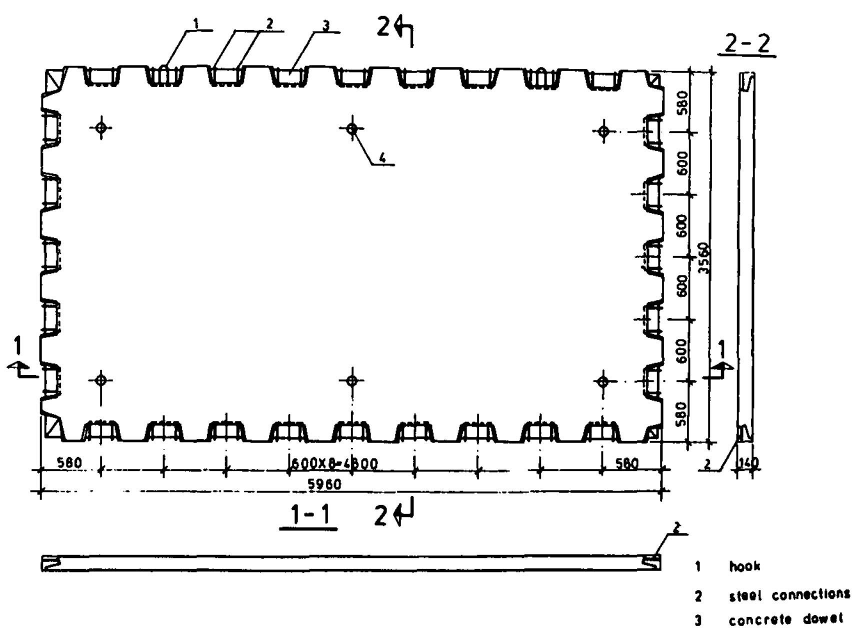


fig. 1.2 Floor panel

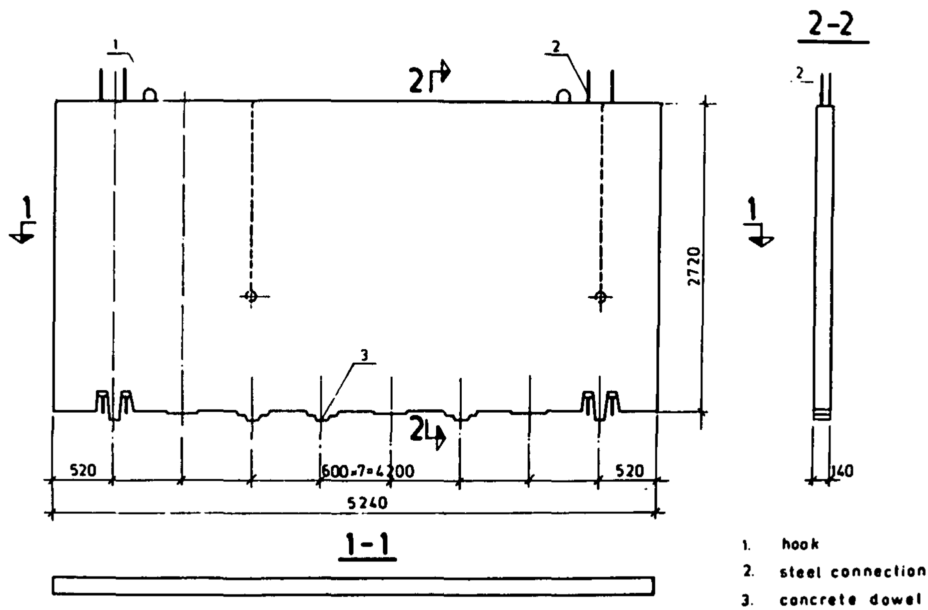


fig. 1.3 Bearing wall element

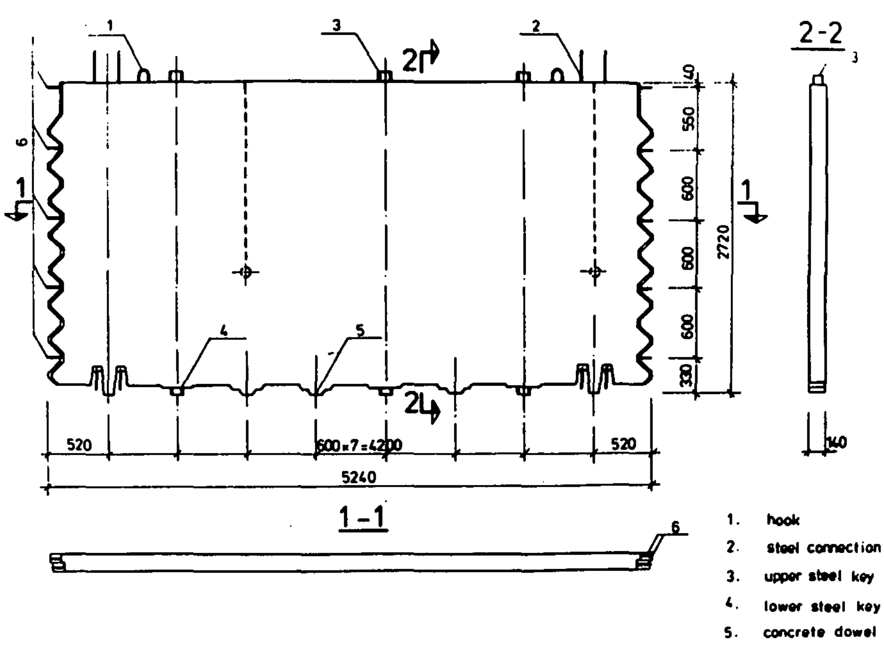


fig. 1.4 Shear bearing wall element

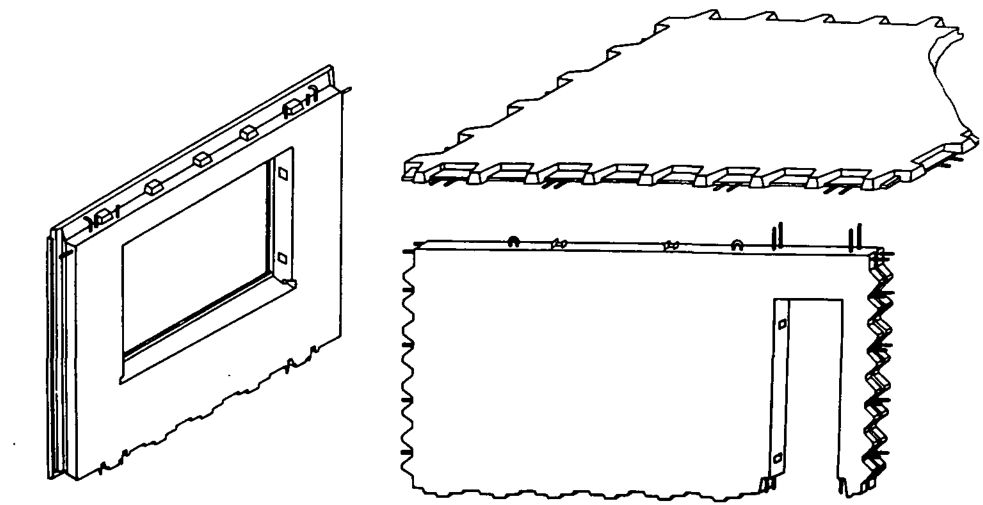


fig. 1.5 Isometric view of exterior, interior and floor panels

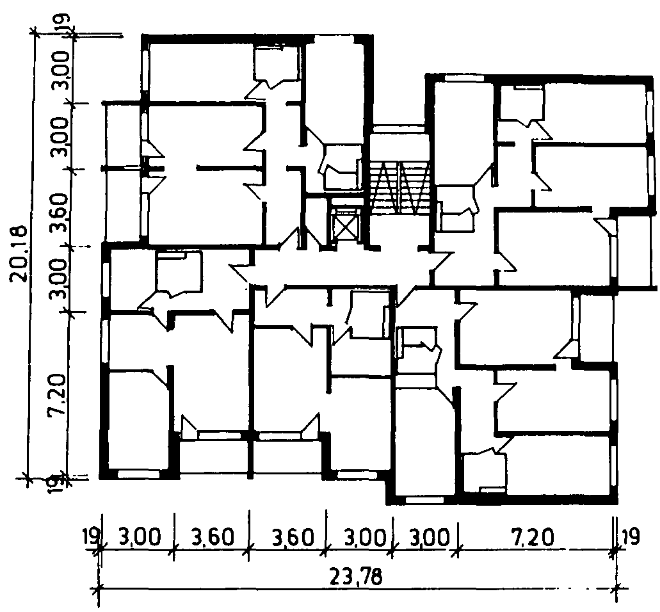


fig. 1.6 Typical floor plan

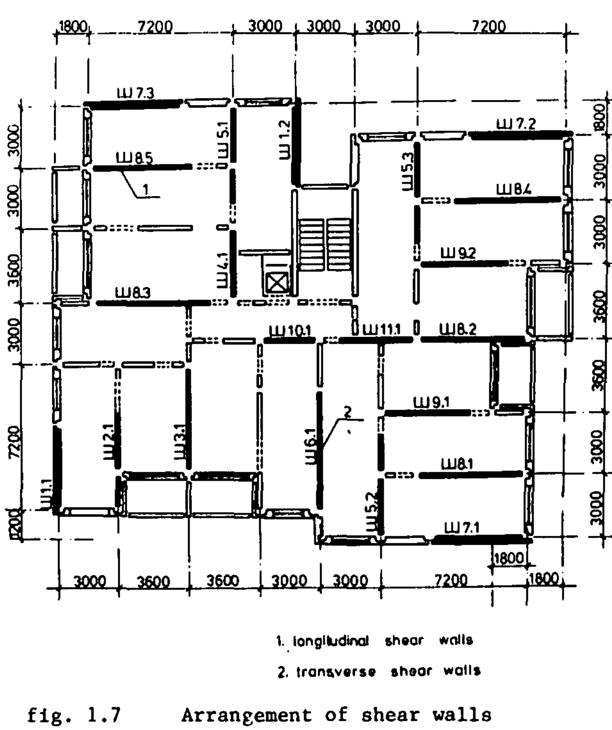


fig. 1.7 Arrangement of shear walls

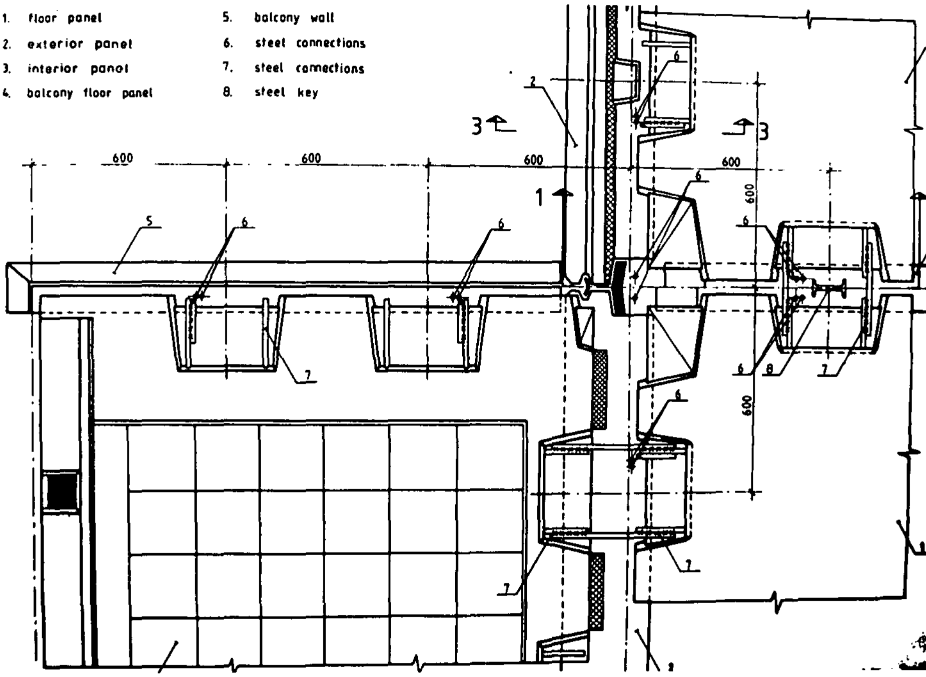


fig. 1.8 Details of connections

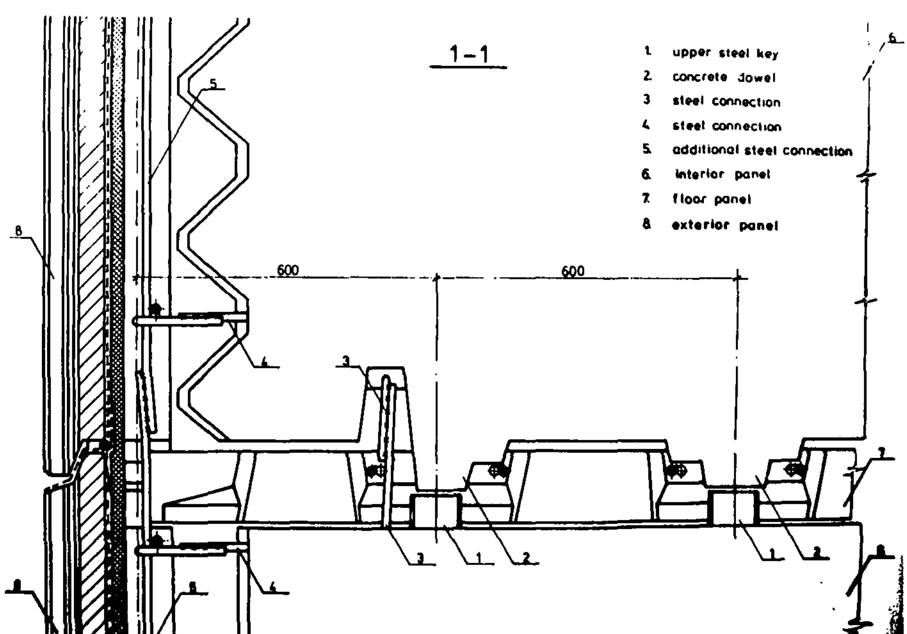


fig. 1.9 Detail of horizontal and vertical connections

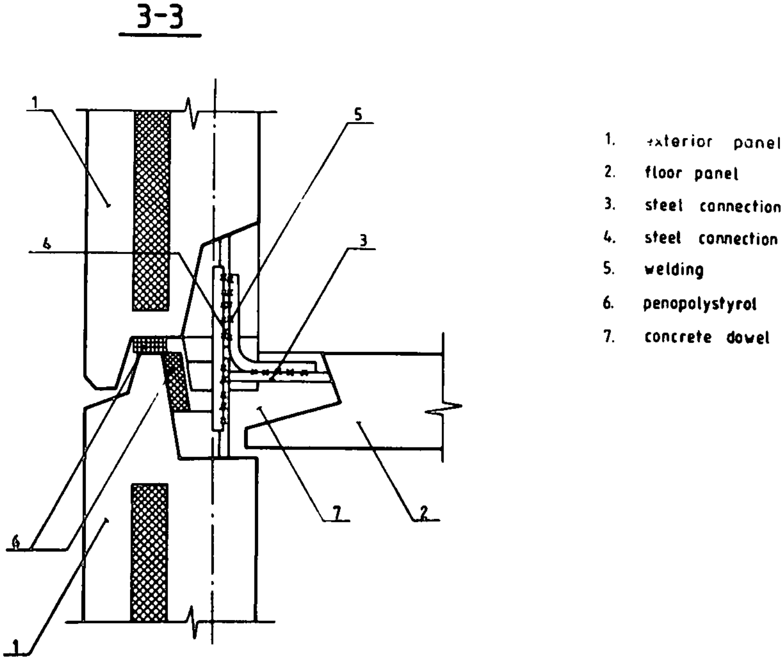


fig. 1.10 Connection between exterior and slab panels

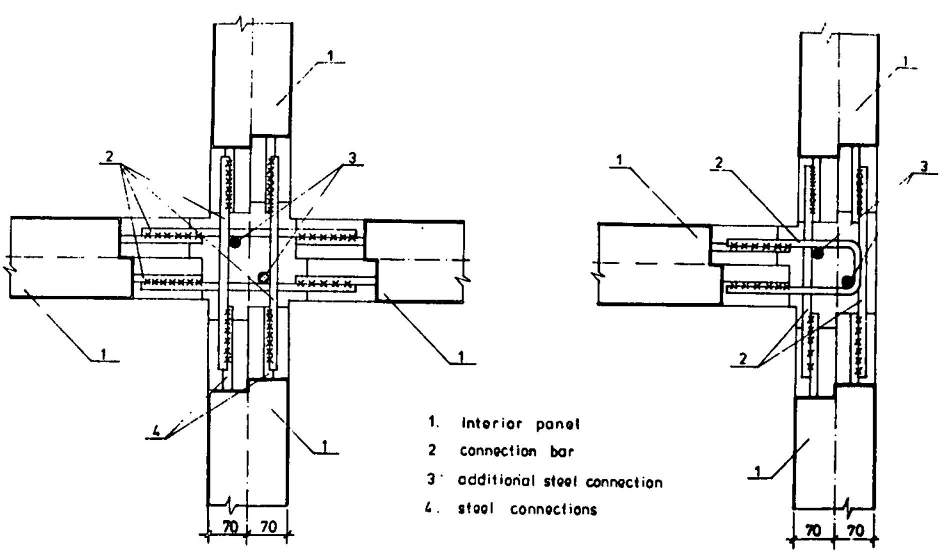


fig. 1.11 Typical vertical connections

NOTATIONS

Report		CEB
A	- reduction factor of the spectral dynamic coefficient	-
E_c	- modulus of elasticity of concrete	E_c
E_s	- modulus of elasticity of reinforcement	E_s
F_s	- area of tension reinforcement	A_s
F'_s	- area of compression reinforcement	A'_s
G	- shear modulus of concrete	-
H_k	- height from foundation level to k-th level	h_k
I	- moment of inertia	I
K	- lateral stiffness	K
K_ϕ	- torsional stiffness	K_ϕ
K_c	- seismic design coefficient	-
M	- flexural moment	M
M_t	- torsional moment	M_t
N	- axial force	N
Q	- shear force	V
Q_c	- shear strength of concrete	V_c
Q_k	- effective gravity load at k-th level	W_k
R_c	- design compressive strength of concrete	f_c
R_s	- design strength of reinforcement	f_s
R_t	- design tension strength of concrete	f_{ct}
S	- design seismic force	F
T	- natural period of vibration	T
a	- distance from shear wall axis to centre of stiffness	-
e	- eccentricity	e
h_o	- effective depth	-
x	- height of compressive zone	-
x	- coordinates	x
y	- coordinates	y
β	- spectral dynamic coefficient	R_a
δ	- unit deflection coefficient	δ
η	- modal response coefficient, slenderness coefficient	-
μ	- distribution coefficient	-
ξ	- relative height of compression zone	-
σ_A	- effective stress in reinforcement	-

References

1. Bulgarian Code for Buildings in Earthquake Regions - 1964, Earthquake Resistant Regulations, a World List, 1973

2. Instructions for Design and Construction of Residential and Public Buildings in Seismic Zones, Sofia, 1978 (in Bulgarian)

3. Concrete and Reinforced Concrete Structures, Design Code, 1980 (in Bulgarian)

4. Manual for Design of Concrete and Reinforced Concrete Structures, 1981 (in Bulgarian)

$$x = \frac{N - R_c' (b_c - b) b_c}{R_c' b} = \frac{6.984 - 16.2\ (0.40 - 0.15)\ 0.40}{16.2 \times 0.15} = 2.207 \text{ m}$$

For the combined flexural and axial loads

$$Ne = R_c' bx\ (h_o - 0.5\ X) + R_c'\ (b_c - b) b_c\ (h_o - 0.5\ h_c) + R_s' F_s'\ (h_o - a) \quad (3.$$

where: $h_o = h - a = 7.60 - 0.20 = 7.40$ m;

$$e = \frac{M}{N} + 0.5\ h - a = \frac{16933.84}{6984.10} + 0.5 \times 7.60 - 0.20 = 6.02 \text{ m;}$$

$b_c = 0.40$ m; $b = 0.15$ m; $h_c = 0.40$ m;

$F_c' = 37.68 \times 10^{-4}\ m^4$

$Ne = 6984.10 \times 6.02 = 42044.3$ kNm.

The right side of (3.5) is:

$$16.2 \times 10^3 \times 0.15 \times 2.207\ (7.40 - 0.5 \times 2.207) +$$
$$+ 16.2 \times 10^3\ (0.40 - 0.15)\ 0.40\ (7.40 - 0.5 \times 0.40) +$$
$$+ 400 \times 10^3 \times 3.768 \times 10^{-3}\ (7.40 - 0.20) = 56284.03 \text{ kNm}$$

42044.30 56284.03. (The strength is provided.)

The shear force at the column to web vertical interface is:

$$T = \frac{Q\ S}{I} h_s = \frac{1168.86 \times 0.576}{8.08} \times 4.20 = 353.34 \text{ kN}$$

where: Q - the maximum shear force for the shear wall SW 1; S - the static moment:

$$S = 0.4^2 \times 3.60 = 0.576\ m^3$$

I - the moment of inertia of the cross-section;
h_s - the storey height.

The shear force T is transmitted by four steel key connections. Each one should carry 88.34 kN.

3.5 SELECTED DETAILS

The column to column connection is performed by welding of steel shoes. Mortar is placed into the column joint space (fig. 3.11). The beams are supported on short column cantilevers. They are connected by welding of em bedded steel shapes.

The floor panels to beam connection and the wall panels to beam connectio are shown on fig. 3.12.

corresponding spectral dynamic coefficient is:

$$\beta_1 = \frac{0.7}{T_1} A = \frac{0.7}{0.67} \times 1.07 = 1.118$$

The computed values of the modal response coefficients for the first mode of vibration, the total seismic force and the internal forces are given in table 3.1.

Table 3.1

STOREY LEVEL k	DESIGN LOADS DISTRIBUTED kN/m^2	DESIGN LOADS TOTAL kN	η_k	SEISMIC FORCES kN	TOTAL SHEAR FORCES kN	TOTAL BENDING MOMENTS kNm
5	9.46	10572.5	1.420	839.22	839.22	0
4	11.12	12427.7	1.035	719.02	1558.24	3021.20
3	11.12	12427.7	0.668	463.37	2021.61	8630.87
2	11.12	12427.7	0.342	239.67	2261.29	15908.68
1	11.12	12427.7	0.110	76.42	2337.71	24049.32
0	-	-	0	-	2337.71	33867.68

3.3.3 Internal Forces

The shear forces, the bending moments and the axial forces at the bottom of the shear walls are given in table 3.2.

Table 3.2

SHEAR WALL	SHEAR FORCE kN	BENDING MOMENT kNm	AXIAL FORCE kN
SW 1	1168.86	16933.84	6984.10
SW 2	1168.86	16933.84	6984.10

3.4 PROPORTIONING

In accordance with the computed values of the internal forces a shear wall type D - 7.2 x 4.2 - 1 is chosen for the first storey and D - 7.2 x 3.6 - 1 for the other storeys, from the catalogue graphs reproduced on fig. 3.10. The dimensions of the shear wall SW 1 are presented on fig. 3.9. The moment of inertia of its cross-section is $I = 8.08\ m^4$. The prefabricated elements are of concrete C 30 with modulus of elasticity E_c = 29 000 MPa and design strength R_c = 13.5 MPa. The reinforcement steel is of class A-III, according to the Bulgarian standards, with design strength R_s = 360 MPa. The design strength of both concrete and reinforcement should be factored by n = 1.2 for earthquake loads (BCBER). Then, $R_c' = 1.2 \times 13.5 = 16.2$ MPa and $R_s' = 1.2 \times 360 = 423$, but not greater than the yield strength - R_s' = 400 MPa.

The longitudinal reinforcement in the columns is 2 x 12 No 20. The web reinforcement is of welded wire fabric with diameter of 6 mm, spaced at 20 cm.

For illustration the strength check of the shear wall SW 1 is presented. According to Ref. |4| the depth of the compression zone is:

conditions of group 3, according to the Bulgarian Code for Buildings in Earthquake Regions (BCBER) |1|. The seismic design coefficient is K_c=0.05

The lateral seismic resistance is provided by reinforced concrete shear walls. They are symmetrically situated in plan.

The dynamic model of the building is assumed to be a fixed-base cantilever with lumped masses at the floor levels.

3.3.1 Design Loads

The design gravity loads considered in the computation of the horizontal earthquake forces include the specified dead load and the effective live load and snow load. The corresponding load factors are:

- dead load of all permanent structural and nonstructural components n=1.0
- live load on the floor slabs - n=0.5
- snow load - n=0.8

The computed design load forces, concentrated at the storey levels are given in table 3.1.

3.3.2 Computation Techniques

For application of the system in the design practice design catalogues are developed with tables and graphs for direct selection of typified structural elements such as prefabricated floor panels, beams, columns and shear wall panels. This simplifies the computational efforts.

The lateral seismic forces are computed in accordance with the following formula (BCBER-64):

$$S_{ik} = \beta_i \eta_{ik} K_c Q_k \tag{3.1}$$

where: S_{ik} - the design seismic force, applied at point k for the i-th mode of vibration; β_i - the spectral dynamic coefficient:

$$2.40 \geq \beta_i = \frac{0.7}{T_i} \geq 0.8 \tag{3.2}$$

T_i - the natural period of the i-th mode of vibration; K_c - the seismic design coefficient; Q_k - the effective gravity load at k-th level; η_{ik} - the modal response coefficient of the i-th mode of vibration for the k-th level

$$\eta_{ik} = \frac{X_{ik} \sum_{j=1}^{n} Q_j X_{ij}}{\sum_{j=1} Q_j X_{ij}^2} ; \tag{3.3}$$

X_{ik} and X_{ij} - the horizontal ordinate at points k and j of the i-th mode of vibration.

According to the Amendment to the BCBER of 1972, the spectral dynamic coefficient should be increased by a factor A:

$$A = 1.07 + 0.06\ (n - 5) \leq 1.3 \tag{3.4}$$

where: n - the number of the storeys.

The computed natural period of the first mode of vibration is T_1 = 0.67 s. Th

3. FRAME SYSTEM YC-73

3.1 SCOPE

The present example deals with earthquake resistant design of prefabricated frame-shear wall system YC-73. Structural analysis of a 5 storey building is presented. It involves computation of earthquake forces, internal forces and proportioning of a shear wall, in accordance with the National Design Codes.

3.2 DESCRIPTION OF THE SYSTEM

The structural system YC-73 is a frame-shear wall system designed for multistorey public and industrial buildings. The basic modulus of 1200 mm in both directions in plane and of 600 mm in height is applied for this system. A typical lay-out of a part of the system is shown on fig. 3.1. The design live loads and the distances between the column axes are presented on the same figure.

The structural system is composed of foundations, columns, beams, prestressed floor panels with cylindric holes and vertical shear walls in both directions. Hinged connections are accepted for the floor panels and the beams.

The lateral seismic resistance of the structure is provided by vertical shear walls and horizontal diaphragms. The floor structures performed by connected together floor panels and beams serve as diaphragms. The shear walls are of prefabricated type and are composed of prefabricated columns, walls and beams.

The location of the shear walls depends on the architectural or the technological design of the building, but generally it is recommended to locate them symmetrically to the main axes of the building. Examples are shown in figs. 3.2 and 3.3.

The foundations of the shear walls are cast-in-place and are individually designed for each building. Anchors and steel bars to connect columns and walls are embedded in the footings.

The floor panels have cylindrical openings. They are prestressed RC 'Spirol' type panels (fig. 3.4). The beams are of reinforced concrete C 40 and they have reverse T cross-section (fig. 3.4).

The columns are of reinforced concrete C 20 to C 40 with cantilevers and with cross-section of 400/400 mm (fig. 3.5).

The wall elements are with thickness of 150 mm. (fig. 3.6 and fig. 3.7). For buildings of small storey height (up to 3.60 m) the wall elements are single and for higher storeys two elements one over another are used. The connection between the elements is made by welding of the embedded steel shapes.

3.3 STRUCTURAL ANALYSIS

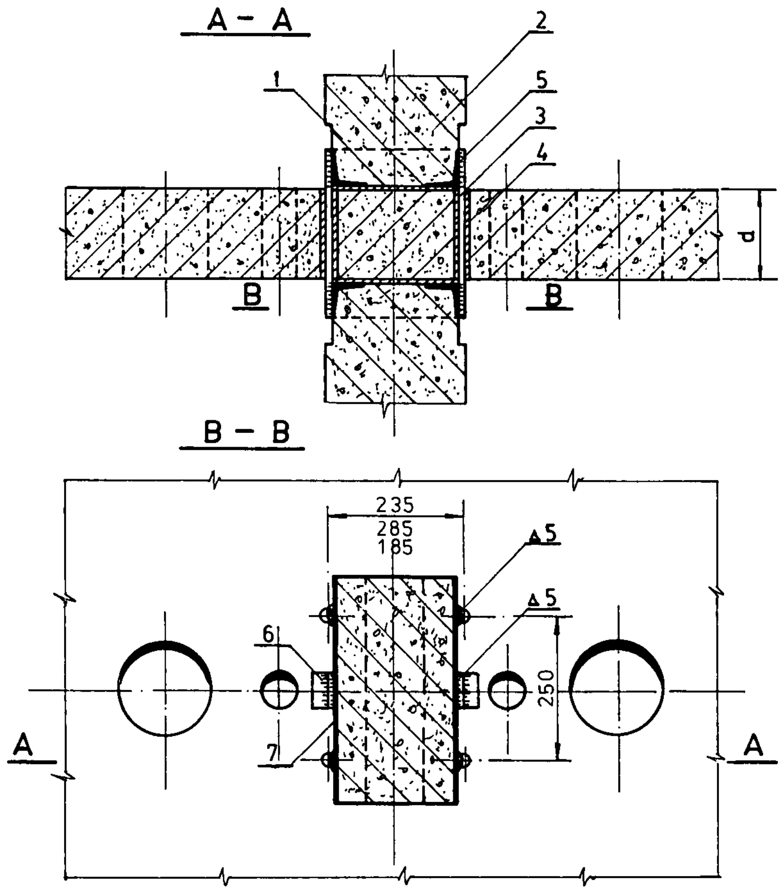
A five storey prefabricated building with dimensions in plan of 50.80 x 22.00 m and storey height of 3.60 m is presented as an example. The distances between the column axes in both directions are of 7.20 m (fig. 3.9).

The structure is situated in a seismic zone of intensity of VIII and soil

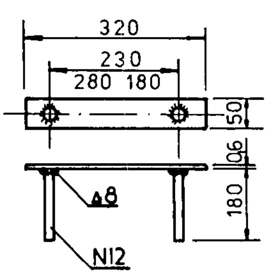
3. REPRESENTATIVE EXAMPLE OF PREFABRICATED FRAME SYSTEM YC-73 BULGARIA

Prepared by S. Simeonov, P. Sotirov, N. Ignatiev, I. Nikolov and A. Nikolov

Editor: P. Sotirov



METAL PLATE M1



1 Column
2 Cement mortar C20
3 Connecting bars min ø 12
4 Mortar
5 Metal shoe
6 Metal plate M1
7 Connecting plank 50.40.6

Fig 2.6

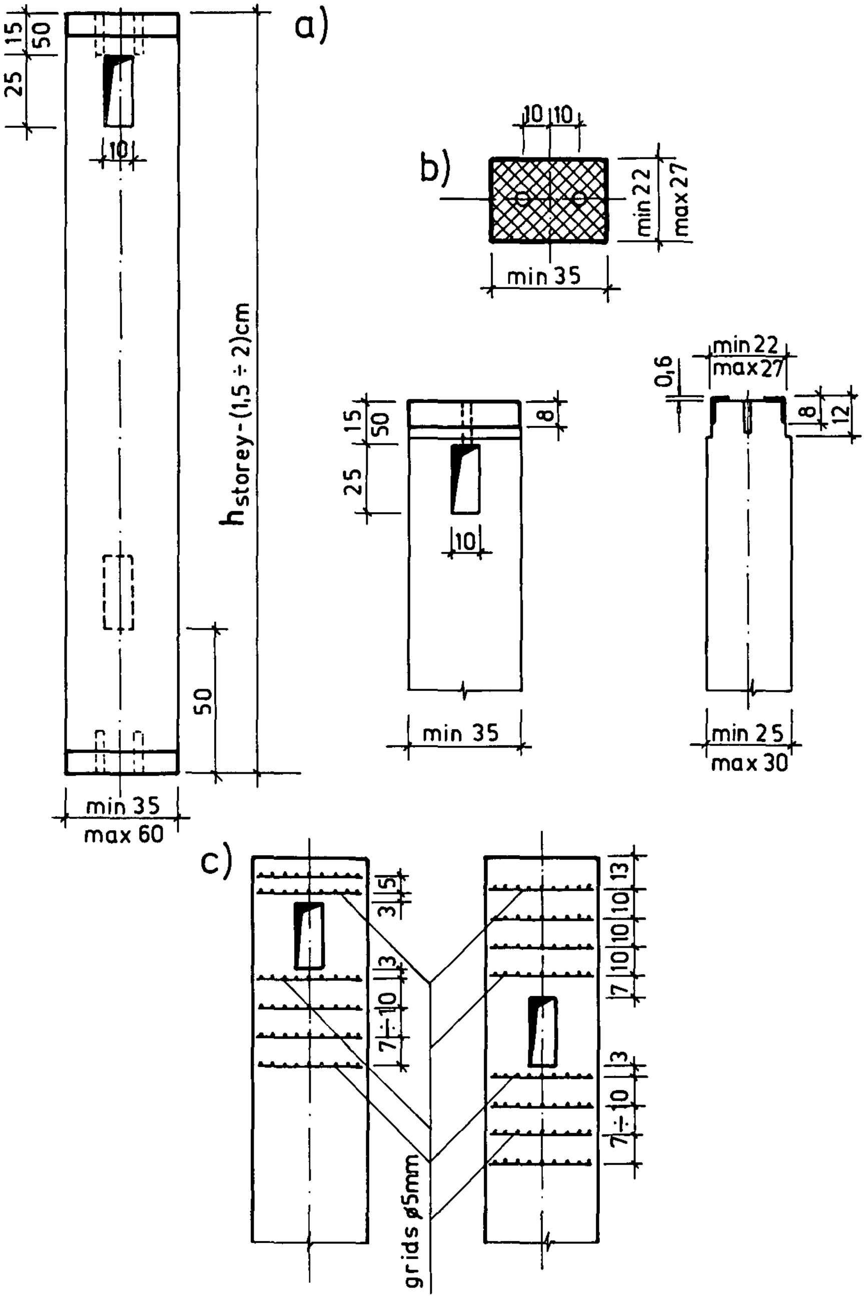
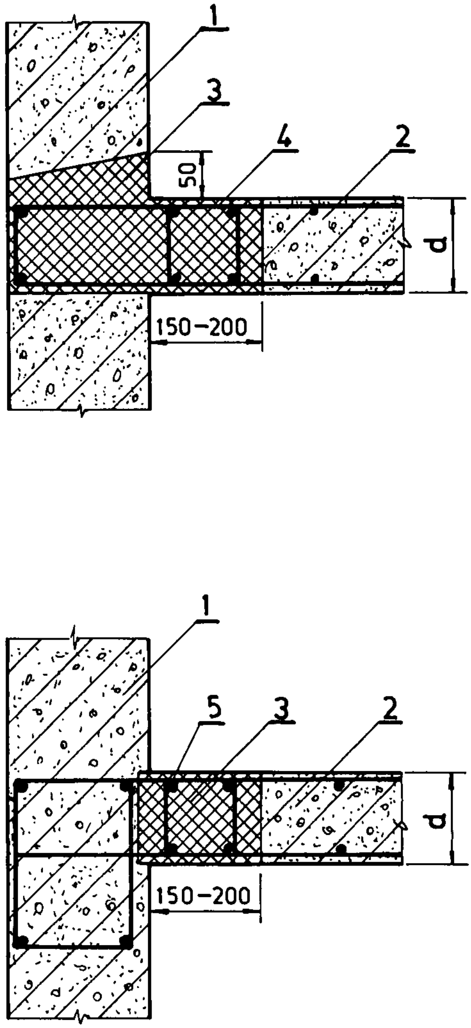
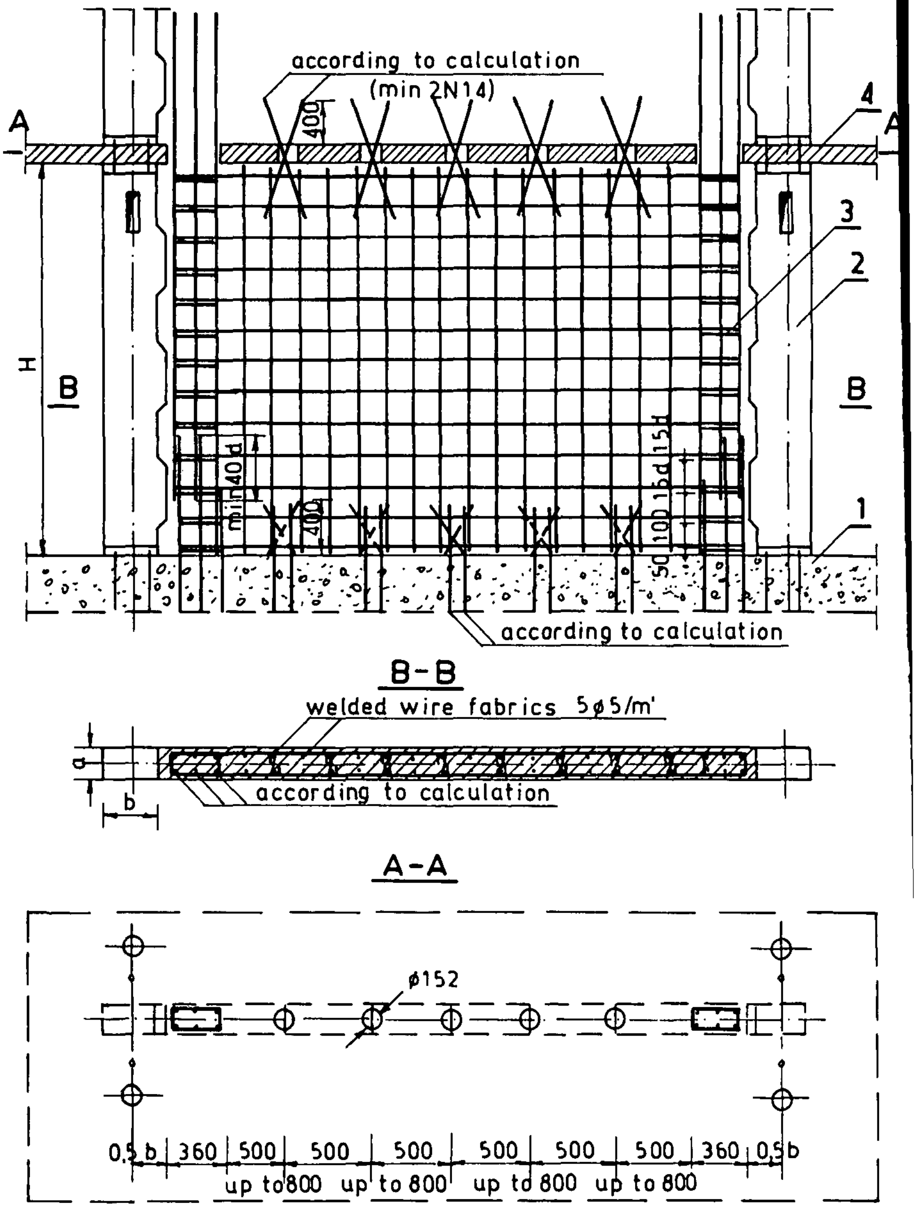

a)
15
50
25
10
h_{storey}-(1,5 ÷ 2)cm
50
min 35
max 60
b)
10 10
min 22
max 27
min 35
15
50
25
8
10
min 35
min 22
max 27
0,6
8
12
min 25
max 30
c)
5
3
3
7÷10
13
10
10
10
7
3
7÷10
grids ø5mm

Fig 2.5



1. Stairwell
2. Floor slab
3. Monolithic connection – concrete C20
4. Reinforcing skeleton
5. Longitudinal reinforcement

Fig. 2.4



1 – foundation

2 – column

3 – reinforced concrete shear wall

4 – floor slab

fig. 2.3

Total seismic forces, moments and shear forces in

SW 2

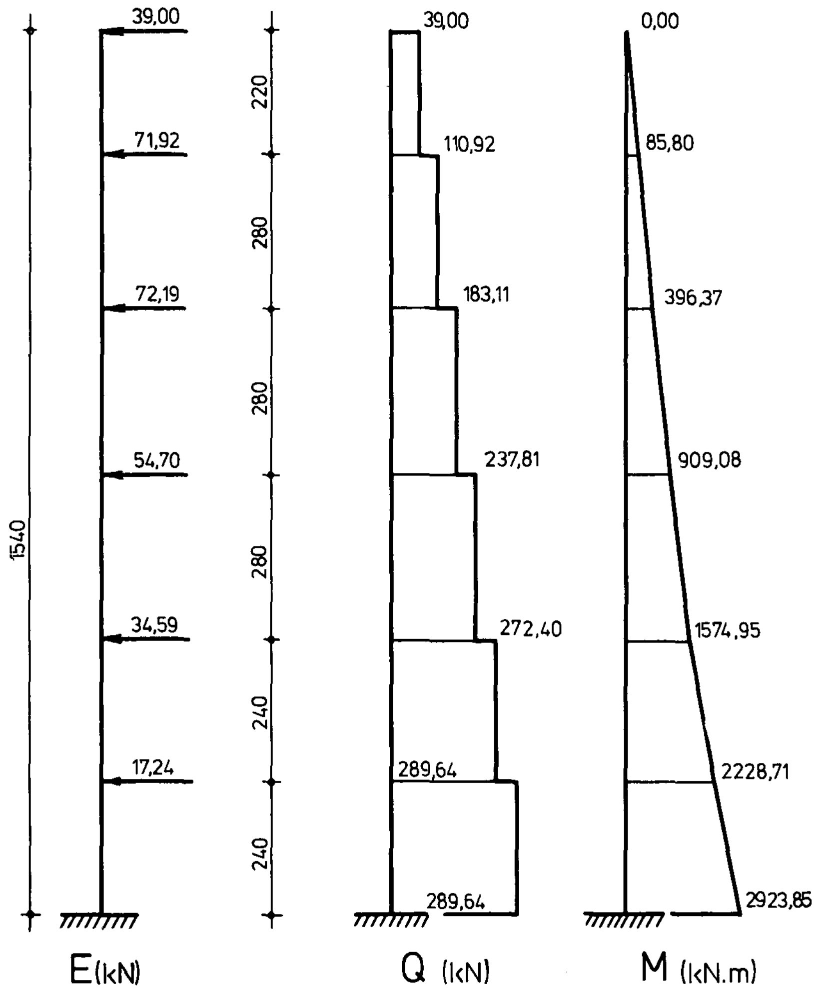


Fig. 2.2

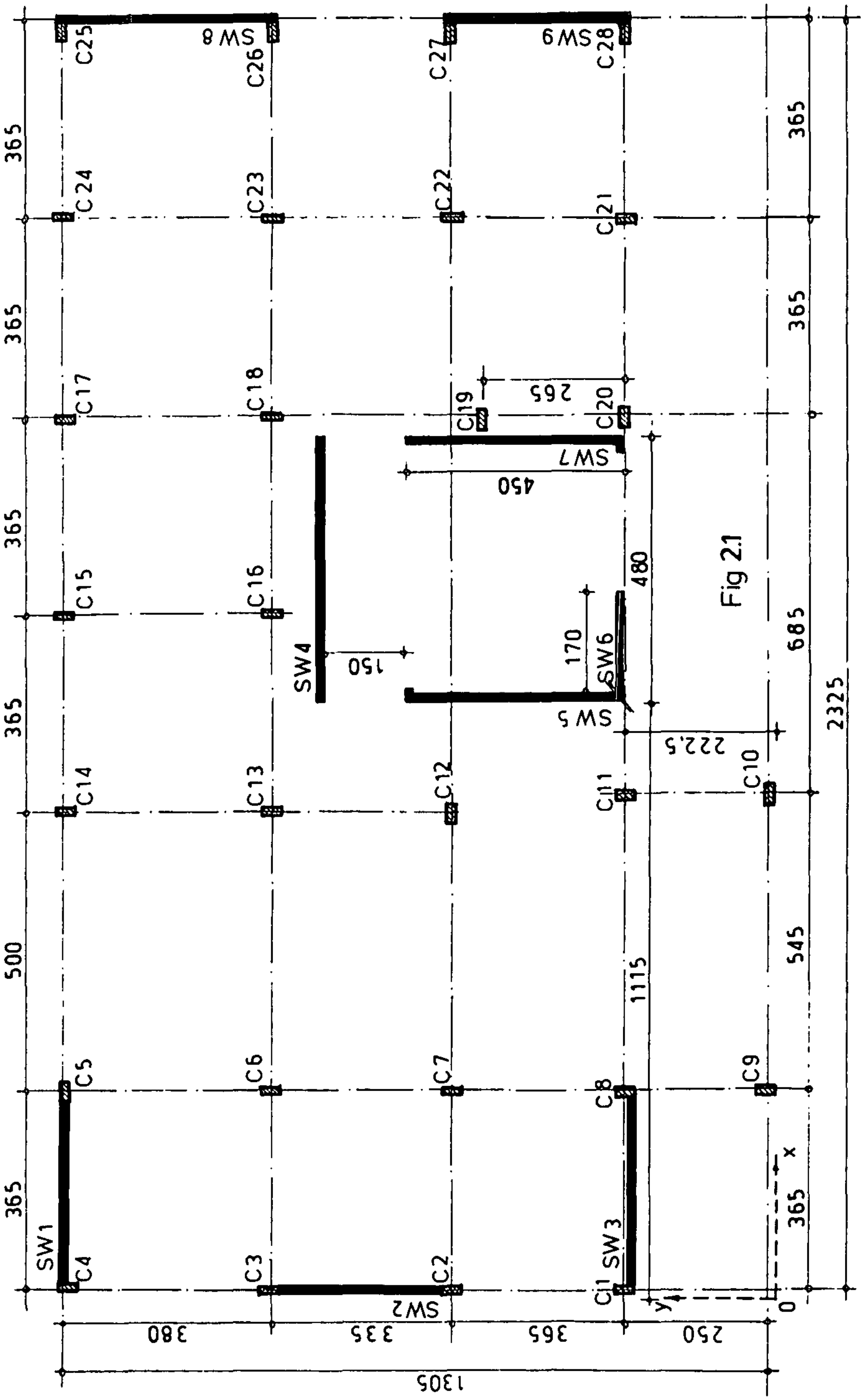

C1
C2
C3
C4
C5
C6
C7
C8
C9
C10
C11
C12
C13
C14
C15
C16
C17
C18
C19
C20
C21
C22
C23
C24
C25
C26
C27
C28
SW1
SW2
SW3
SW4
SW 5
SW6
SW7
SW 8
SW9
365
500
365
365
365
365
380
335
365
250
1305
365
545
685
365
365
2325
1115
480
450
265
170
150
222,5
x
y
0

Fig 2.1

NOTATIONS

Report		CEB
A	- reduction factor of the spectral dynamic coefficient	-
E_c	- modulus of elasticity of concrete	E_c
E_s	- modulus of elasticity of reinforcement	E_s
F_s	- area of tension reinforcement	A_s
F'_s	- area of compression reinforcement	A'_s
G	- shear modulus of concrete	-
H_k	- height from foundation level to k-th level	h_k
I	- moment of inertia	I
K	- lateral stiffness	K
K_ϕ	- torsional stiffness	K_ϕ
K_c	- seismic design coefficient	-
M	- flexural moment	M
M_t	- torsional moment	M_t
N	- axial force	N
Q	- shear force	V
Q_c	- shear strength of concrete	V_c
Q_k	- effective gravity load at k-th level	W_k
R_c	- design compressive strength of concrete	f_c
R_s	- design strength of reinforcement	f_s
R_t	- design tension strength of concrete	f_{ct}
S	- design seismic force	F
T	- natural period of vibration	T
a	- distance from shear wall axis to centre of stiffness	-
e	- eccentricity	e
h_o	- effective depth	-
x	- height of compressive zone	-
x	- coordinates	x
y	- coordinates	y
β	- spectral dynamic coefficient	R_a
δ	- unit deflection coefficient	δ
η	- modal response coefficient, slenderness coefficient	-
μ	- distribution coefficient	-
ξ	- relative height of compression zone	-
σ_A	- effective stress in reinforcement	-

or $F_s = F_s' = 18.80$ cm^2. The reinforcement bars can be 6 No 20 with F = 18
The reinforcement ratio is $\mu = 0.0054$.

The shear strength provided by concrete is $Q_c = 0.6\ R_t'\ bh_o$

The design tension strength of concrete C20 is $R_t = 0.75$ MPa and respec
vely $R_t' = 1.20 \times 0.75 = 0.9$ MPa. Then

$$Q_c = 0.6 \times 0.9 \times 0.25 \times 2.775 \times 10^3 = 347.63 \text{ kN.}$$

The computed shear force is $Q = 289.64$ kN. In this case shear reinforcem
shall be provided in accordance with the minimum shear reinforcement requ
ments.

2.5 SELECTED DETAILS

For illustration some typical details of the Package Lift-Slab System
chosen. A shear wall in the basement storey is shown on fig. 2.3. Two
types of slab to stair-well connections are presented on fig. 2.4.
connections are performed after the lifting process is completed. Th
cast-in-place concrete should not be less than C20.

Some details of the prefabricated columns can be seen on fig. 2.5. The
holes in the upper part of the columns are designed for temporary su
porting of the lifting devices. The typical column to slab connection
shown on fig. 2.6.

2.6 CONCLUSIONS

Package Lift-Slab System has extensive use in Bulgaria in seismic and non
seismic areas. During the Vrancea Earthquake of March 4, 1977, the
Northern part of Bulgaria was strongly affected. The post earthquake obse
vatious of the buildings have shown that Package Lift-Slab is a reliable
system for earthquake excitation.

The presented design example is performed in accordance with the current
design philosophy and practice.

References

1. Bulgarian Code for Buildings in Earthquake Regions - 1964, Earth-quake Resistant Regulations, a World List, 1973

2. Instructions for Design and Construction of Residential and Publi Buildings in Seismic Lones, Sofia, 1978 (in Bulgarian)

3. Concrete and Reinforced Concrete Structures, Design Code, 1980 (in Bulgarian)

4. Recommendations for Design and Construction of Package Lift-Slab Buildings, 1972 (in Bulgarian)

5. Manual for Design of Concrete and Reinforced Concrete Structures, 1981 (in Bulgarian)

2.4 PROPORTIONING

For this example the proportioning of shear wall SW 2 is chosen. The maximum values of the internal forces at 0 level are taken from table 2.5. They are: bending moment M=2 923.85 kNm. Shear force Q=289.64 kN and axial force N=721.85 kN. The dimensions of the horizontal cross-section of that shear wall are taken from table 2.1. They are: a=2.975 m and b= 0.25 m.

The shear walls are designed to be constructed by concrete of grade C-20, with modulus of elasticity E_c=24000 MPa and design compression strength R_c=9 MPa. The reinforcement steel is of class A-III (according to the Bulgarian standards) with design strength R_s=360 MPa and yield strength of 400 MPa. For earthquake loading the design strength of concrete and reinforcement should be increased by a factor of n_1=1.20. Then, R_c=1.2 x x 9=10.8 MPa; R_s'=1.2 x 360=432>400 and R_s' =400 MPa.

The bearing capacity of the cross-sections normal to the vertical axis of the shear wall are to be computed for simultaneous axial force and bending moment. The effective depth of the cross-section is

$$h_o = a - a_o = 2.975 - 0.20 = 2.275 \text{ m}.$$

The eccentricity e_o is

$$e_o = \frac{M}{N} = \frac{2\ 923.85}{721.85} = 4.05 \text{ m}.$$

The slenderness effect is taken into account by coefficient η

$$e_o' = e_o\eta \ : \quad \eta = \frac{1}{1-\frac{N}{N_c}}$$

The critical value of the axial force N_c in the wall plane, in comparison with the computed value of the axial force N, is very high and therefore $\eta \simeq 1$.

$$e = e_o + 0.5\ a - a_o = 4.05 + 0.5 \times 2.875 - 0.20 = 5.338 \text{ m}$$

Check if $\xi \gtrless \xi_R$?

where: ξ - the relative height of the compression zone; ξ_R- the ultimate relative height of the compression zone.

$$\xi = \frac{x}{h_o} = \frac{N}{R_o b h_o} = \frac{721.85 \times 10^{-3}}{10.8 \times 0.25 \times 2.775} = 0.096$$

$$\xi_R = \frac{\xi o}{1 + \frac{\sigma A}{400}\left(1 - \frac{\xi o}{1.1}\right)} = \frac{0.764}{1 + \frac{400}{400}\left(1 - \frac{0.764}{1.1}\right)} = 0.585$$

where: $\xi_o = 0.85 - 0.008\ R_c' = 0.764$ and $\sigma_A = R_S'$ = 400 MPa. It is evident that $\xi < \xi_R$. Then the necessary area of reinforcement is

$$F_s = F_s' = \frac{N}{R_s'\ (h_o - a_o')} \left\{e - h_o + \frac{N}{2R_c b}\right\}$$

$$F_s = \frac{721.85 \times 10^{-3}}{400\ (2.775 - 0.20)} \left\{5.338 - 2.775 + \frac{721.85 \times 10^{-3}}{2 \times 10.8 \times 0.25}\right\} = 18.80 \times 10^{-4} \text{ m}^2$$

The eccentricity for the first three storeys is assumed to be e_3=5.625 m, for the next three storeys e_6=5.622 m.

The additional lateral forces due to torsional effect are considered only s tically. They are determined in accordance with the formula:

$$S_{jk} = \frac{K_{jk}\, a_{jk}}{K_{\phi k}} M_{tk} \qquad (2$$

where: S_{jk} - the additional lateral force acting on the j-th vertical bear component (shear wall) at the k-th level; K_{jk} - as in formula (2.3); a_{jk} - distance from the j-th shear wall to the centre of stiffness; $K_{\phi k}$ - the to sional stiffness at k-th level:

$$K_{\phi k} = \sum_{j=1}^{n} (K_{jkx}\, a^2_{jkx} + K_{jky}\, a^2_{jky}) \qquad (2$$

K_{jkx}, K_{jky} - the stiffnesses of the shear walls in both directions x and y; a_{jkx}, a_{jky} - the distances of the shear walls to the axes x and y going through the centre of stiffness.

The distribution coefficients are computed also for the 3-rd and 6-th level (table 2.4). The additional torsional forces, distributed to the shear wall in transversal direction are given in table 2.4.

2.3.3 Internal Forces

Bending moments and shear forces for each shear wall are computed using the obtained storey seismic forces. The computed values of bending moments and shear forces in shear wall SW 2 are given in table 2.5. The computed axi forces according to the Bulgarian codes are given in the same table.

SHEAR FORCES, AXIAL FORCES AND BENDING MOMENTS FOR SHEAR WALL SW 2

Table 2.5

STOREY LEVEL	TRANSLATION		TORSION		TOTAL		AXIAL FORCE
	SHEAR FORCE	BENDING MOMENT	SHEAR FORCE	BENDING MOMENT	SHEAR FORCE	BENDING MOMENT	
6	15.28	0	23.72	0	39.00	0	0
5	43.46	33.62	67.46	52.18	110.92	85.80	51.15
4	71.74	155.30	111.07	241.07	183.11	396.37	161.15
3	93.57	356.18	144.24	552.90	237.81	909.08	301.33
2	107.37	618.17	165.03	956.78	272.40	1574.95	441.50
1	114.25	875.86	175.39	1352.85	289.64	2228.71	581.68
0	114.25	1150.06	175.39	1773.79	289.64	2923.85	721.85

(x_m = 9.865 m; y_m = 6.453 m). The coordinates of the centre of the stiffness are slightly different. For example for the 6-th storey level - x_s = 15.487; y_s = 8.046 and for the 3-rd storey level x_s = 15.490; y_s = 8.019.

STOREY SEISMIC FORCES FOR TRANSLATIONAL VIBRATIONS IN TRANSVERSAL DIRECTION IN kN

Table 2.3

STOREY LEVEL	SHEAR WALLS					TOTAL
	SW 2	SW 5	SW 7	SW 8	SW 9	
6	15.28	50.86	50.86	25.53	31.07	173.60
5	28.18	93.82	93.82	47.07	57.31	320.20
4	28.28	94.17	94.17	47.25	57.53	321.40
3	21.83	67.12	67.12	35.67	42.95	234.70
2	13.80	42.44	42.44	22.56	27.16	148.40
1	6.88	21.16	21.16	11.25	13.55	74.00
	DISTRIBUTION COEFFICIENTS					
6 to 4	0.088	0.293	0.293	0.147	0.179	1.00
3 to 1	0.093	0.286	0.286	0.152	0.183	1.00

ADDITIONAL STOREY SEISMIC FORCES DUE TO TORSION

Table 2.4

STOREY	TORSIONAL MOMENT	SHEAR WALLS				
		SW 2	SW 5	SW 7	SW 8	SW 9
6	975.98	23.72	22.45	1.07	20.50	24.98
5	1800.16	43.74	41.40	1.98	37.80	46.08
4	1806.91	43.91	41.56	1.99	37.95	46.26
3	1320.19	32.87	28.52	1.32	27.59	33.14
2	834.75	20.79	18.03	0.83	17.45	20.95
1	416.25	10.36	8.99	0.42	8.70	10.45
	DISTRIBUTION COEFFICIENTS					
6 to 4		0.0243	0.0230	0.0011	0.0210	0.0256
3 to 1		0.0249	0.0216	0.0010	0.0219	0.0251

The computed value of $T_1=0.409$ sec. In this case, according to the BCBER, the seismic forces are computed only for the first mode of vibration. The dynamic coefficient β is

$$\beta = \frac{0.7}{0.409} = 1.71$$

According to the Amendment to the BCBER of 1972 for package lift-slab structures, β should be increased by a coefficient A. For a 5 storey building A=1.07. Then,

$$\beta = 1.71 \times 1.07 = 1.83$$

The modal response coefficient η_k for the first mode of vibration is determined accepting linear distribution with a maximum at the top level $\eta_n=1.35$. (Table 2.2).

The computed total seismic forces at each floor level and the total shear forces and bending moments are given in table 2.2.

The lateral forces are distributed to the vertical bearing components of the seismic resisting system, assuming that the floor slabs are infinitely rigid in their own plane, i.e.:

$$S_{jk} = \frac{K_{jk}}{\sum_{s=1}^{n} K_{sk}} S_k = \mu_{jk} S_k \tag{2.3}$$

where:

S_{jk} - the seismic force to vertical component j at k-th level;

K_{jk} - the lateral stiffness of the j-th vertical component (shear wall) for the k-th level:

s - the number of the storey (1 to n)

The lateral stiffnesses are evaluated as an inverse of the unit deflection coefficient taking into account both shear and bending deformations:

$$\delta_{jk} = \frac{C_1 H_k}{G F_j} + \frac{H_k^3}{3EI_j} \tag{2.4}$$

where: E,G - modulus of elasticity and shear modulus; H_k - height from foundation level to k-th level; I_j,F_j moment of inertia and cross-section area of the j-th shear wall; C_1 - coefficient of the cross-section ($C_1=1.20$ for rectangular cross-section). The cross-sections and the moments of inertia of the shear walls for the given example are constant. Therefore the distribution coefficients of the successive levels will change slightly. For simplification of the computation the distribution coefficients can be determined only for two or three levels. In this example the distribution coefficients are calculated for the 3-rd and the 6-th levels. Their values are listed in table 2.3 together with the distributed seismic forces to the shear walls, for the transversal direction.

The storey torsional moment due to eccentricity between the centres of mass and stiffness is computed as a storey seismic force times the eccentricity, perpendicular to the considered direction of excitation:

$$M_{tk} = S_k e_k \tag{2.5}$$

where: M_{tk} - storey torsional moment at k-th storey; S_k, e_k - storey seismic force and eccentricity at k-th storey.

The coordinates of the centre of the mass are the same for each storey

Table 2.1

SHEAR WALLS		LENGTH $\|m\|$	THICKNESS $\|m\|$	X $\|m\|$	Y $\|m\|$	HEIGHT $\|m\|$
TRANSVERSAL DIRECTION	SW 2	2.975	0.25	0.125	7.887	15.40
	SW 5	4.50	0.25	11.150	4.475	15.40
	SW 7	4.50	0.25	15,700	4.475	15.40
	SW 8	3.56	0.25	23.375	11.525	15.40
	SW 9	3.80	0.25	23.375	4.250	15.40
LONGITUDINAL DIRECTION	SW 1	3.40	0.25	1.950	13.550	15.40
	SW 3	3.40	0.25	1.950	2.500	15.40
	SW 4	4.80	0.25	13.425	8.350	15.40
	SW 6	1.70	0.25	12.125	2.350	15.40

Table 2.2

STOREY LEVEL k	DESIGN LOADS		η_k	SEISMIC FORCES $\|kN\|$	TOTAL SHEAR FORCES $\|kN\|$	TOTAL BENDING MOMENTS $\|kNm\|$
	UNIFORMLY DISTRIBUTED $\|kN/m^2\|$	TOTAL $\|kN\|$				
6	4.20	1403	1.350	173.6	173.6	0.0
5	9.04	3020	1.157	320.2	493.8	381.9
4	11.51	3845	0.912	321.4	815.2	1764.6
3	11.51	3845	0.667	234.7	1049.9	4047.1
2	11.51	3845	0.421	148.4	1198.3	6986.8
1	11.51	3845	0.210	74.0	1272.3	9862.8
0	-	-	0.0	0.0	1272.3	12916.3

All vertical bearing elements should start from the foundation and should not be interrupted.

2.3 STRUCTURAL ANALYSIS

A 5 storey residential building is presented as a design example. It is situated in a zone of design intensity of VIII, according to the MSK scale. The local soil conditions are classified in group 3 in accordance with the Bulgarian Code for Buildings in Earthquake Regions - 1964 (BCBER) |1| The corresponding seismic design coefficient for the given intensity and the soi conditions is K_c=0.050. Some other Bulgarian codes and design regulations, connected with the presented design example are used |2,3,4,5|

The typical floor plan of the building with location of the shear walls is shown on fig. 2.1. The shear walls are of concrete C20 and have the same cros section along the height of the building. The geometric characteristics of th shear walls are presented in table 2.1.

The dynamic model of the building is assumed to be a fixed base cantilever beam with lumped masses at the level of the floor slabs.

The computation only for the transversal direction of excitation is presented The same principles are applied for the longitudinal direction. The computed internal forces for both directions are considered separately.

2.3.1 Design Loads

According to the BCBER the design gravity loads considered in the computation of the horizontal earthquake forces include the specified dead load and the effective live load and snow load. The corresponding load factors are:

- dead load of all permanent structural and nonstructural components - n=1.0
- live load on the floor slabs - n=0.5
- snow load - n=0.8

The computed design load forces, concentrated at the storey levels are given in table 2.2.

2.3.2 Computation Techniques

The lateral seismic forces are computed in accordance with the following formula (BCBER):

$$S_{ik} = \beta_i \eta_{ik} K_c Q_k \tag{2.1}$$

where: S_{ik} - the design seismic force, applied at point k for the i-th mode of vibration; β_i - the spectral dynamic coefficient:

$$2.40 \geq \beta_i = \frac{0.7}{T_i} \geq 0.8 \tag{2.2}$$

T_i - the natural period of the i-th mode of vibration; K_c - the seismic desig coefficient; Q_k - the effective gravity load at k-th level; η_{ik} - the modal response coefficient of the i-th mode, for the k-th level.

2. PACKAGE LIFT-SLAB SYSTEM

2.1 SCOPE

The present example deals with earthquake resistant design of Package Lift-Slab System, originally developed in Bulgaria. Structural analysis of a 5 storey building is presented. It involves computation of earthquake forces, internal forces and proportioning of a selected shear wall, in accordance with the National Design Codes.

2.2 DESCRIPTION OF PACKAGE LIFT-SLAB SYSTEM

The package lift-slab is a Bulgarian system for construction of residential public and industrial buildings. All reinforced concrete slabs of the building are cast-in-place one above the other at the ground level and then the whole package is lifted in height by special equipment. The storey slabs, supported by prefabricated reinforced concrete columns, are placed one by one at their design story levels. The loading capacity at one point provided by the lifting equipment is 600 kN. Grouping of two or more lifting devices at one point, permits increasing of the point loads.

The maximum number of storeys for equipment with 600 kN point load is the following:

- one section buildings - up to 14 storeys
- multisectional buildings - up to 12 storeys
- public buildings - up to 10 storeys
- industrial buildings - up to 8 storeys

The floor structure is a flat slab type. The column to slab connection is designed as a hinged one. The thickness of the slab is from 14 to 16 cm for residential buildings and from 18 to 25 cm for public buildings. The slabs are made of concrete with compression strength from 20 to 40 MPa.

The columns are prefabricated and have rectangular cross-section and height equal to the clear storey height, reduced by 2 cm (the thickness of the mortar between the column and the slabs). The strength of the concrete for columns should not be smaller than 30 MPa. The columns have cross-sections from 25/40 to 30/60, and they are designed to carry only gravity loads.

The vertical bearing elements - stair-wells and shear walls are designed to resist to the lateral seismic loads. The floor slabs are considered as rigid horizontal diaphragms transferring the seismic forces to the vertical bearing elements.

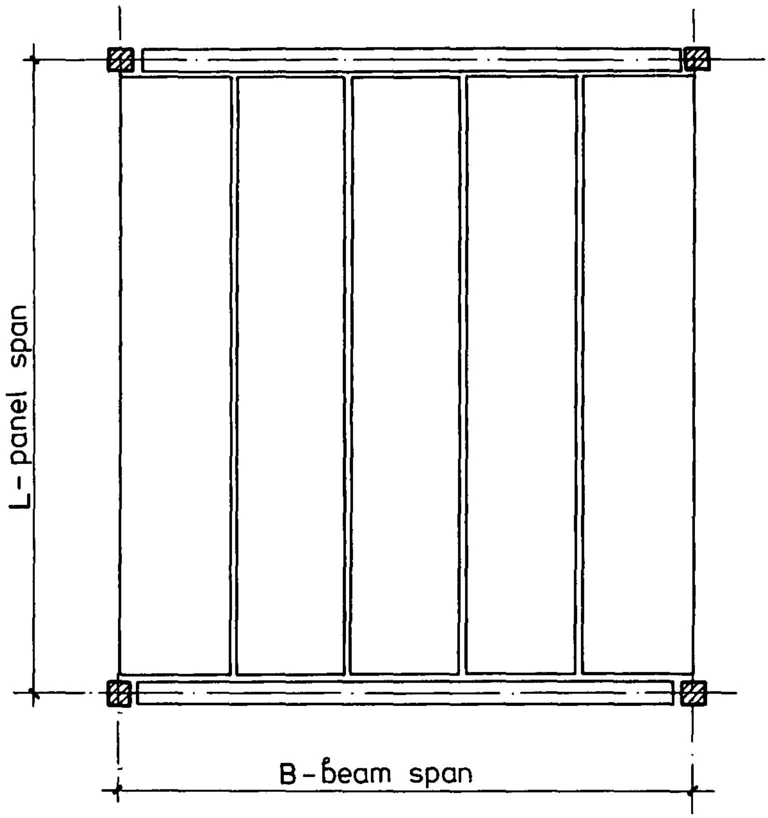
The stair-wells are the main bearing elements for lateral loads. They are usually made of reinforced concrete with thickness not less than 20 cm. During the lifting process the stair-wells serve as guiding and bracing elements. The stair-wells are founded either on stripe footings or on foundation slabs.

The shear walls are cast-in-place reinforced concrete structural walls usually situated between prefabricated columns. The reinforcement of the shear walls is concentrated near the columns - fig. 2.3. It passes through the slab and it is connected with the reinforcement of the upper floor by development length of 40 ϕ, but not less than 0.2 of the floor height. The reinforcement can also be connected by welding. Openings in the slabs with a diameter of ϕ 150 mm, at a distance of 50-80 cm are left for connection and concreting of the shear walls. The connection reinforcement is computed, but it should not be less than 2 ϕ 14 mm in one opening.

2. REPRESENTATIVE EXAMPLE OF PACKAGE LIFT-SLAB BUILDING SYSTEM BULGARIA

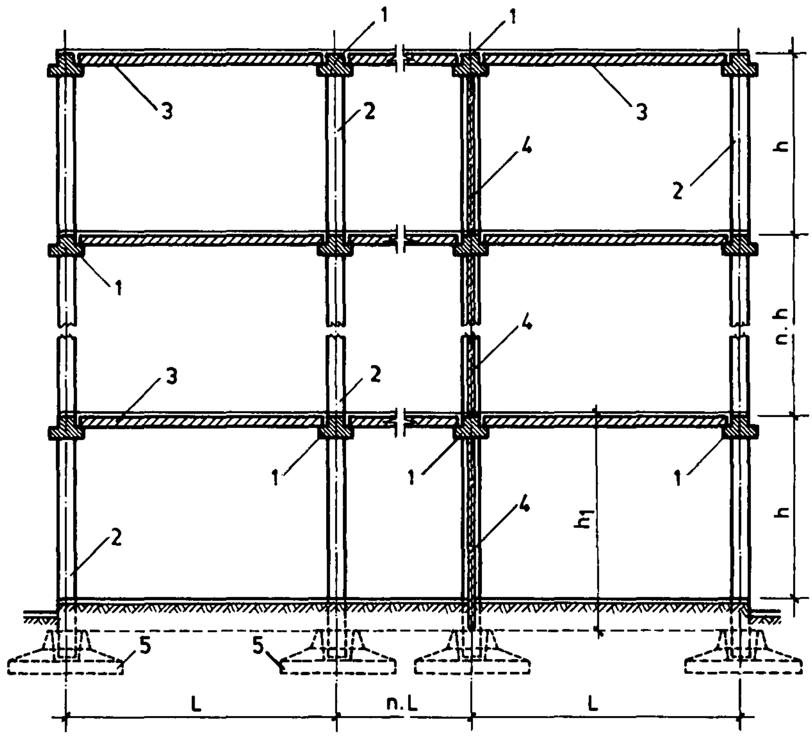
Prepared by S. Simeonov, P. Sotirov, N. Ignatiev and M. Dimitrov

Editor: P. Sotirov



L \ B	4,80	6,00	7,20	live load
4,80	•	•	•	15,00 kN/m²
6,00	•	•	•	15,00 kN/m²
7,20	•	•	•	10,00 kN/m²
8,40	•	•	•	5,00 kN/m²
9,60	•	•	•	3,00 kN/m²

fig. 3.1



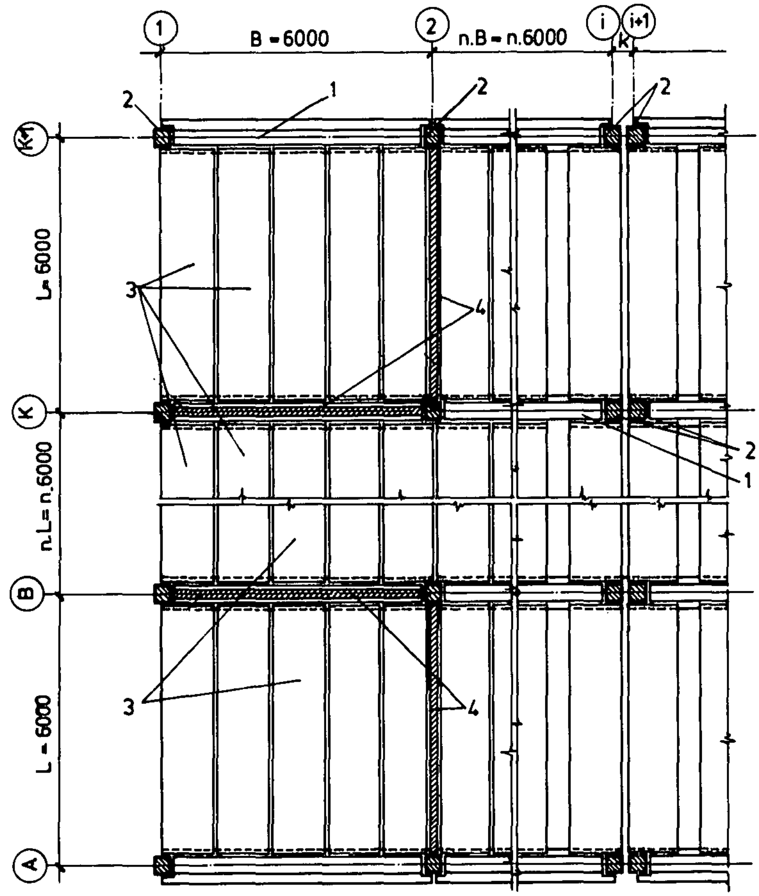
L = 4,80; 6,00; 7,20; 8,40; 9,60 m

h = 3,00; 3,30; 3,60; 4,20; 4,80; 5,40; 6,00 m

$h_1 = h + 0{,}60$

1.– beam
2.– column
3.– floor panel
4.– shear wall
5.– footing

fig. 3.2



1.- beam
2.- column
3.- floor panel
4.- shear wall

fig. 3.3

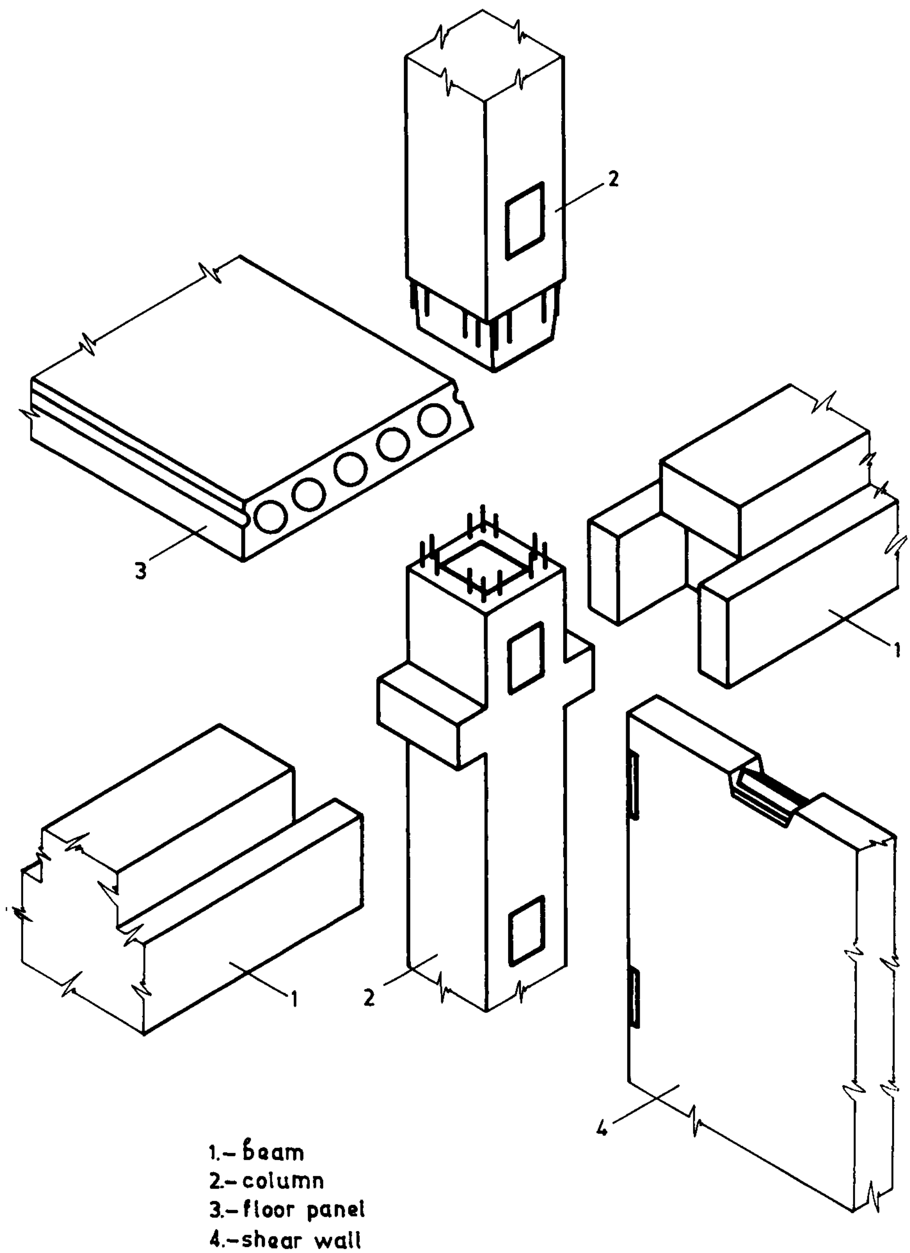
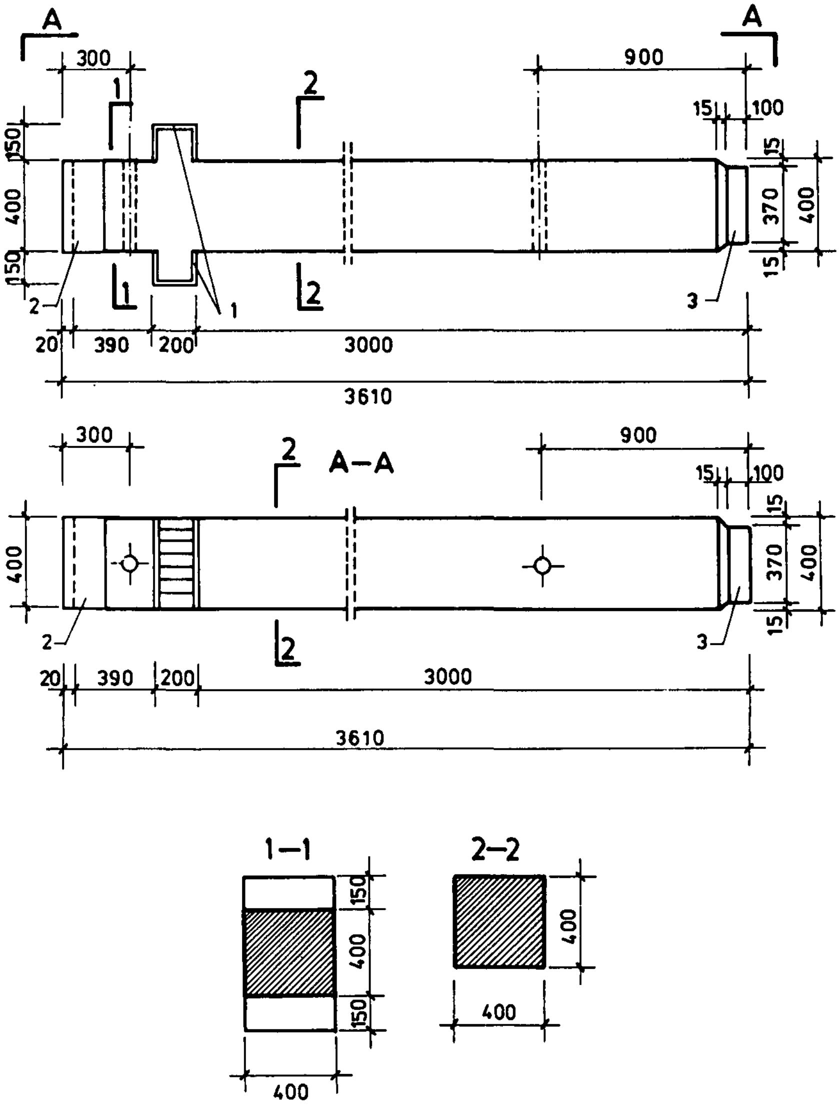


fig. 3.4



1.-embedded steel plate
2.-embedded bottom steel shoe
3.-embedded top steel shoe

fig. 3.5

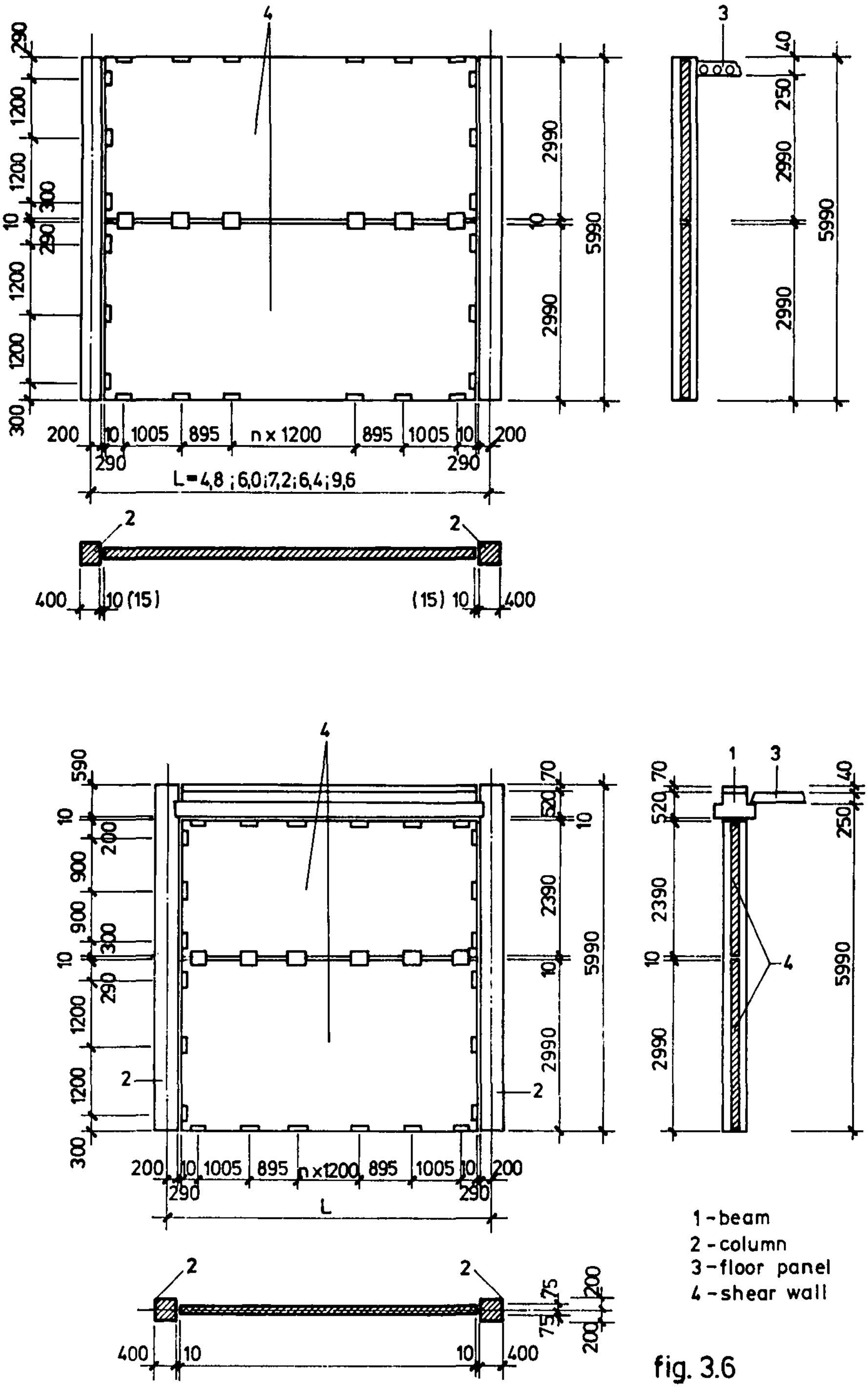


fig. 3.6

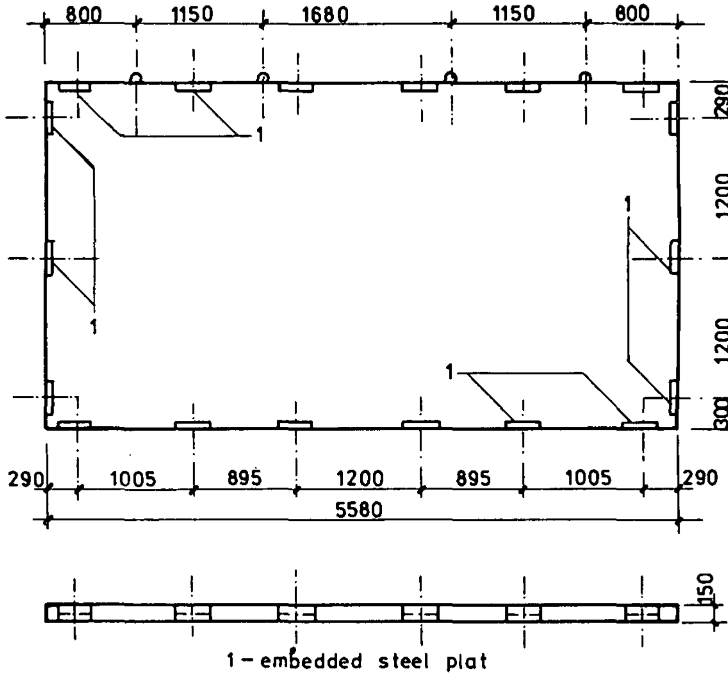

800
1150
1680
1150
800
290
1200
1200
300
2990
1
1
1
1
290
1005
895
1200
895
1005
290
5580
150
1 – embedded steel plat

fig. 3.7

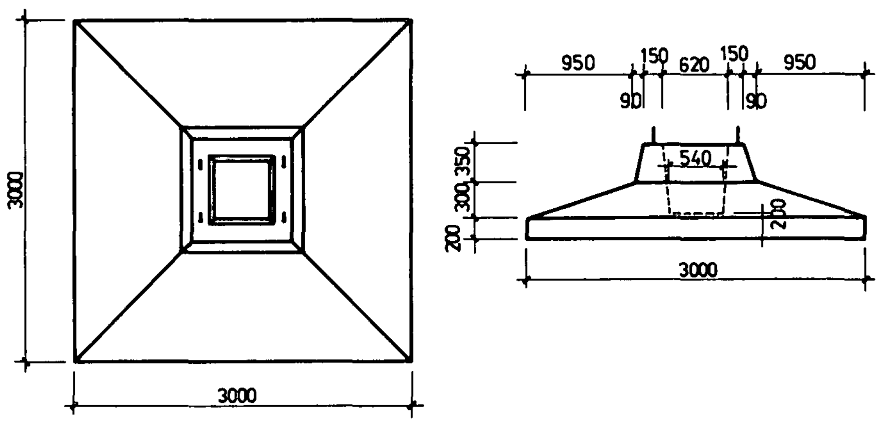

3000
3000
950
150
620
150
950
90
90
540
350
300
200
200
3000

fig. 3.8

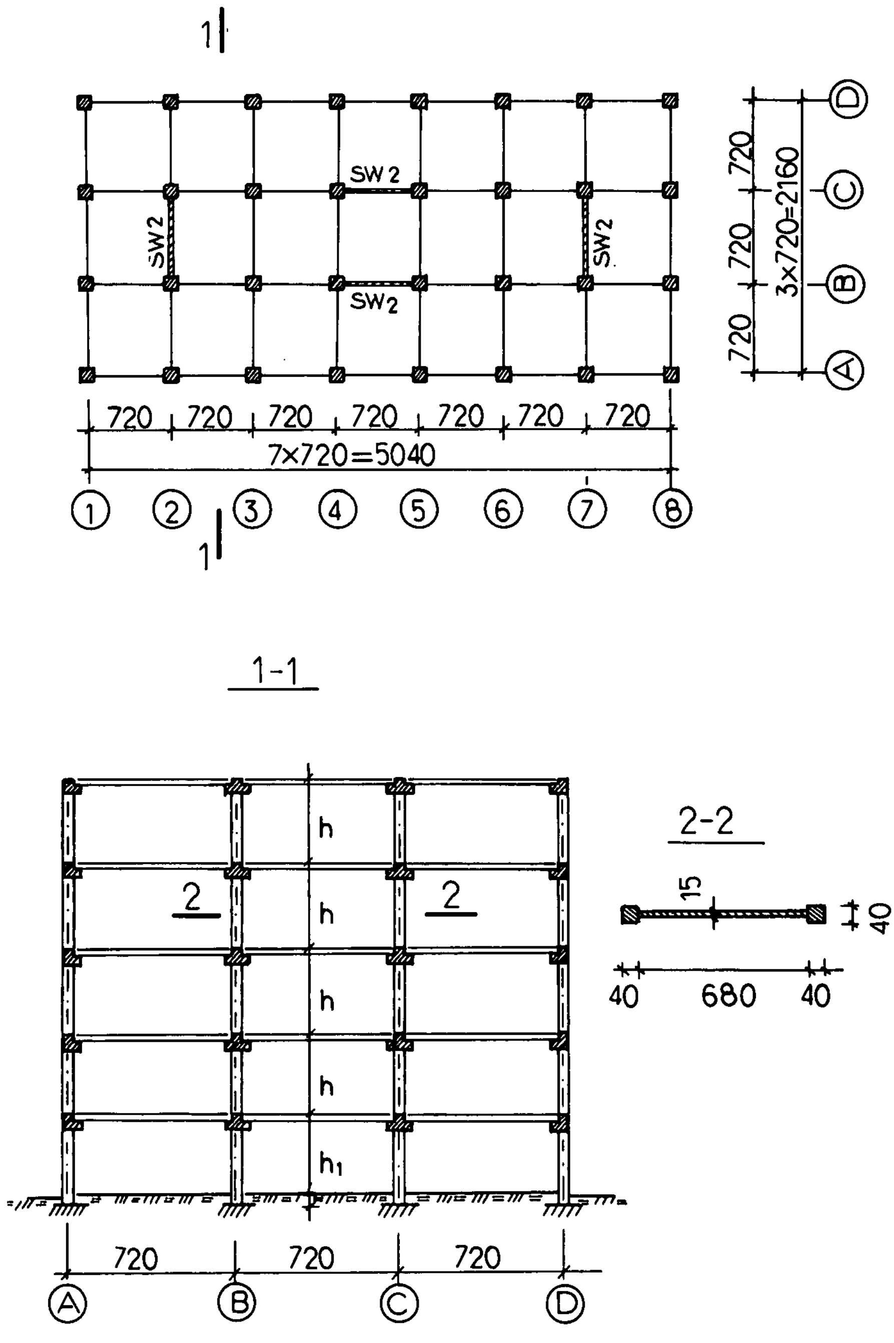
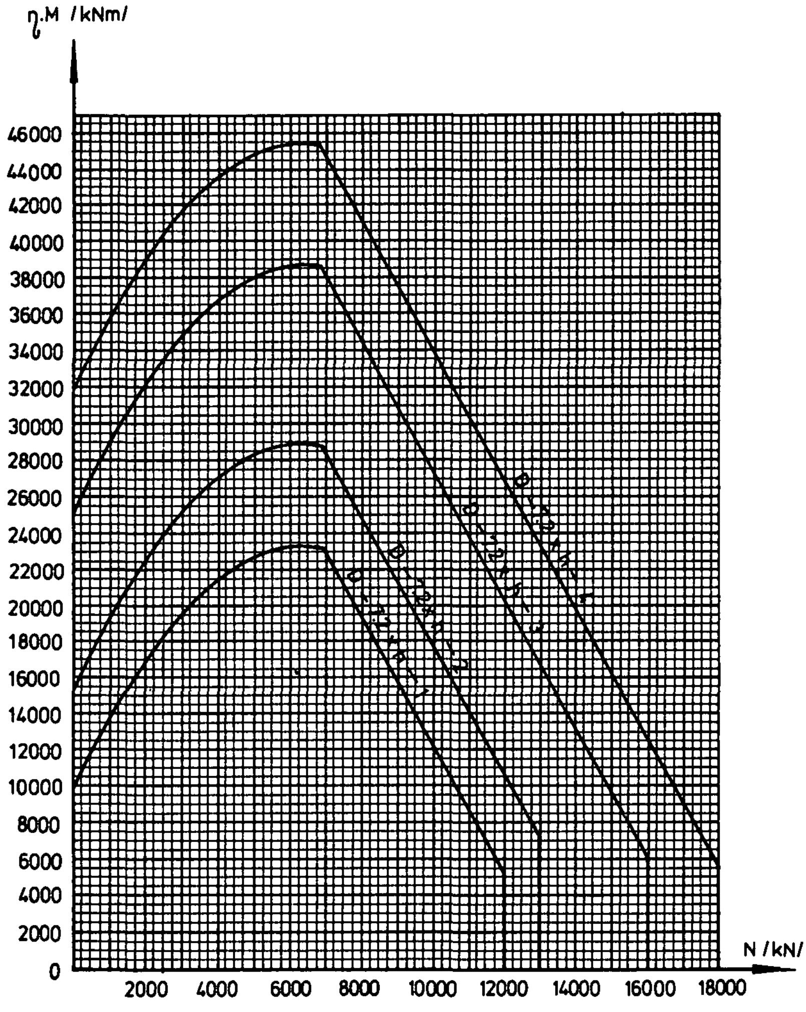

1
SW 2
SW2
SW2
SW2
720
720
720
3×720=2160
720
7×720=5040
1
2
3
4
5
6
7
8
A
B
C
D
1
1-1
h
h
h
h
h_1
2
2
2-2
15
40
40
680
40

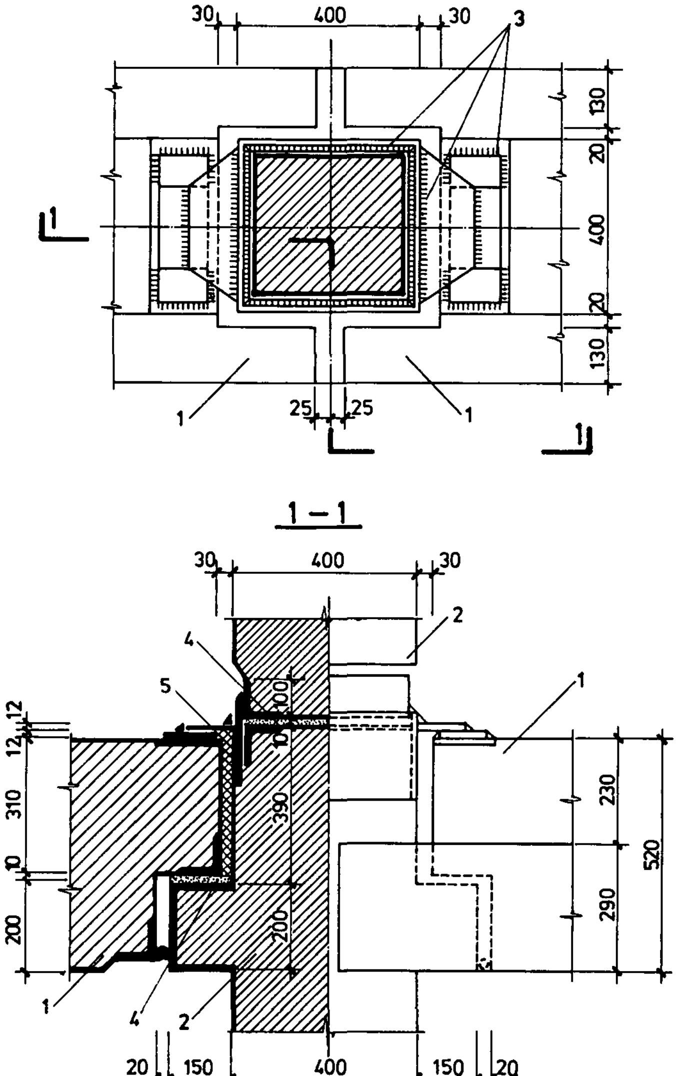
fig. 3.9



Qmax = 1430 /kN/

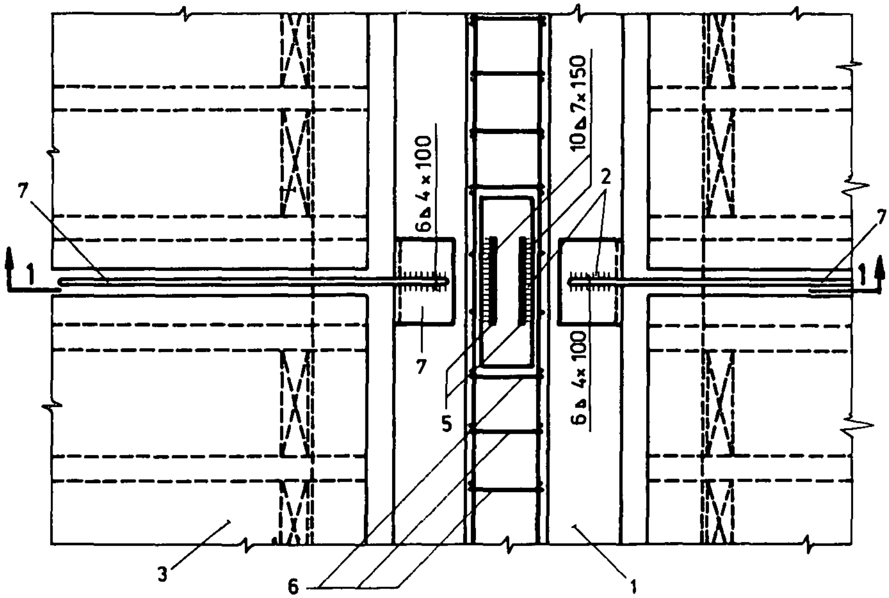
Bearing capacity of shear wall D-7.2xh-n

fig 3.10



1 – beam
2 – column
3 – welding
4 – 1cm mortar
5 – concrete 20

fig. 3.11



1 - Beam
2 - welding
3 - floor panel
4 - shear wall
5 - steel connection
6 - reinforcement
7 - embedded steel angle
8 - mortar
9 - concrete 20

1 — 1

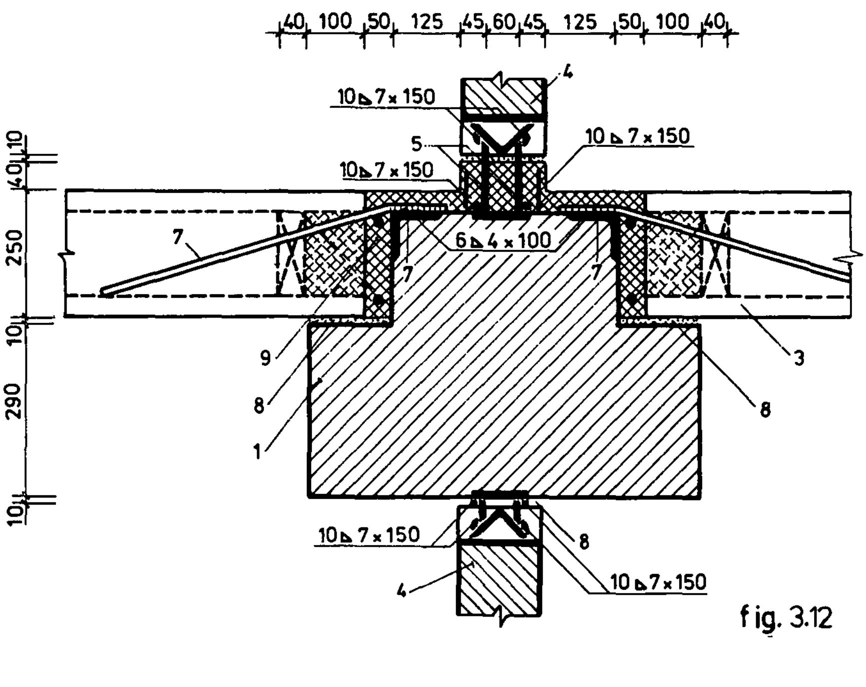


fig. 3.12

4. REPRESENTATIVE DESIGN EXAMPLE OF R. C. LARGE PANEL BUILDING GREECE

Prepared by D.G. TSOUKANTAS and O.C. VAGELATOU

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4.1. SCOPE

The purpose of this design example is to show the recent tendencies in Greece regarding the manner in which problems related to the behaviour of "full prefabricated" large panel buildings built in seismic areas, are treated.

According to the present stage of (international) knowledge, the "structural analysis" of prefabricated large panel buildings is carried-out under the assumption of m o n o l i t h i c, e l a s t i c behaviour of the whole structure; thus, for the Analysis, the traditional computing methods as for monolithic "cast in-situ" wall structures are used, taking into account actions (earthquake included) according to specific National Codes.

The major difference, concerning the design, between prefabricated R.C. large panels' structures and traditional monolithic cast in-situ R.C. wall buildings, is reflected in the proportioning of the specific parts of the structure which characterizes these two basic construction methods.

Undoubtably the "design" of connections is of fundamental importance when dealing with prefabricated buildings, especially under seismic conditions; notwithstanding the importance of the relevant construction problems.

That is why in the design example presented hereafter, emphasis is given to the proportioning of characteristic "vertical" and "horizontal" connections of a 7-storeys high R.C. large panel building.

In order to assist the reader to follow step by step the way in which the proportioning of vertical and horizontal joints was made a short summary of a relevant guide-text is presented in Annex II.

4.2. DESCRIPTION OF THE STRUCTURAL SYSTEM

4.2.1. General remarks

In Greece construction Companies are allowed to select any type of structures (using local or foreign licenses) under the condition that the local design and construction specifications and rules shall be respected.

For this reason, the design example presented here does not represent a "representative" system of any construction company. As a consequence, this example will be limited only in the way in which the structural system of a R.C. large panel construction is treated, from the point of view "structural Analysis" and "proportioning", and it will not deal with construction and other details which would possibly characterise a specific "system".

4.2.2. Structural system

The structural system under investigation is a "two-way"(*) large panel system. All bearing members of the structure are made by precast large

(*) Note that "cross-wall" as well as "long-wall" large panel systems are not encouraged in Greece due to the high seismic risk in the Country. Pref. structures in which some or all vertical walls are cast in-situ and/or Pref. structures with the use of monolithic slabs cast on top of thin precast slabs are much more prefered.

panels. For the superstructure, in-situ concrete is used only in the joints between the large panels in order to connect them.

The external walls are provided with two layers, an internal thicker bearing layer and an external thinner not structural layer securing the insulation of the structure.

The internal walls, the bearing part of the external walls as well as the slabs are made by plain precast reinforced concrete of quality C20 (characteristic strength). For the reinforcement of the precast members steel grade S42 (42/50) is used with yield stress $f_{sy} \approx 420$ MPa, while for the reinforcement of the connections steel grade S22 (22/34) is used (yield stress $f_{sy} \approx 240$ MPa).

A concrete grade C20 is used for the connections. The foundations of the building are made of in-situ reinforced concrete and are detailed (according to the soil conditions) in correspondance with the assumptions made during the Analysis of the structure.

The external and internal dimensions of the structural system, as well as the arrangement of vertical walls and slabs are shown in Fig. 4.1.

In Fig. 4.2 a cross section of the building is shown.

4.3. STRUCTURAL ANALYSIS

4.3.1. Design loads (according to the Greek Code)

a) Vertical loads

Dead loads: R.C. members (g) 24 KN/m^3
Floor finishing (g') 1.0 KN/m^2
Live loads (q): Floors 2.0 KN/m^2
Staircase 3.5 KN/m^2

b) Horizontal loads

- Wind forces
  In Fig. 4.3 the wind loads according Code are shown as well as the distribution of wind forces in the x-axis and y-axis of the building.
- Seismic actions
  - Base shear coefficient C = 0.06
  - Distribution of seismic forces (see Fig. 4.4)
    A triangular distribution of the seismic forces F along the height of the building has been taken into account according to the following relationship:

$$F_i = V \frac{W_i h_i}{\sum_{i=1}^{n} W_i h_i}$$

where

$$V = C \cdot \sum_{i=1}^{n} W_i \quad \text{(base shear)}$$

$W_i = G_i + Q_i$

G_i = vertical dead load in the i-level

Q_i = vertical live load in the i-level

h_i = distance of the level i from the level ±0.0

4.3.2. Analysis Technique

The Analysis of the structure against vertical and lateral loads has been carried out with EQUIVALENT STATIC METHOD, using the computing programme ETABS (of the University of California-Berkeley) as modified by the Chair of Earthquake Engineering of the N.T.U. of Athens.

4.3.3. Results of the Analysis in characteristic members of the structure

The composite wall (2) - (2) (see Fig.4.1) has been selected as representative of the structural system.

In Fig. 4.10 an elevation of this composite wall is schematically presented.

The horizontal connections (J_1, J_2) between the third and fourth level and the vertical connection (J_3) in the fourth level of the composite wall (2) - (2) will be selected (see Fig4.10) to be checked.

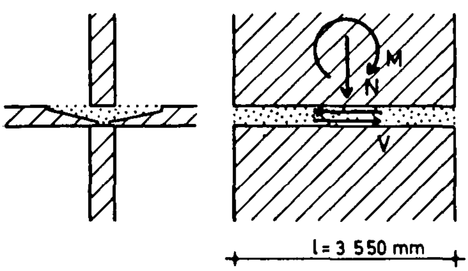
In Table 4.1, results for same types of connections (horizontal joints J_4 and J_5 and vertical joint J_6 - Fig. 4.10) of the first and second floor will be presented for comparison.

In Fig. 4.5 and Fig. 4.6 the distribution of the seismic and wind internal forces along the height of the composite wall (2) - (2) is presented, respectively. The internal forces are considered concentrated in the floor levels. Further distribution of these forces in the component walls I and II is shown in Fig. 4.7 and Fig. 4.8 for earthquake and wind actions respectively.

In Fig. 4.9 the vertical forces due to dead and live loads are presented, together with their eccentricity towards the wall-components I and II.

4.3.3.1. Load-actions on the horizontal joints

a) Joint J_1



N_g = -85.0 x 4 = - 340.0 KN

N_q = -19.1 x 4 = -76.4 KN

e_g = 0.2 m

e_q = 0.2 m

M_g = 340.0 x 0.2 = 68.0 KNm

M_q = 76.4 x 0.2 = 15.3 KNm

M_E = (21.0x4+12.4x3+10.1x2+7.9)x0.3 = = 447.9 KNm

M_W = (2.8x4+3.2x3+3.0x2+2.3)x3.0= = 87.3 KNm

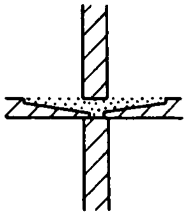
V_E = 21.0+12.4+10.1+7.9 = 51.4 KN

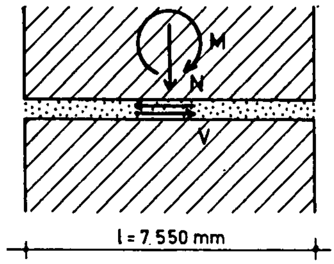
V_W = 2.8+3.2+3.0+2.3 = 11.3 KN

b) Joint J_2

N_g = -199.0 x 4 = -796.0 KN

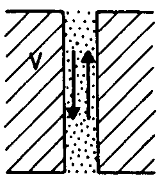
N_q = -39.9 x 4 = -159.6 KN





$e_g = 0.18$ m
$e^g = 0.18$ m
$M^q = 796.0 \times 0.18 = 143.3$ KNm
$M^g = 159.6 \times 0.18 = 28.7$ KNm
$M_E^q = (47.9 \times 4 + 39.3 \times 3 + 33.7 \times 2 + 27.8) \times 3.0 = 1214.1$ KNm
$M_W = (5.4 \times 4 + 9.7 \times 3 + 9.5 \times 2 + 8.0) \times 3.0 = 233.1$ KNm
$V_E = 47.9 + 39.3 + 33.7 + 27.8 = 148.7$ KN
$V_W = 5.4 + 9.7 + 9.5 + 8.0 = 32.6$ KN

4.3.3.2. Load-actions on the vertical joint J_3



	Wall 2 - 2	Wall B - B
V_g	= 4.9 KN	-
V^g	= 1.0 KN	-
V_E^q	= 66.7 KN	81.9 KN
V_W	= 14.6 KN	10.2 KN

4.4. PROPORTIONING (according to the guide-text in Annex II)

4.4.1. Proportioning of horizontal joints

4.4.1.1. Horizontal Joint J_1 (see Fig.4.10)

- Combination of actions to be considered

$\min N_u = -1.35 \times 340.0 - 1.5 \times 76.4 = -573.6$ KN

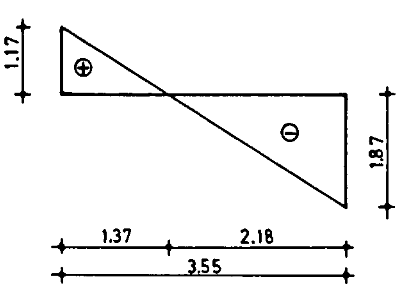
$\max N_u = -1.00 \times 340.0 + 1.5 \times 3 \times 0.06 \times 340.0 = -248.2$ KN

$$M_u = \max \begin{cases} 1.35 \times 68.0 + 1.5(15.3 + 0.5 \times 0.7 \times 447.9 + 0.5 \times 87.3) = 415.4 \text{ KNm} \\ 1.35 \times 68.0 + 1.5(0.7 \times 447.9 + 0.4 \times 15.3 + 0.5 \times 87.3) = 636.8 \text{ KNm} \\ 1.35 \times 68.0 + 1.5(87.3 + 0.4 \times 15.3 + 0.5 \times 0.7 \times 447.9) = 467.1 \text{ KNm} \end{cases}$$

$$V_u = \max \begin{cases} 1.5 \times (0.7 \times 51.4 + 0.5 \times 11.3) = 62.4 \text{ KN} \\ 1.5 \times (11.3 + 0.5 \times 0.7 \times 51.4) = 43.9 \text{ KN} \end{cases}$$

- Normal stresses (estimation of l_t, l_c)

$$\sigma_{cc}^{ct} = \frac{\max N_u}{A_j} \pm \frac{M_u}{W_j} = -\frac{248.2}{0.71 \times 10^3} \pm \frac{636.8}{0.42 \times 10^3} = \begin{cases} \rightarrow 1.17 \text{ MPa} \\ \rightarrow -1.87 \text{ MPa} \end{cases}$$



- Check versus tension

$t_j = 0.2$ m (width of the joint)

Tensile force:

$Z = \frac{1}{2} \times 0.2 \times 1.37 \times 1.17 \times 10^3 = 160.3$ KN

$A_s = \frac{160.3}{24} = 6.68 \text{ cm}^2$

- Check versus shear in the tension area

$$\tau_{act,u} = \frac{V_u}{l_j t_j} = \frac{62.4}{3.55 \times 0.2 \times 10^3} = 0.09 \text{ MPa}$$

$$\tau_{R,t} = \gamma_{n,E} \frac{\Delta A_s f_{sy}}{l_t t_j} = 0.45 \frac{\Delta A_s \times 240}{0.20 \times 1.37 \times 10^4} = 3.94 \times 10^{-2} \Delta A_s$$

$$\tau_{R,t} \geqslant \tau_{act,u} \rightarrow \Delta A_s = 2.28 \text{ cm}^2$$

Total ammount of reinforcement for tension and shear combined

$A'_s = 6.68 + 2.28 = 8.96 \text{ cm}^2 \rightarrow 4\phi 18$ (see Fig. 4.11)

- Check versus shear in the compressive area of the joint

$$\tau_{act,u} = \frac{V_u}{l_j t_j} = \frac{62.4}{3.55 \times 0.2 \times 10^3} = 0.09 \text{ MPa}$$

$$\sigma_{cc,G} = \frac{\max N_u}{l_c t_j} = -\frac{248.2 \times 10^{-3}}{2.18 \times 0.2} = -0.57 \text{ MPa}$$

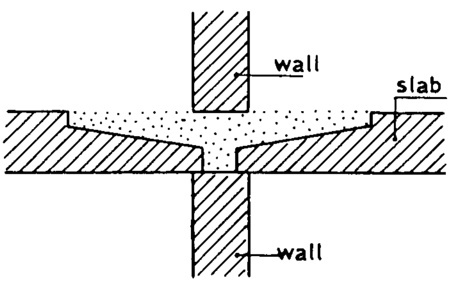
$\tau_o = 0.4$ MPa
$\sigma_o = -0.6$ MPa Qualification of contractor A-level (see Annex II)

$|\sigma_{cc,G}| < |\sigma_o|$ no need of reinforcement

$\tau_{act,u} < \tau_o$ (Nevertheless the joint is provided with a) loops $\phi 6/30$, b) one intermediate key, see Fig. 4.11)

- Check of the min $\sigma_{cc,u}$

$$\min \sigma_{cc,u} = \frac{N_u}{A_j} - \frac{M_u}{W_j} = -\frac{573.6 \times 10^{-3}}{0.71} - \frac{636.8 \times 10^{-3}}{0.42} = -2.33 \text{ MPa}$$



for the type of the connection used it is:

$f_{cc,u} \approx 0.8(1 - \frac{0.09}{2}) \times 20 = -15.3$ MPa < -2.33 MPa

- Horizontal tie-reinforcement in the horizontal joint $1\phi 18$

4.4.1.2. Horizontal joint J_2 (see Fig. 4.10)

- Combination of actions to be considered

$\min N_u = -1.35 \times 796 - 1.5 \times 159.6 = -1314.0$ KN

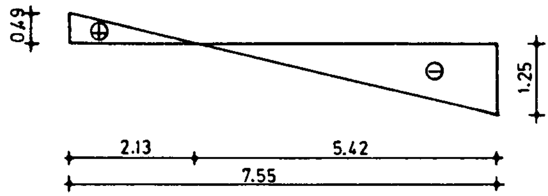
$\max N_u = -1.00 \times 796 + 1.5 \times 3 \times 0.06 \times 796.0 = -581.1$ KN

$$M_u = \max \begin{cases} 1.35 \times 143.3 + 1.5(28.7 + 0.5 \times 0.7 \times 1214.1 + 0.5 \times 233.1) = 1048.7 \text{ KNm} \\ 1.35 \times 143.3 + 1.5(0.7 \times 1214.1 + 0.5 \times 233.1 + 0.4 \times 28.7) = 1660.3 \text{ KNm} \\ 1.35 \times 143.3 + 1.5(233.1 + 0.4 \times 28.7 + 0.7 \times 0.5 \times 1214.1) = 1197.7 \text{ KNm} \end{cases}$$

$$V_u = \max \begin{cases} 1.5(0.7 \times 148.7 + 0.5 \times 32.6) = 180.6 \text{ KN} \\ 1.5(32.6 + 0.5 \times 0.7 \times 148.7) = 127.0 \text{ KN} \end{cases}$$

- Normal stresses (estimation of l_t, l_c)

$$\sigma_{cc}^{ct} = -\frac{581.1}{1.51 \times 10^3} \pm \frac{1660.3}{1.90 \times 10^3} = \begin{matrix} 0.49 \text{ MPa} \\ -1.25 \text{ MPa} \end{matrix}$$



- Check vs tension

$$Z = \frac{1}{2} \times 2.13 \times 0.49 \times 0.2 \times 10^3 = 104.4 \text{ KN}$$

$$A_s = 4.35 \text{ cm}^2$$

- Check vs shear in the tension area

$$\tau_{act,u} = \frac{180.6 \times 10^{-3}}{7.55 \times 0.2} = 0.12 \text{ MPa}$$

$$\tau_{R,t} = \frac{0.45 \times \Delta A_s \times 240}{0.2 \times 2.13 \times 10^4} = 2.54 \times 10^{-2} \, \Delta A_s$$

$$\tau_{R,t} \geqslant \tau_{act,u} \rightarrow \Delta A_s = 4.72 \text{ cm}^2$$

Total ammount of reinforcement for tension and shear combined
$A'_s = 4.35 + 4.72 = 9.07 \text{ cm}^2 \rightarrow 4\varnothing 18$ (see Fig. 4.11)

- Check versus shear in the compressive area of the joint

$$\tau_{act,u} = \frac{180.6 \times 10^{-3}}{7.55 \times 0.2} = 0.12 \text{ MPa} \qquad \tau_o = 0.4 \text{ MPa}$$

$$\sigma_{cc,G} = \frac{581.1 \times 10^{-3}}{0.2 \times 5.42} = 0.54 \text{ MPa} \qquad \sigma_o = 0.6 \text{ MPa}$$

$0.12 < 0.4$ no need of reinforcement
$|0.54| < |0.6|$ (Nevertheless the joint is provided with a) loops $\varnothing 6/30$
b) three intermediate keys, see Fig. 4.11)

- Check of the min $\sigma_{cc,u}$

$$\min \sigma_{cc,u} = -\frac{1314.0}{1.51 \times 10^3} - \frac{1660.3}{1.90 \times 10^3} = -1.74 \text{ MPa}$$

$$f_{cc,u} \approx 0.8 \left(1 - \frac{0.12}{2}\right) \times 20 = -15.68 \text{ MPa} < -1.74 \text{ MPa}$$

- Horizontal tie-reinforcement in the horizontal joint $1\varnothing 18$

4.4.2. Proportioning of the vertical joint J_3 (see Fig.4.10)

a) In the level of wall ② - ②

- Combination of actions

$$V_u = \max \begin{cases} 1.35 \times 4.9 + 1.5(1.0 + 0.7 \times 0.5 \times 66.7 + 0.5 \times 14.6) = 54.0 \text{ KN} \\ 1.35 \times 4.9 + 1.5(0.7 \times 66.7 + 0.5 \times 14.6 + 0.4 \times 1.0) = 88.2 \text{ KN} \\ 1.35 \times 4.9 + 1.5(14.6 + 0.4 \times 1.0 + 0.5 \times 0.7 \times 66.7) = 64.1 \text{ KN} \end{cases}$$

$$\tau_{act,u} = \frac{88.2 \times 10^{-3}}{2.81 \times 0.2} = 0.15 \text{ MPa} > 0.10 \text{ MPa}$$

keys needed

- Geometrical characteristics of the joint

it will be chosen:
4 keys ($n = 4$)
Loops Ø8 (Total $A_s = 4 \times 2 \times 0.5 = 4 \text{ cm}^2$)
$h_o = 200$ mm
$a = 25$ mm
$\alpha^o = 30^o$
(see Fig. 4.13) $l_j = 2810$ mm
$t_j = 200$ mm
density of the keys

$$\lambda = \frac{nh_o}{l_j} = \frac{4.20}{281} = 0.28$$

Concrete of the joint: $f_{ck} = 20$ MPa
Transversal reinforcement: $f_{sy} = 240$ MPa

- Resistance of the joint

$$\tau_{R,u} = \frac{1}{\gamma_c} \gamma_{nc}\, 0.1\, f_{ck} \lambda + \frac{1}{\gamma_s} \gamma_{ns} \left(\frac{A_s f_{sy} - N}{l_j t_j}\right)$$

$\gamma_c = 1.5$
$\gamma_s = 1.15$
$\gamma_{nc} = \gamma_{ns} = 0.55$ (for $C = 0.06$)
$N = 0$

$$\tau_{R,u} = \frac{1}{1.5} \times 0.55 \times 0.1 \times 20 \times 0.28 + \frac{1}{1.15} \times \frac{0.55 \times 4 \times 240}{281 \times 20} = 0.28 \text{ MPa}$$

- Check $\tau_{R,u} = 0.28 \text{ MPa} > 0.15 \text{ MPa} = \tau_{act,u}$
- Vertical reinforcement, 1Ø18
- Check of the anchorage length of the connecting bars

$$l_{ang} = \frac{50 \times 240}{\pi \times 8 \times 0.6 \times 2} \times 1.2 = 477 \text{ mm}$$

b) In the level of wall B - B

$\tau_{act,u} = 0.166$ MPa

$\tau_{R,u} = 0.28 \text{ MPa} > 0.166 \text{ MPa}$

Table 4.1: Results on joints J_1, J_2, J_3, J_4, J_5, J_6 (see Fig. 4.10)

A.- H O R I Z O N T A L   J O I N T S   (J_1, J_2 J_4, J_5) (see Fig.4.11)

		Joint checked	J_1	J_2	J_4	J_5
Actions		N_g(KN)	-340.0	-796.0	-510.0	-1194.0
		N_q(KN)	- 76.4	-159.6	-114.6	- 239.4
		M_g(KNm)	68.0	143.3	102.0	214.9
		M_q(KNm)	15.3	28.7	22.9	43.1
		M_E(KNm)	447.9	1214.1	804.9	2285.1
		M_W(KNm)	87.3	233.1	176.4	501.3
		V_E(KN)	51.4	148.7	61.8	186.7
		V_W(KN)	11.3	32.6	16.1	48.7
Checks in the tensile area	vs tension	A_s (cm^2)	6.68	4.35	12.93	10.63
	vs shear	ΔA_s(cm^2)	2.28	4.72	2.91	6.76
	Total	A'_s(cm^2)	8.96 4∅18	9.07 4∅18	15.84 5∅20	17.39 3∅20+2∅22
Checks in the area under compression	vs shear and compression combines	A_s(cm^2)	No need min A_s ∅6/30	No need min A_s ∅6/30	No need min A_s ∅6/30	No need min A_s ∅6/30

B.- V E R T I C A L   J O I N T S   (J_3, J_6)   (see Fig. 4.13)

Joint checked →		J_3		J_6	
		composite wall		composite wall	
		2-2	B-B	2-2	B-B
Actions	V_g(KN)	4.9	-	4.9	-
	V_q(KN)	1.0	-	1.0	-
	V_E(KN)	66.7	81.9	82.9	104.0
	V_W(KN)	14.6	10.2	21.6	15.6
Reinforcement (λ = 0.28)	A_s(cm^2)	4.0	4.0	4.0	4.0

4.5. CONCLUDING REMARKS

This design example in no way may be considered as complete. Nevertheless it reflects one of the design philosophies and structural analysis of low-rise R.C. large panel structures in Greece. The main purpose was to show the way in which the dimensioning of vertical and horizontal joints between prefabricated R.C. large panels subjected to seismic actions is carried out. It may be noted that the formulae used for the dimensioning of the joints as well as reductions factors γ_n used for the estimation of their strength degradation due to earthquake are based on relevant experimental and theoretical research (part of which may be seen in Annex I).

The following remarks are also worth mentioning:

- The structural configuration is chosen as regular as possible. The height of the building meets the restriction of Table 1 of Annex II (height versus seismicity).
- The structural analysis has been carried out under the assumption of monolithic behaviour. Non-linear characteristics of the structure has not been taken into account.

- Special care should be paid to the dimensioning and d e t a i l i n g of lintels (above the openings of the walls) since lintels also contribute to energy dissipation of the structural system. Experimental evidence (see also Annex I) assures the ductility of the joints used.

R E F E R E N C E S

1. Current Greek Code for seismic design of R.C. structures

2. Draft R.C. Code of Greece, (§19). Edition of NTU Athens, 1979 (in greek)

3. T.P. TASSIOS - G. GAZETAS: "Draft proposal for a new Greek Code for the design of R.C. structures in seismic regions", NTU Athens, 1979 (in greek)

4. CEB-FIP Model Code for concrete structures

5. S.G. TSOUKANTAS: "Investigation of the behaviour of R.C. large panels' joints under static and dynamic loading", NTU Athens, 1981,(Doctor Thesis) in greek.

6. T.P. TASSIOS - S.G. TSOUKANTAS: "Serviceability and ultimate limit-states of large panels' connections under static and dynamic loading". Proceedings of the RILEM-CEB-CIB Symposium on Mechanical and Insulating properties of joints of precast R.C. Elemenets, Vol. I, Athens, 1978.

7. S.G. TSOUKANTAS - T.P. TASSIOS - S. COUNADIS: "R.C. large panels' connections under fully reversed shearing actions", 5th Greek Concrete Symposium, Nicosia, Cyprus, 1981.

8. T.P. TASSIOS - S.G. TSOUKANTAS: "R.C. precast panels' connections under cyclic actions", 7th European Conference on Earthquake Engineering (ECEE), Athens, Sept. 1982.

9. T.P. TASSIOS - E. VINTZELEOU: "Shear transfere through dowel action under cyclic loading", 5th Greek Concrete Symposium, Nicosia, Cyprus 1981 (in greek).

N O T A T I O N S

Report		CEB
A_c	net concrete cross sectional area	A_c
A_s	area of the steel reinforcement	A_s
A_j	cross sectional area of a joint	-
E_c	modulus of elasticity of concrete	E_c
E	earthquake action	E
F	action in general	F
G	permanent action	G
I	second moment of a plane area	I
M	flexural moment	M
M_g	flexural moment due to permanent load	
M_q	flexural moment due to live load	
M_E	flexural moment due to earthquake	
M_W	flexural moment due to wind load	
N	nominal (axial) load	N
$N_{g,q,E,W}$	as for flexural moment	
Q	variable action	Q
R	strength (resisting load effect)	R
V	shear force	V
$V_{g,q,E,W}$	as for flexural moment	
W	wind load; modulus of inertia $(\frac{I}{y})$	W
c	concrete	c
e	eccentricity	e
f	strength of a material	f
g	distributed permanent load	g
l	length of an element	l
q	distributed variable load	q
s	steel	s
γ	safety factor	γ
λ	density of keys of a keyed joint	-
σ	axial stress	σ
τ	shear stress	τ
C	base shear coefficient	C

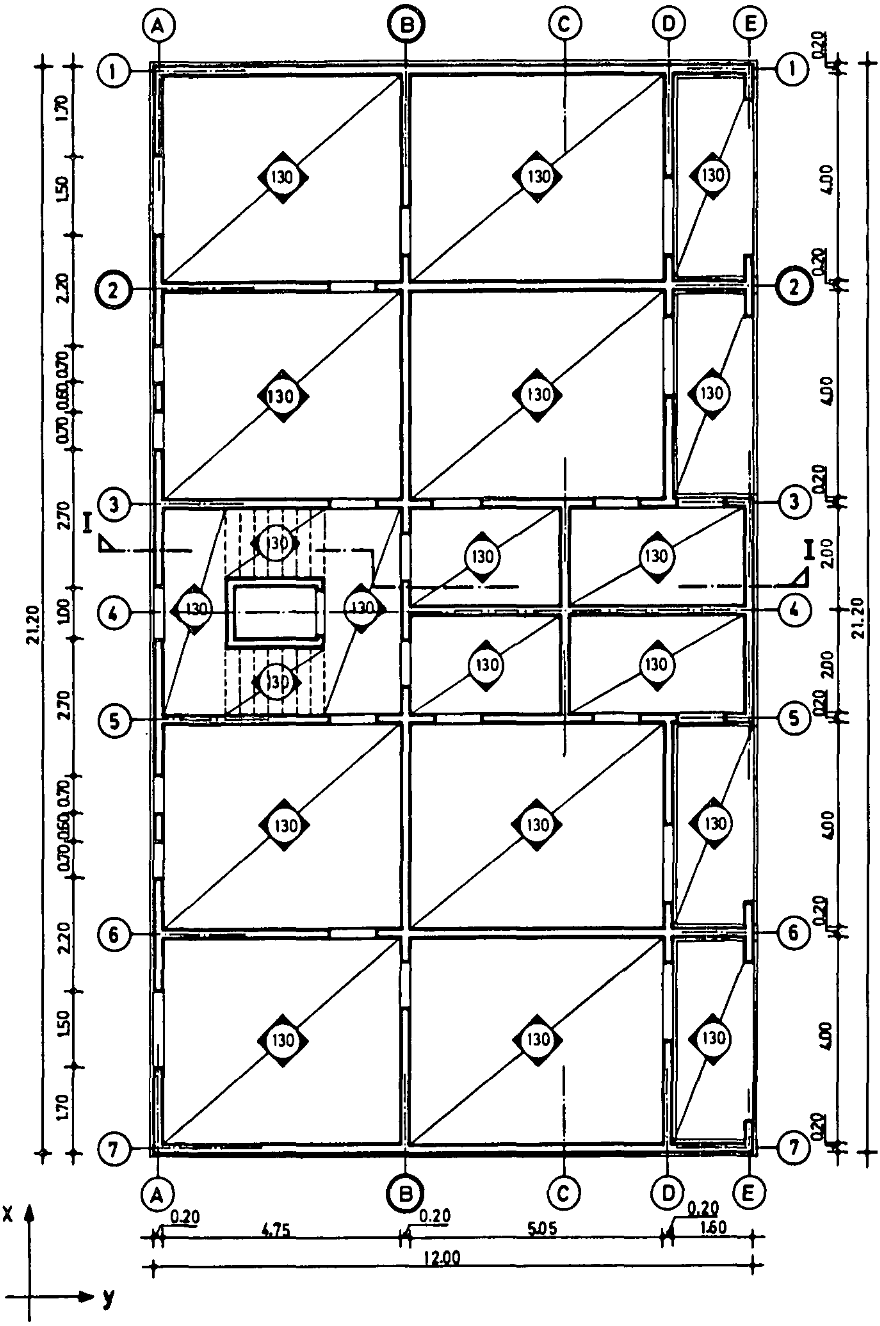


Fig. 4.1: TYPICAL FLOOR PLAN (Main dimensions in m slab's thickness in mm)

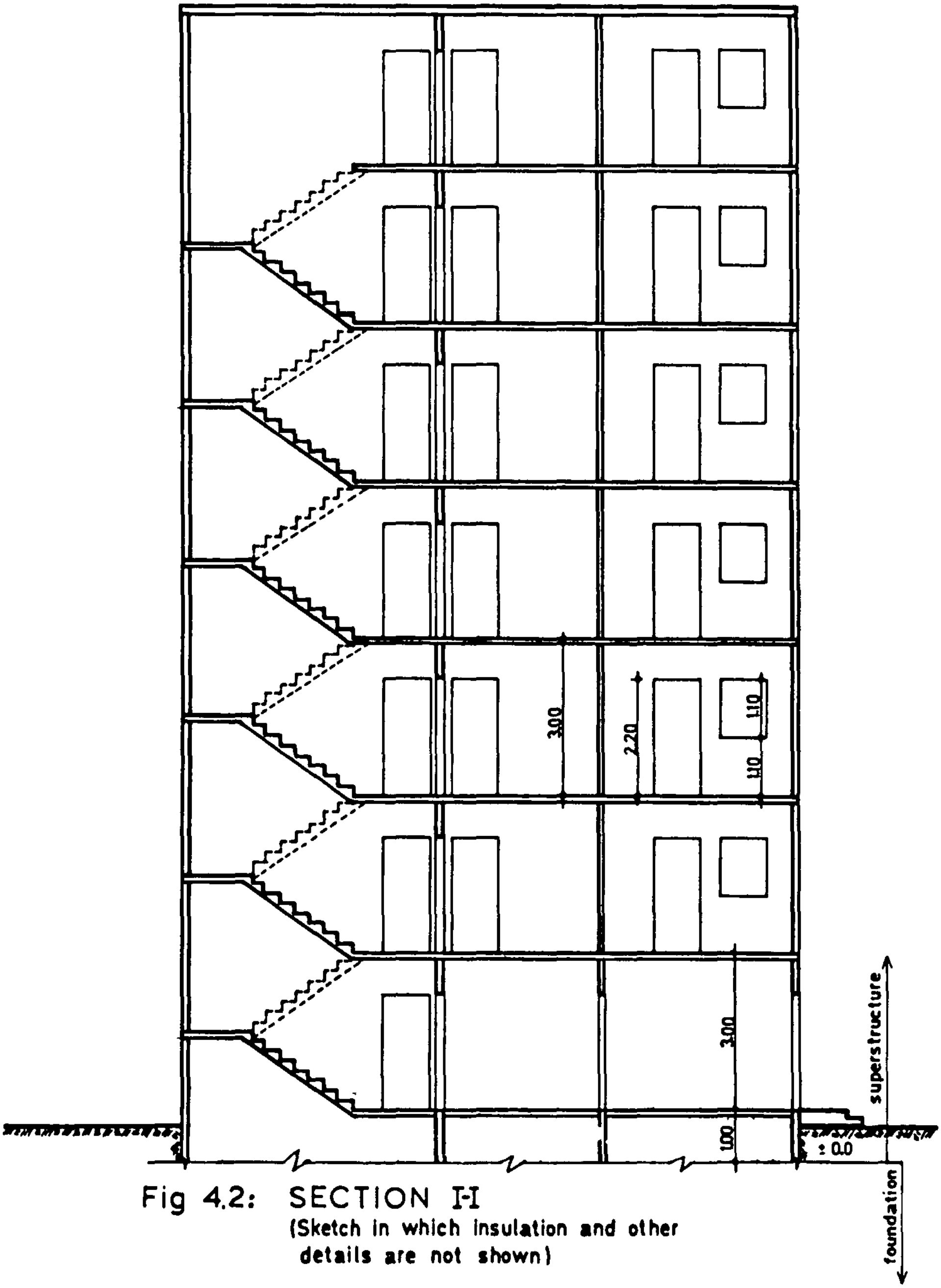


Fig 4.2: SECTION I-I
(Sketch in which insulation and other details are not shown)

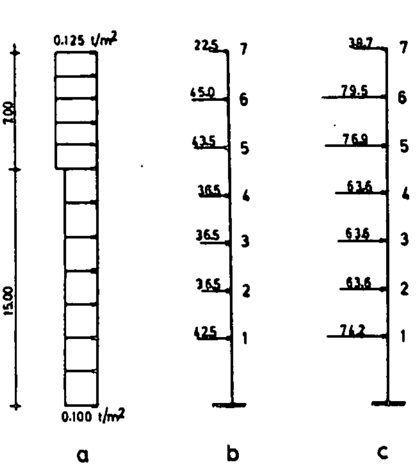


Fig. 4.3: a) Wind Loads according code (t/m^2)
b) Distribution of wind forces in the X-axis of the building (KN)
c) Distribution of wind forces in the y-axis of the building (KN)

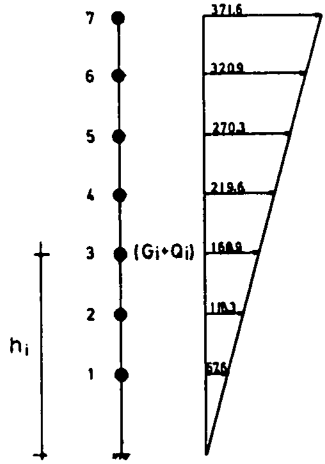


Fig.4.4: Distribution of equivalent seismic actions along the height of the building (KN)

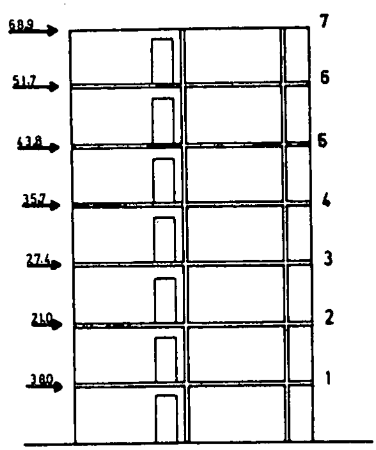


Fig.4.5: Distribution of the (internal) seismic forces (KN) along the height of the composite wall ② - ②

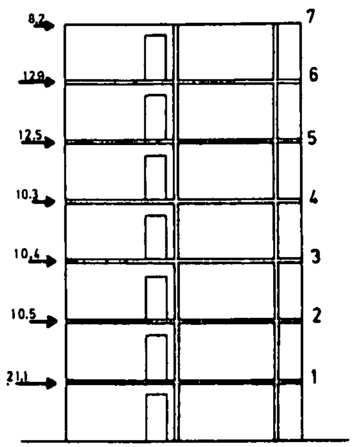


Fig.4.6: Distribution of the (internal) wind forces (KN) along the height of the composite wall ② - ②

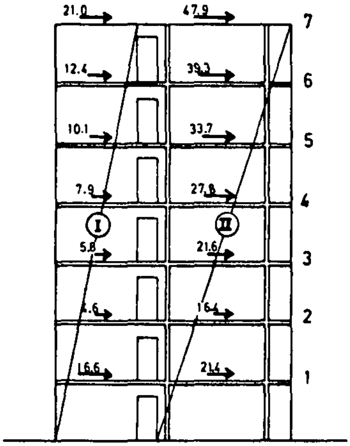


Fig.4.7: Distribution of the (internal) seismic forces (KN) in the component walls I and II

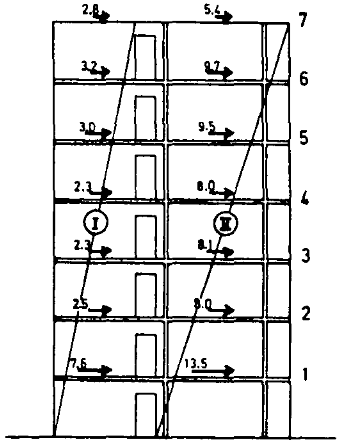


Fig.4.8: Distribution of the (internal) wind forces (KN) in the component walls I and II

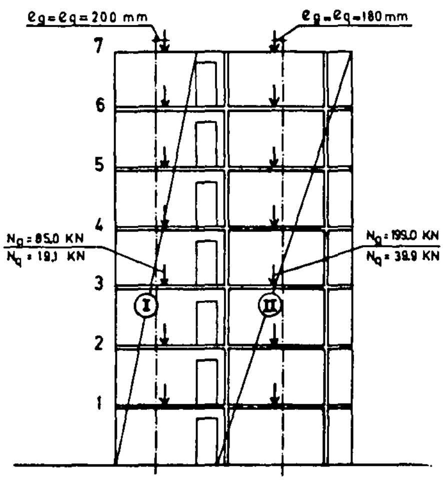


Fig.4.9: Vertical loading

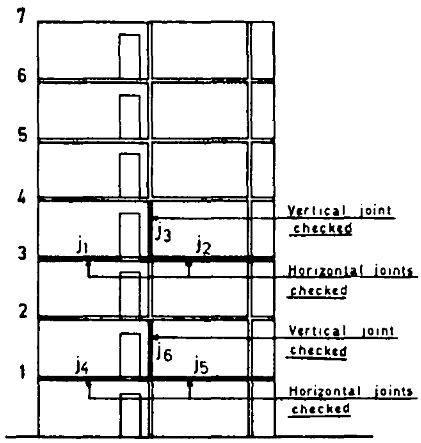


Fig.4.10: Composite wall ② - ② joints checked

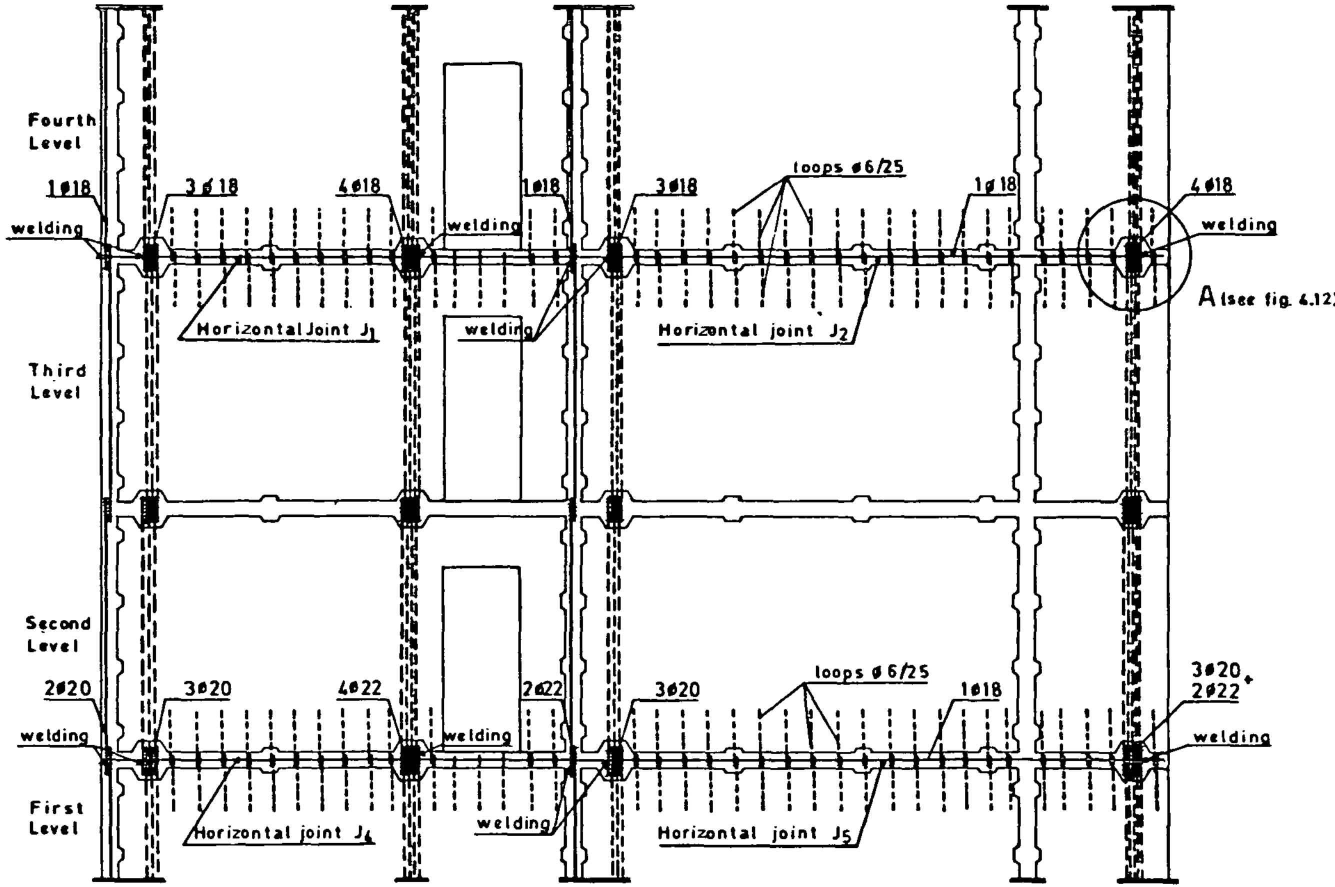


Fig. 4.11: Reinforcement of the horizontal joints J_1, J_2, J_4, J_5 only
(Other reinforcements and details are not shown)

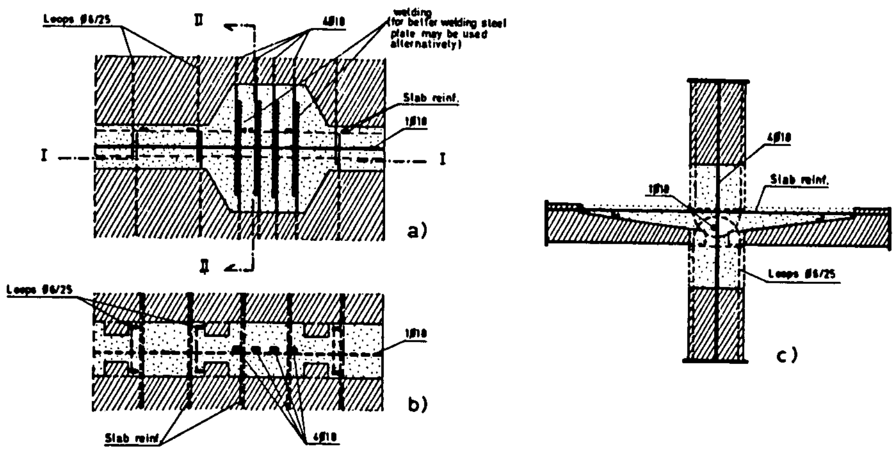


Fig.4.12: a) Detail A (see Fig.4.11)
b) Section I-I
c) Section II-II

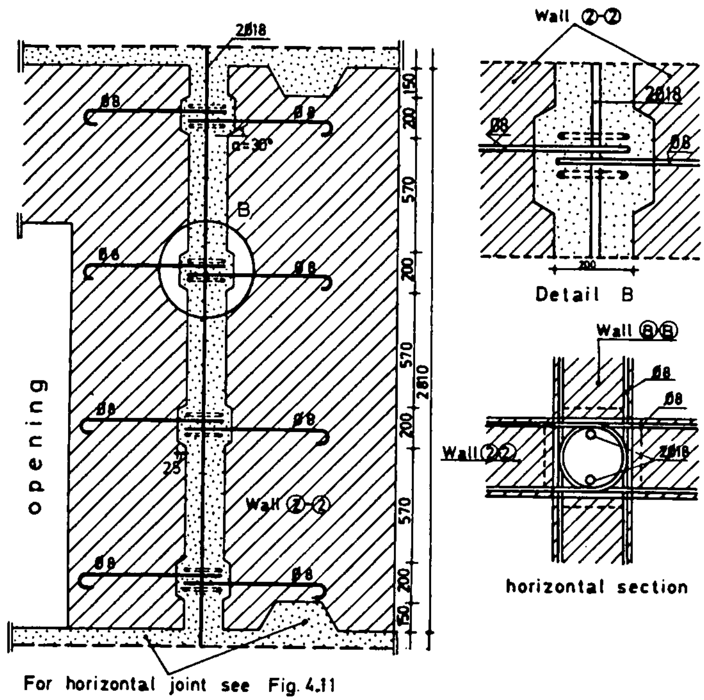


Fig.4.13: Reinforcement of vertical joint J_3. Other reinforcements and details are not shown (Longitudinal reinforcement 1Ø8 due to horizontal joint dimensioning, see Fig.4.11)

ANNEX I

STRUCTURAL BEHAVIOUR OF R.C. PRECAST PANELS'CONNECTIONS

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Dr. S.G. TSOUKANTAS, NTU Athens

1.- INTRODUCTION

The most vulnerable regions of buildings made of large precast panels, are the connections between these panels, especially under earthquake loading.

In Tassios - Tsoukantas (1978), as well as in Tsoukantas (1981), the mechanical behaviour of such connections under both monotonic and repetitive dynamic loading are presented. In Tsoukantas - Tassios - Kounadis (1981), as well as, more recently in Tassios - Tsoukantas (1982) experimental findings of some fully reversed actions are reported.

In this Annex a short presentation is made of the main results of the above mentioned reports completed with some more recent data, together with an attempt for quantitative conclusions regarding the seismic behaviour and design, of precast panels' joints.

2.- MONOTONIC LOADING

A theoretical model for the prediction of the ultimate bearing capacity of keyed shear connections provided with distributed connecting bars along the length of the joint is represented in Fig. 1 and it is described by Equ. 1:

$$\tau_u = \min \left\{ \begin{array}{l} \frac{1}{4}\frac{b}{h_o}\lambda^2 . f_{cc} \\ \\ 0{,}15 . \lambda . f_{cc} \end{array} \right\} + \mu_u . \rho(f_{sy} - \sigma_N) + 1{,}8\,\rho . f_{ct}\sqrt{f_{sy}}\ \text{MPa} \qquad [1]$$

SHEAR STRENGTH	Diagonal compressive strength within joint	Friction mobilised by transverse steel	Dowel action

where

b = joint width
h_o = height of keys (inside the panel)
λ = the density of the keys
f_{cc} = compressive strength of joint concrete
f_{ct} = tensile strength of joint concrete
f_{sy} = yield stress of the connecting bars
μ_u = friction coefficient, taken as follows

$\rho . f_{sy}$	10		15	
μ_u	1,0	0,8	0,8	0,6

ρ = the percentage of transversal steel
σ_N = compressive stress due to external compressive force acting (into the level of the panels to be connected), perpendicularily to the joint

3.- REPETITIVE DYNAMIC LOADING

A loading history $0 - \tau_{max} - 0$ has been dynamically applied ($T \simeq 0,8$ sec) in a series of tests, under several conditions of shear level and several hundreds of cycles. Fig. 2 is a schematic presentation of the related f a t i g u e strength data.

4.- FULLY REVERSED ACTIONS

Several fully reversed shear-displacements $\pm s$ have been applied in a series of joints [5].

A simple physical model of the corresponding behaviour is represented in Fig. 3, as used in actual development of a computerised model. Fig. 4 shows a typical hysteretic behaviour of the connection "ΔE_8" tested; on the basis of all test results of this programme, the approximate behavioural models shown in Fig. 5 have been prepared for possible use in computer analysis of composite walls. The degradation of shear-force response of connections submitted to full shear-displacement reversals is shown in Fig. 6.

Inspite the characteristic shear pinching effect, apparent in Fig. 5 , a considerable amount of relatively stable hysteretic damping (Fig. 7) may be secured.

Another important feature of these connections is their remarkably extended plastic behaviour a f t e r l a r g e c y c l i c a c t i o n s: Fig. 8 shows average stress-displacements curves, after severe cyclic actions. It becomes apparent that compared to the monotonic tests lower peak values are encountered (Fig. 9). In addition four to five times larger shear displacements are observed at these peak values, after the cyclic actions imposed. Finally a considerable amount of residual strength is available for very large displacements (Fig. 10).

5.- SEISMIC DESIGN

For a <u>rational</u> design under seismic loading, the following data have to be estimated as an input (see Fig. 11):

- Expected maximum normalised displacement ("imposed ductility" $\delta = 3 \div 4$).
- Expected maximum number of effective full reversals ($n \approx 2 \div 5$).

Under these conditions, the following mechanical characteristics should be found in order to carry out a Seismic design:

- Shear force response available ($\tau_{resp}^{n,\delta}$) [Fig. 6]
- Last shear stiffness $K_u = \tau_{resp}^{n,\delta} : s_{max}$
- Last hysteretic damping "ζ"
- Ultimate (static) bearing capacity after cyclic straining ($\tau'_{u,max}$)
- Residual strength ($\tau'_{R,min}$) under very large monotonic deformations preceded by cyclic straining.

Some diagrammes shown in Fig. 9 and Fig. 10, contain data regarding the two last characteristics.

On the basis of this philosophy, design values have been proposed for the connections of large precast panels under seismic conditions.

These values have taken into account the increasing force response degradation when a) larger shear displacements are imposed and b) larger number of full reversals are applied.

Since it is practically impossible to "calculate" these two characteristics, the corresponding degraded values of force-response ("resistance") have been related (Tsoukantas, 1981) to the conventional intensity of seismic actions expected, i.e. to the base shear coefficients applicable to each particular case.

The authors have proposed a degradation factor:

$$\lambda_{\gamma} = 1 - (10 \div 15)\, C^{2} \nless 0.3$$

where
"C" denotes the relevant base shear coefficient (however for european seismic conditions).

Therefore the corresponding design formulae expressing the strength of vertical and horizontal joints may be factored accordingly.

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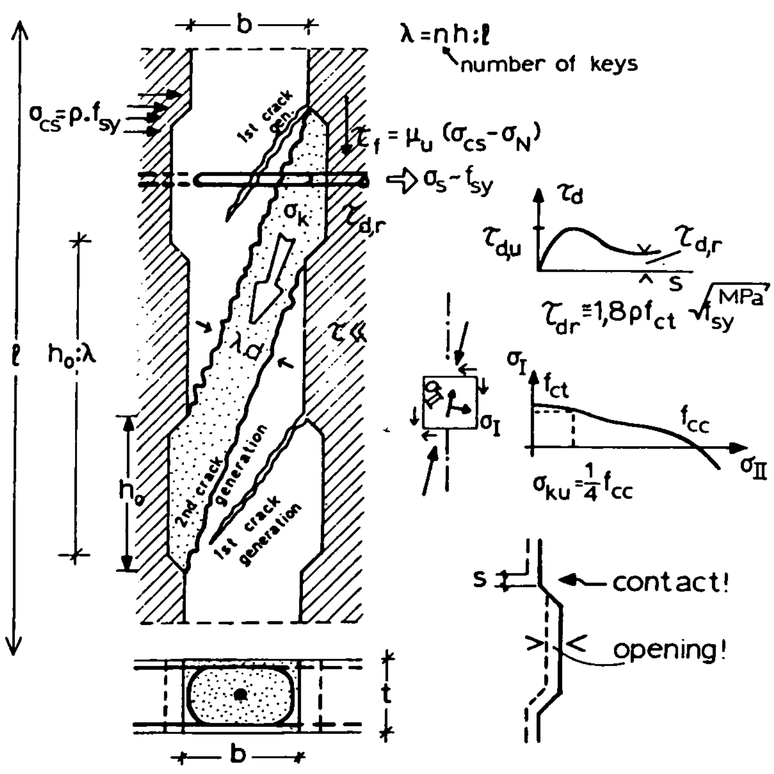


Fig. 1: A simplified model for shear transfer within L.P. Panels' connections

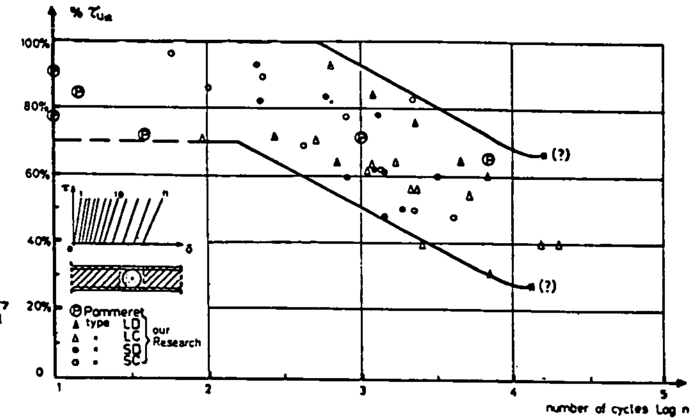


Fig. 2: Shear strength degradation under REPETITIVE (not reversed) loading

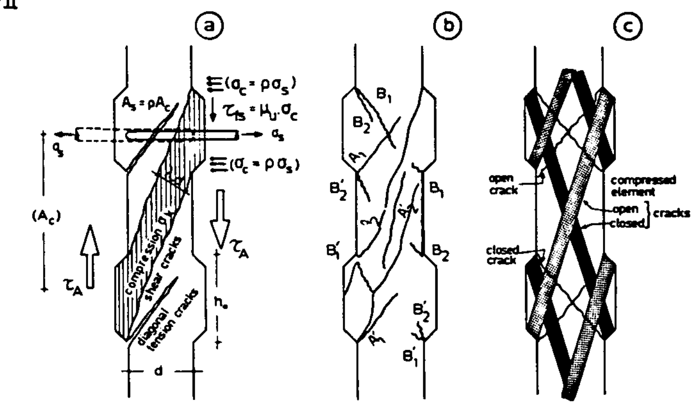


Fig. 3: Physical model for joints under fully reversed actions

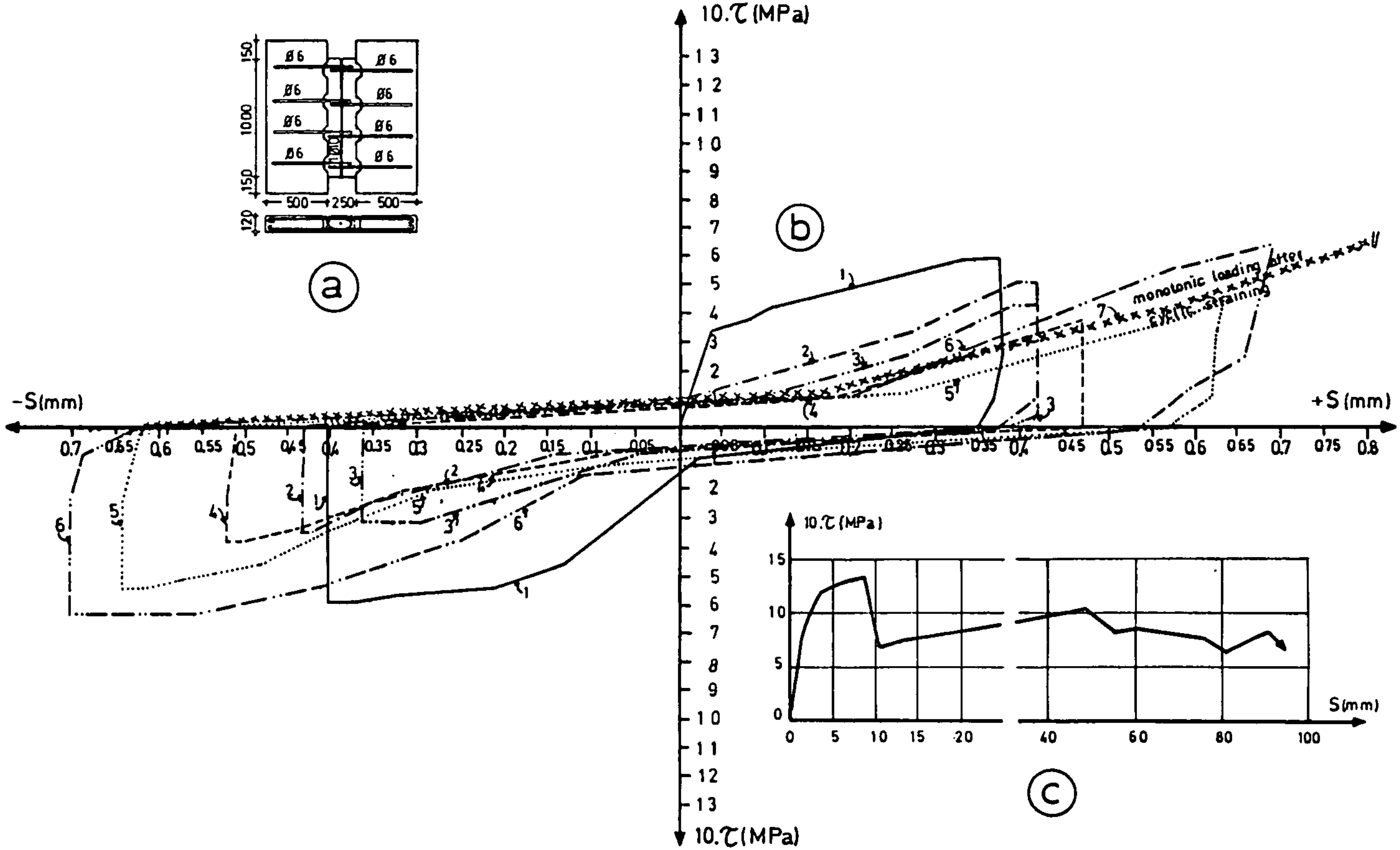


Fig. 4: Large panels' connection under cyclic shear deformation
a) Connection tested
b) Shear force vs. shear displacements
c) Monotonic loading after cyclic straining (τ vs s)

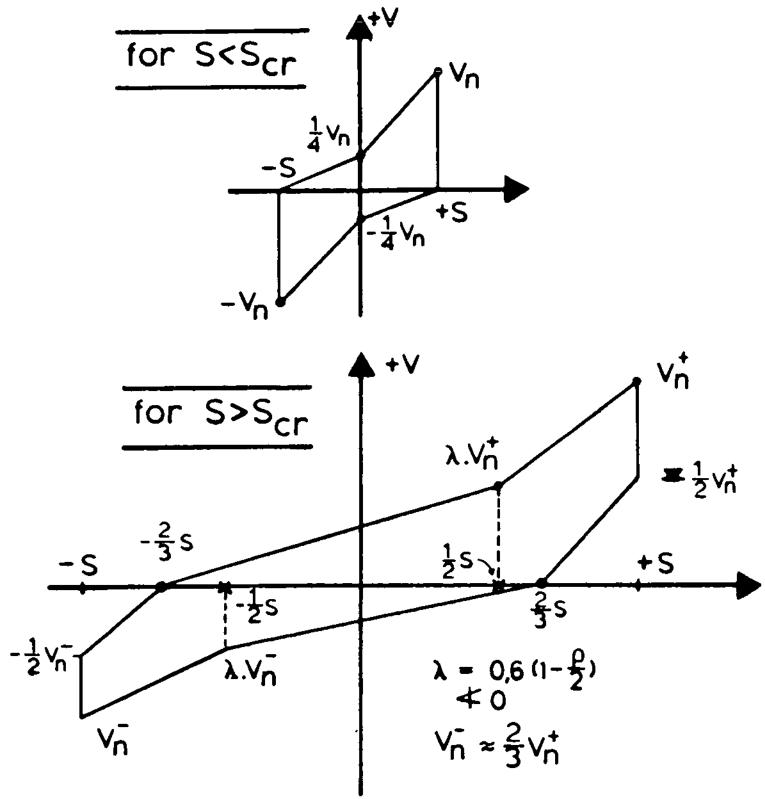


Fig. 5: Formalistic constitutive laws for shear-force vs. shear displacement of large panels' connections (ρ = percentage of transversal reinforcement, V_n = degraded shear-force response, s.Fig. 6)

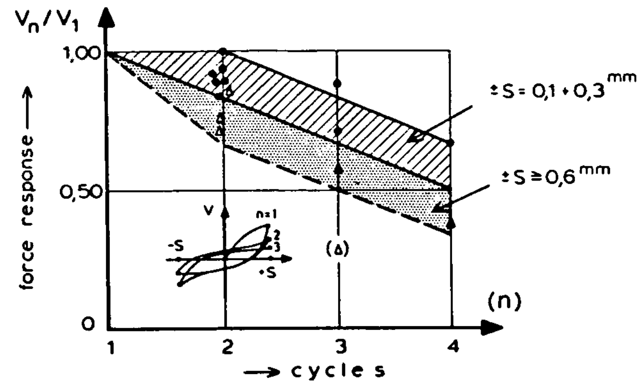


Fig. 6: Shear force degradation during fully reversed cyclic deformations ±S for a high percentage of transversal reinforcement $\rho \approx 1,0\%$

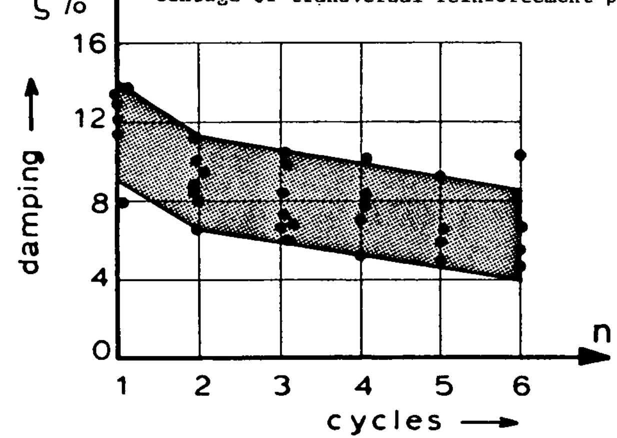


Fig. 7: Hysteretic damping during reversed cyclic deformations $|\pm S| = 0,1 \div 1,1$ mm (when $\rho = 0,2 \div 1,0\%$)

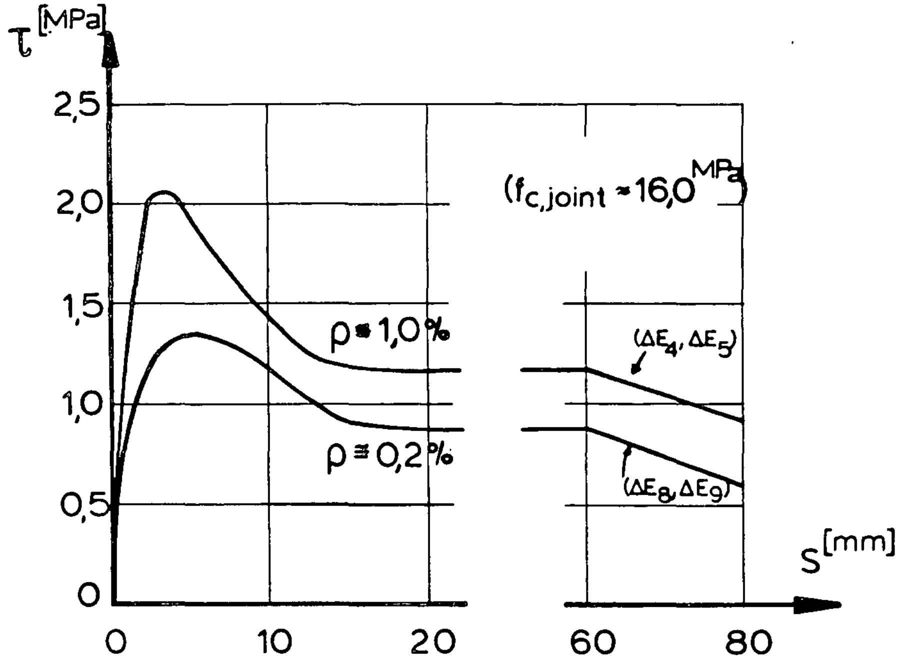


Fig. 8: Post seismic behaviour (smoothened curves) of large panel connections (after approx. 7 full reversals)

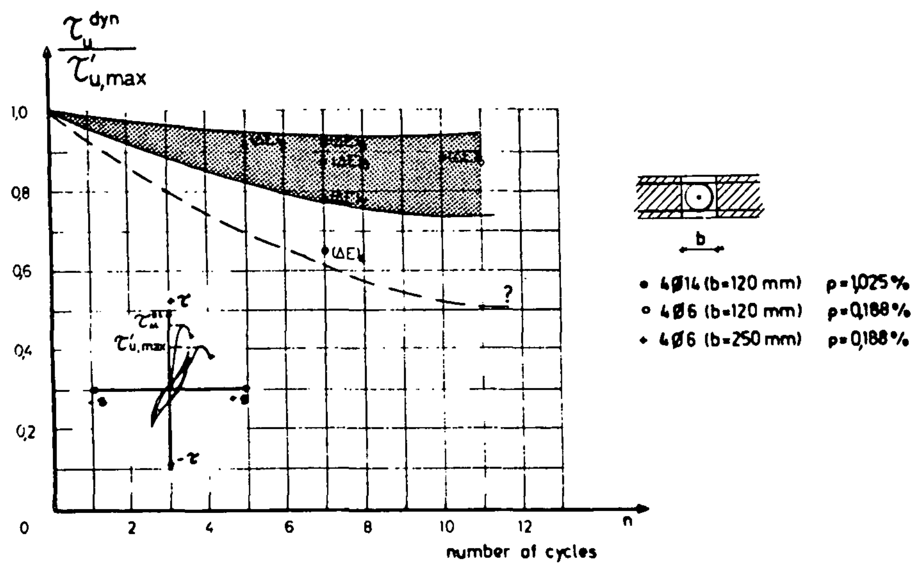


Fig. 9: Ultimate (static bearing capacity of precast panels' joints, after fully reversed actions

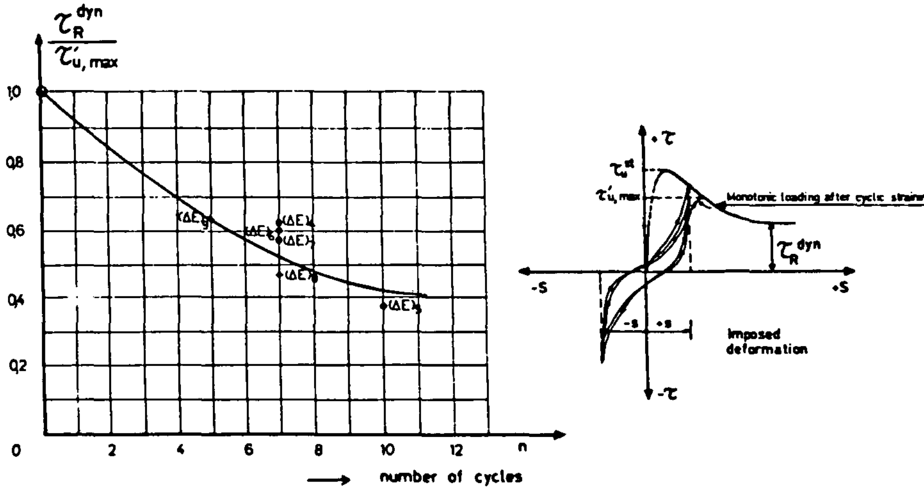


Fig. 10: Residual static strength (for very large deformations) after the cyclic straining)

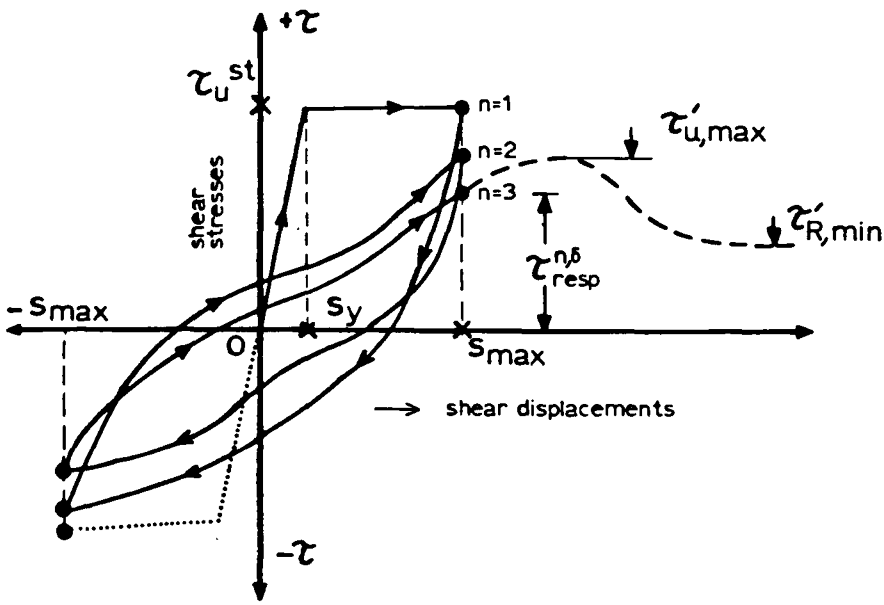


Fig. 11: For a given input ("δ"= $s_{max} : s_y$, and "n") several mechanical characteristics may be found ($\tau^{n,\delta}_{resp}$, $K_u = \tau^{n,\delta}_{resp} : s_{max}$, $\tau'_{u,max}$ etc)

A N N E X I I

GUIDE TEXT(*) FOR THE PROPORTIONING OF THE HORIZONTAL AND VERTICAL JOINTS OF § 4.4.

1.- Maximum permissible number of stories depending on spans and the seismicity of the area (see Table 1)

Table 1

Seismicity		I		II		III	
Spans in (m)		<5.5	≥5.5	<5.5	≥5.5	<5.5	≥5.5
Qualification of the contractor(**)	A-level	10	7	5	3	3	2
	B-level	3	2	2	1	1	0

2.- Minimum admissible effective thickness (Fig. 1) of panels (see Table 2)

TABLE 2

TYPE OF ELEMENT		Qualification of the contractor(**)	t_{min} (mm)
Panels	Internal walls	A	100
		B	120
	External walls	A	120
		B	140

3.- DESIGN OF CONNECTIONS

3.1. Actions to be considered

The actions S_u will be calculated according to the general formula (1)

$$S_u = \gamma_{fg}G + \gamma_{fp}F_p + \gamma_{fq}(Q_{1k} + \sum_{i>1} \psi_o Q_{2k}) \qquad (1)$$

where:
S_u =the design action under consideration
γ_{fg}, γ_{fp}, γ_{fq} = are the partial safety coefficients given in Table 3
G = denotes the influence of permanent actions
F_p = denotes the influence of eventuel prestressing force
Q_{1k}= denotes the basic variable action
Q_{2k}= denotes the remaining variable actions
ψ_o= is the combination's coefficient for variable actions Q_{2k}, given in Table 4

(*) This guide text is based on a proposal made by the NTU of Athens for a Draft Greek Code concerning precast R.C. Structures (design of vertical and horizontal joints in R.C. precast large panel buildings).

(**) A contractor may be classified on category "A" if he can prove that he has sufficientexperience on the same type of structures and can also prove that he possesses equipment of high quality and has under his orders the suitable number of qualified personnel. High level of inspection is also requested.

NOTE: Each variable action (seismic action(*) is included) should be taken succesively as the basic action and the most favourable result of the sum $Q_{1k} + \sum_{i>1} \psi_o Q_{2k}$, will be taken into account in Equ. (1).

TABLE 3

factor	permanent γ_{fg}	prestress γ_{fp}	variable γ_{fq}
favourable effect (for the load-action to be checked)	1.0	0.9	not to be taken into account
unfavourable effect (for the load-action to be checked)	1.35	1.2	1.5

TABLE 4

factor		ψ_o
Service Loadings	Dwellings	0.4
	Offices and retail stores	0.6
	Parking areas	0.6
Environmental (wind - snow)	General	0.5

3.2. Shear connections

Such connections are primarily resisting shear forces (V). Sometimes, shear forces are accompanied by normal forces "N" (compression or tension).

3.2.1. Notations (see also Fig. 1)

l_j = joint's length
b = joint's width
t_j = effective thickness of joint
t_k = effective thickness of key
t = panel's thickness
h_o = key's length (inside the panel)
h_1 = key's length (inside the joint area)
a = key's depth
α^o = slope of key's edge
n = number of keys
λ = the density of keys

$\lambda = \frac{nh_o}{l_j}$ for open joints (see Fig. 1.a)

$\lambda = \frac{nh_o t_k}{l_j t_j}$ for closed joints (see Fig. 1.b)

3.2.2. Design of keyed shear connections

a) Geometrical requirements for keys (see Fig. 1)

$\alpha^o < 30^o$
$a > 20$ mm

(*) The seismic action will be taken into account 70% of its nominal value

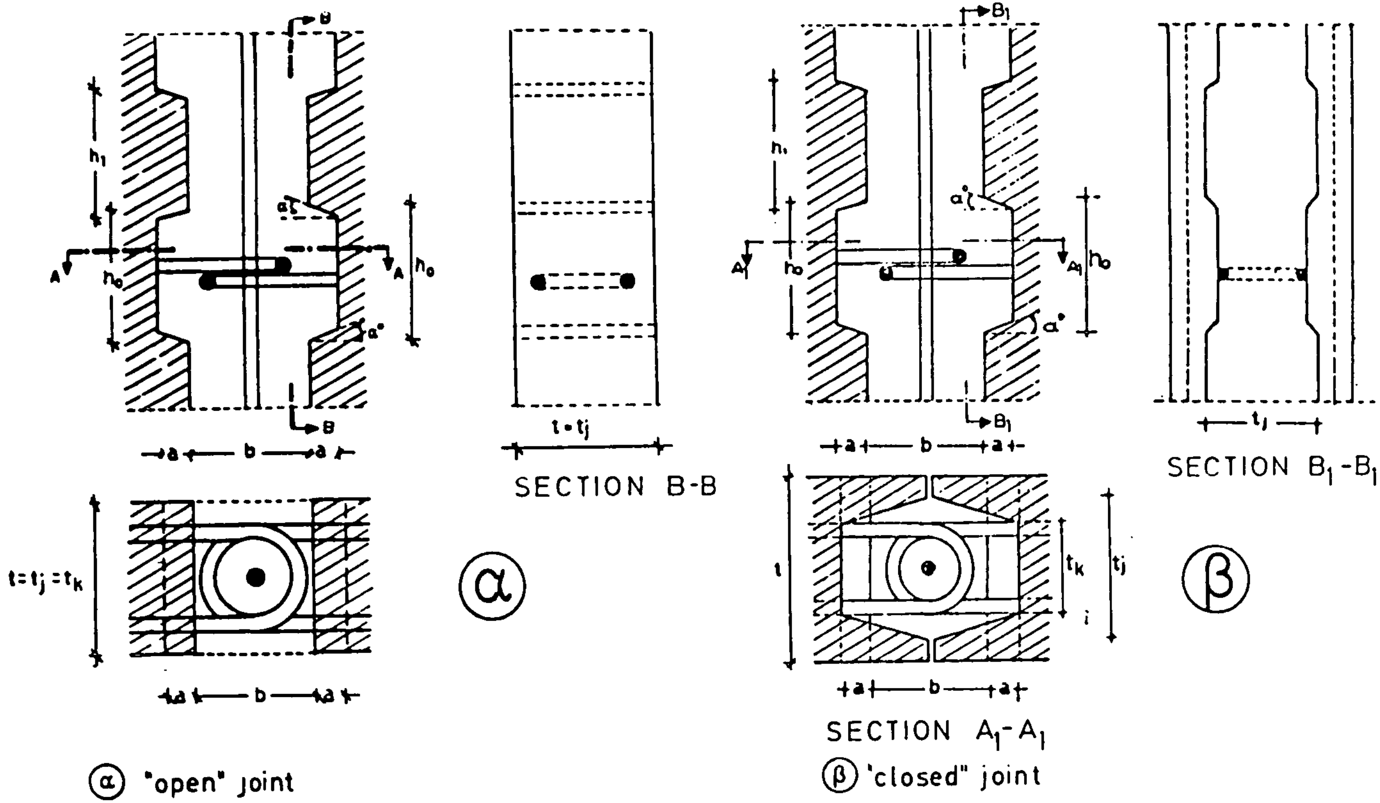


Fig. 1: Typical vertical joint

$\frac{h_o}{a} < 8$

$\lambda = 0.2 \div 0.5$

$n > 3$

$h_o < 500$ mm

b) Design equation

$\tau_{act,u} < \tau_{R,u}$

where $\tau_{act,u}$ represents the acting design shear stress taken according to Equ (3) and $\tau_{R,u}$ is the ultimate design shear resistance stress of the joint taken according to Formula (4).

$$- \tau_{act,u} = \frac{V_u}{l_j t_j} \qquad (3)$$

where,

V_u = the design shear force according to Equ.(1)

$$- \tau_{R,u} = \frac{1}{\gamma_c} \gamma_{nc} \; 0.1 \; f_{ck} \; \lambda + \frac{1}{\gamma_s} \gamma_{ns} \left(\frac{A_s f_{sy} - N}{l_j t_j} \right) \qquad (4)$$

where,

$\tau_{R,u}$ = ultimate design shear resistance of the joint

λ = the density of keys (see also § 3.2.1)

f_{ck} = characteristic compressive strength of the concrete of the joint

A_s, f_{sy} = the total area and the yield stress of the connecting bars respectively

N = external force acting into the plane of the panels to be connected perpendicularely to the main axis of the joint. In equ (4) tensile forces will be taken into account as positive while compression forces will not be taken into account

l_j, t_j = the length and the effective joint's thickness respectively

γ_c, γ_s = partial safety factorsfor the material used, concrete and steel, taken 1.5 and 1.15 respectively

γ_{nc}, γ_{ns} = correction factors against the uncertainties of the design model (for concrete and steel respectively), depending also on the seismicity of the area. They will be taken according to Table 5.

TALBE 5

factors / seismic base shear coefficient	γ_{nc}	γ_{ns}
no earthquake	0.7	0.7
C = 0.04	0.65	0.65
0.06	0.55	0.55
0.08	0.45	0.45
0.10	0.35	0.35
0.12	0.10	0.30

3.2.3. Minimum joint reinforcement

a) Transversal reinforcement

- for internal wall joints $A_{s_{min}} = 1.7$ cm^2

- for external wall joints $A_{s_{min}} = 3.0\ cm^2$

b) Longitudinal reinforcement $A_{s_{min}} = 2.0\ cm^2$

3.3. Connections under prevailing compression

Such connections are primarily resisting to compression forces N, accompanied normally by bending moments as well as by shear forces V acting in the direction of the long axis of the connection.

3.3.1. Notations (see Fig. 2)

l_j = total length of the horizontal joint
l_c = length of the joint (of total length l_j) under compression
l_t = length of the joint (of total length l_j) under tension
t_j = width of the joint
h_j = thickness of the joint (height)

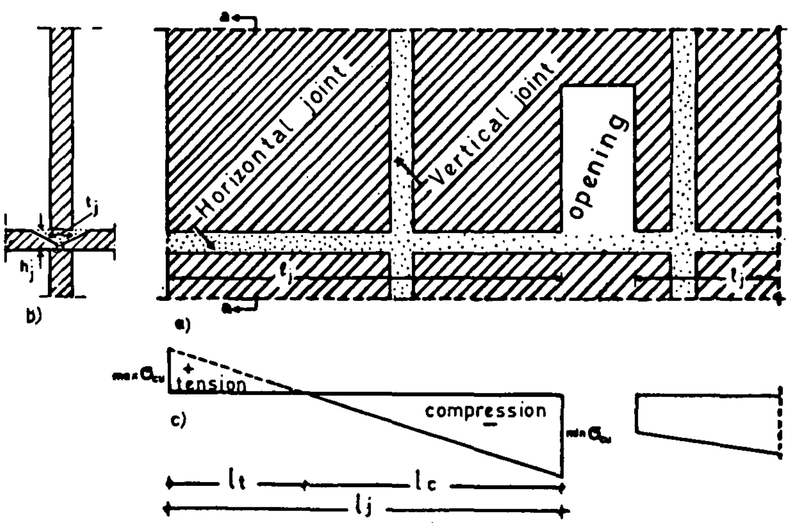


Fig. 2: Sketch in which geometrical or reinforcement details are not shown

3.3.2. Actions to be considered

- The normal force min N_u, the shear force V_u and the bending moment M_u (Vector acting perpendicularly to the plane of the panels to be connected) will be calculated according to Equ (1).

- The normal tensile stress maxσ_{cu} as well as the length of the tension-zone and the length of the compression-zone will be estimated according to formula (5)

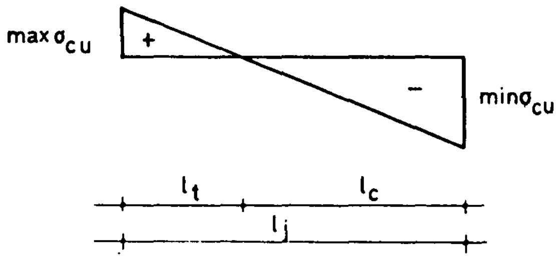
$$\begin{matrix}\max\\\min\end{matrix}\sigma = \frac{\max N}{A_j} \pm \frac{M_u}{W_j} \qquad (5)$$

In the above formula (5) are:
$A_j = l_j t_j$, the area of the horizontal joint

$W_j = \frac{t_j l_j^2}{6}$ the moment of resistance

max N will be taken according to Equ (11)

M_u will be taken according to Equ (1).

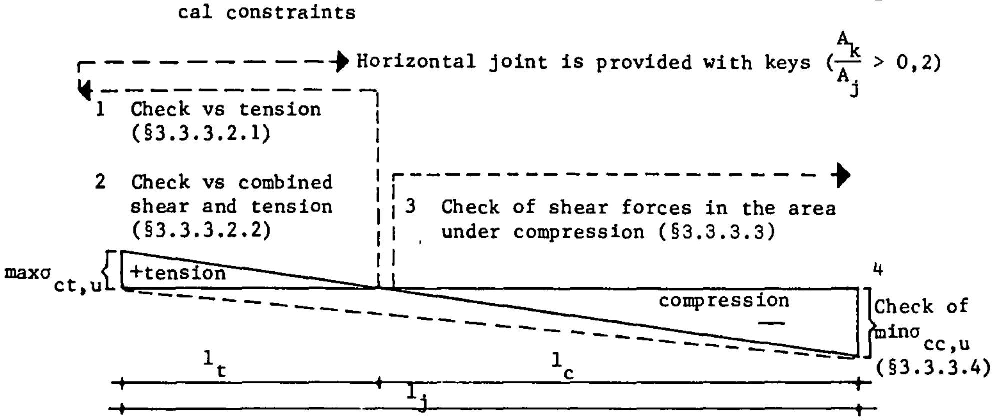


- The shear stress will be calculated according to the formula

$$\tau_{act,u} = \frac{V_u}{l_j \cdot t_j} \tag{6}$$

3.3.3. Strength requirements

3.3.3.1. A shematical presentation of necessary verifications and geometrical constraints



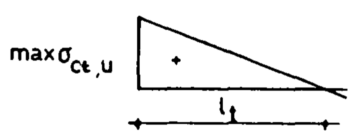
3.3.3.2. Verifications along the length of the tension zone of the horizontal joint

3.3.3.2.1. Check versus tension

- The values of maxσ_{cu} and l_t will be calculated according to Equ (5).

- The necessary area of reinforcement A_s will be calculated according to the following formula (7)

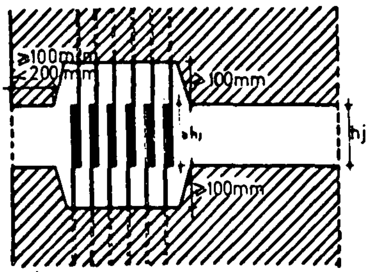
$$A_s = \frac{Z}{f_{sy}} \quad (7)$$



where,

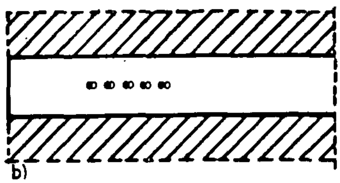
$Z = \frac{1}{2} t_j \cdot l_t \cdot \max\sigma_{ct,u}$ (tensile force)

- The tensile reinforcement will be arranged according to Fig. 5.

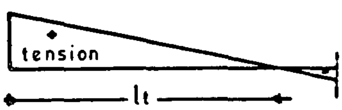


a)

Fig. 5: Horizontal joint with a single "shear-tension key"
a) Vertical section
b) Horizontal section through the body of the key
c) Diagram of stresses according to Equ (5)


b)

• Sketch in which other reinforcements or details are not shown



c)

3.3.3.2.2. Check versus shear in the tension area

- Design Equation

$$\tau_{act,u} < \tau_{R,u} \tag{8}$$

$\tau_{act,u}$ will be calculated according to Equ (6)

$$- \tau_{R,t} = \gamma_{n,\varepsilon} \frac{\Delta A_s f_{sy}}{l_t t_j} \tag{9}$$

where,

ΔA_s = section of the additional reinforcement (in addition to A_s required against tension)

$\gamma_{n,\varepsilon}$= correction factor depending on the seismicity of the area, taken according to the Table 6

TABLE 6

Seismic base shear coefficient		$\gamma_{n,\varepsilon}$
c	0.04	0.55
	0.06	0.45
	0.08	0.35
	0.10	0.30
	0.12	0.25

- This additional reinforcement will be arranged along the area under tension of the joint; it may also be placed as it is shown in Fig. 5.

3.3.3.3. Check versus shear in the compressive area:

a) The following stresses have to be calculated:

$\tau_{act,u} = \frac{V_u}{l_j t_j}$ according to Equ (6)

and

$$\sigma_{cc,G} = \frac{\max N_u}{l_c t_j} \tag{10}$$

where max N_u denotes the possible lowest value of axial compressive force according to the following formula

$$\max N_u = 1.0\, N_G - 1.5(3\,c)\, N_G \tag{11}$$

where

$1.5(3\,c)\,N_G$ expresses the possible negative action of the vertical component of the seismic force

b) Checks, (Fig. 6)

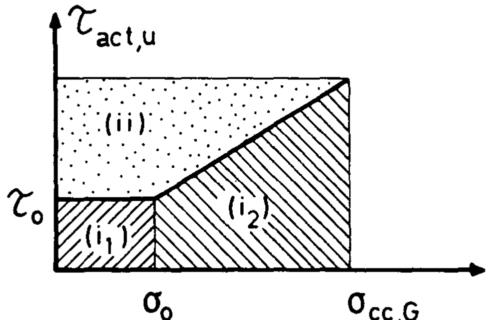


Fig. 6

i) Reinforcement is not needed when:

(i_1) $|\sigma_{cc,G}| < |\sigma_o|$ (12)

and concurrently

$\tau_{act,u} < \tau_o$ (13)

or

(i_2) $|\sigma_{cc,G}| > |\sigma_o|$ (14)

concurrently

$\tau_{act,u} < 0.7\, |\sigma_{cc,G}|$ (15)

The values of σ_o and τ_o shall be taken from Table 7

TABLE 7

Qualification of contractor	σ_o [MPa]	τ_o [MPa]
A - level	0.60	0.40
B - level	0.45	0.30

ii) If the above conditions are not satisfied, transversal reinforcement will be provided in order to satisfy the following inequality:

$$\tau_{act,u} < \tau_{R,u} = \frac{1}{1.5}\gamma_{nc,\mu}\ 0.7\sigma_{cc,G} + \frac{1}{1.15}\gamma_{ns}\frac{A_s f_{sy}}{l_c l_j} \tag{16}$$

where,

A_s the area of the transversal reinforcement

$\gamma_{nc,\mu}$ correction factor related to the frictions coefficient and depending on the seismicity of the area (see Table 8)

γ_{ns} correction factor related to the degrading participation of the reinforcement in the shear resistance of the joint, depending on the seismicity of the area (see Table 8)

TABLE 8

factor / Seismic base shear coefficient	$\gamma_{nc,\mu}$	γ_{ns}
0.04	0.6	1.0
0.06	0.6	0.8
0.08	0.6	0.8
0.10	0.6	0.7
>0.10	0.5	0.6

3.3.3.4. Check of the min$\sigma_{cc,u}$

- The min$\sigma_{cc,u}$ will be calculated according to the formula

$\min\sigma_{cc,u} = \frac{N_u}{A_j} - \frac{M_u}{W_j}$ where N_u and M_u will be calculated according to the general Equ (1)

- It must be

$$\min\sigma_{cc,u} > f_{cc,u}$$

where,

$f_{cc,u}$ = the limit value for the compressive strength of the concrete of the joint; it may be estimated according to Equ (18) and Equ (19) for connections as it is shown in Fig. 7 and Fig. 8 respectively.

$$f_{cc,u} = \left(1 - \frac{\frac{2e}{t}}{1-\frac{b}{t}}\right)\left[\frac{E_j}{E_{sl}}\left(1-\frac{2b}{t}\right) + \frac{2b}{t}\right]\left(1 - \frac{\tau_{act,u}}{\tau'_{Ru}}\right).f_{ck} \tag{18}$$

$$f_{cc,u} = (0.85 \div 0.95)\left(1 - \frac{\tau_{act,u}}{\tau'_{Ru}}\right)f_{ck} \tag{19}$$

where,

$\tau_{act,u}$ as in Equ (6)

τ'_{Ru} = ultimate shear resistance of the concrete of the joint

E_j, E_{sl} = Moduli of elasticity of the concrete of the joint and the slab respectively

e = the eccentricity of the vertical load due to transversal moments

b = the supported length of the slabs

t = the thickness of the walls to be connected

f_{ck} = the characteristic strength of the concrete of the joint

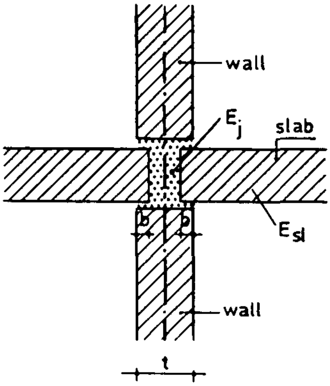


Fig. 7 (Reinforcements and other details are not shown)

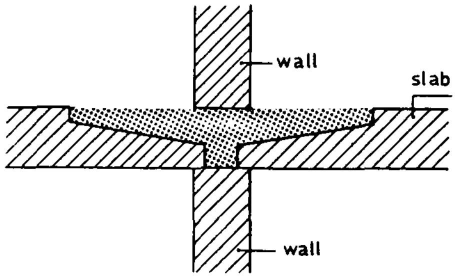


Fig. 8 (Reinforcement and other details are not shown)

3.3.4. Minimum reinforcement and keys (see Fig. 9)

- keys: two, one in each edge of the horizontal joint
- reinforcement: 2∅12 in both the lower and the upper panel, welded inside the key area.

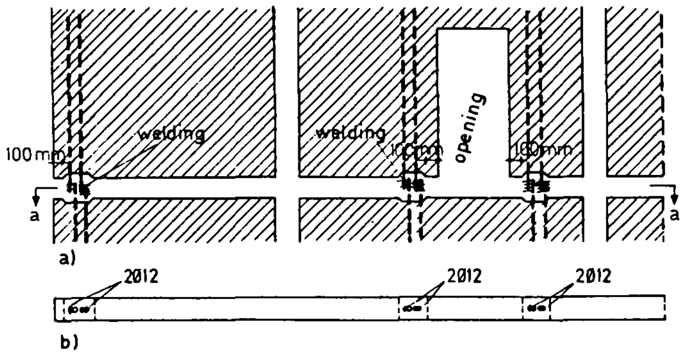


Fig. 9: Sketch showing the minimum requirements for keys and reinforcement in the horizontal joint (Other geometrical or reinforcement details are not shown)

5. REPRESENTATIVE EXAMPLE BUILDING HUNGARY

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INTRODUCTION

The representative (significant) prefabricated building system in Hungary is the large panel system building.Present and inherent housing needs are basically coverd at present time with products provided by 12 Large Panel Factories.

System buildings in general, large panel buildings in particular, are to be designed according to the constitutive lows of the General System Theory (Bertalanffy) following a hierarchical differentiation of system components, organized in an modular order.

The interaction of precast panel components is achived by multifunctional (e.q. Load transfer, heat-sound insolation) jointing system. Horizontal and vertical joint types are standardised as far as geometric and shape parameters are concerned in a keyed shear joint type, differentiated following a partitioning in strength and ductility with the help of amount (quantity) and quality of tying reinforcements.

5.1. SCOPE

The objective of this Part is to show the application of National Seismic Code Regulations correlated with a generalized design philosophy to a mass produced, standardised large panel building system, representative in the hungarian building industry.

5.2. DESCRIPTION OF THE STRUCTURAL SYSTEM

Industrialized building technique adapted in Hungary is developed following an ordered modular coordination with a high degree of flexibility and stability as far as types of lay-outs and storey high is concerned large panel systems are designed and prefabricated in the Country in a modulus of 60 cm, resulting in axial distances between walls of: 5,40 m; 4,80 m; 4,20 m and 3,60 m in both main directions.

Thickness of structural wall components (cross and longitudinal walls) and that of floor slabs is 15 cm.

Independent of role and size, up to 18 stones in height, the uniform quality requirement is as follows:
- Concrete: B200 (cube strength 20 MPa)
- Steel in members: C.15.H (yield strength 420 MPa)
- Steel in joints: B.38.24 (yield strength 240 MPa)
- Non structural steel in face panels: ANTICOR.

The structural system under investigation is a mixed, two way (cross and longitudinal) slab wall system.

The general floor lay-out of sample, one staircase section - representing a dilatation unit - with three flats is shown in Figure 5.1. Periferial and/or face walls are layered (sandwich) panels with or without openings, having a polyurethan thermal insolation. Internal wall members and the structural part of external walls (except lintels and columns) are provided with the minimum amount of reinforcement (edge reinforcement) sufficient and enough to resist temporary technological loading (e.g. lifting, impact, transportation etc.).

Internal particions in concrete, L.W. concrete or gypsum are non structural (non load bearing) components.

To demonstrate the influence of the tollness upon the seismic response, the calculation is extended over two different building heights (Figure 5.2. and 5.3.) namely:
- an 11 storey high (H = 33,83 m), and
- a 5 storey high (H = 17,20 m)

residential building, having the same cross sectional characteristics and storey height of 2,80 m.

Both investigated buildings lying on cast-in-place r.c. plate foundation are supposed to be elastically supported.

5.3. STRUCTURAL ANALYSIS

5.3.1. Design Loads

Loading in general, earthquake loading in particular is a model simulation of a random environmental event. A quantitative approach in assessing

loading intensity (severity) levels, which may occur during the designed life span of a building can be determined on a probability basis (see H.C.* Part I). Accordingly the severity of seismic loading is classified in three main cathegories ($k = 0,1,2$). These are:
- serviceability level (microtremor: $k = 0$),
- limit level (moderate earthquake: $k = 1$),
- accidental level (strong earthquake: $k = 2$).

The presence of earthquake loading in design procedure requires a modified combination of loadings [6] which is carried out according to the following format:
- serviceability level ($k = 0$)

$$X_o = 1,1\ X_D + 0,6\ X_{LP} + 0,3\ X_{LS} + 0,0\ X_{LW} + 1,0\ X_E$$

- limit level ($k = 1$)

$$X_1 = 1,1\ X_D + 0,6\ X_{LP} + 0,3\ X_{LS} + 0,0\ X_{LW} + 1,4\ X_E$$

- accidental level ($k = 2$)

$$X_2 = 1,1\ X_D + 0,6\ X_{LP} + 0,3\ X_{LS} + 0,0\ X_{LW} + 1,8\ X_E$$

where

X_D is the action (force, moment) caused by dead loads,

X_{LP} is the action generated by long term live loads,

X_{LS} is the action generated by snow loading,

X_{LW} is the action generated by wind loading,

X_E is the action generated by earthquake.

Since the analitical calculation is performed with the help of a uniform structural model, gravity loading and masses are considered to be uniformly distributed over the full height of the representative building. Hence, the specific design value of the gravity loading, according to the combination low of loadings (H.C. Part I.) is as follows:
- dead load

$$q = 1,1\ \frac{G}{H} = 1,1\ \frac{28640}{33,83} = 931\ \text{kN/m}$$

- live load

$$p = 0,6\ \frac{P}{H} = 0,6\ \frac{7800}{33,83} = 138\ \text{kN/m}$$

Total gravity loading: $q + p =$ 1069 kN/m

Note: Snow load is neglected here. The mass per unit height is

$$m = \frac{1069}{9,81} = 108,9\ \text{kN s}^2\ \text{m}^{-2} \approx 10,89\ \text{Mp s}^2\ \text{m}^{-2}$$

*H.C. = Hungarian Contribution to the Design Philosophy (1982)

5.3.2. Material Characteristics

Strength values to be considered in different limit states (k, i=0,1,2) are given in Table 5.1. [2].

Table 5.1.

Stresses (MPa)	Concrete	Steel		Soil
	B200	C.15.H	B.38.24	Clay
	Compression	Tension		Compression
Admissible (i=0)	7	280	160	0,15
Design (i=1)	10	350	210	0,25
Characteristic (i=2)	17	420	240	0,38

Material stiffness values (Joung's moduli, rocking subgrade) according to different limit states are representing the effect of material degradation due to seismic effects (cycling loading) taken as shown in Table 5.2.[9].

Table 5.2.

Stiffness Values (MPa, MPa/m)	Joung's Moduli		Subgrade Moduli
	Concrete	Steel	Soil
	E_c	E_s	C_ϕ
Serviceability (i=0)	23000	21000	$150 \frac{\sigma_s}{B_B}$
Limit (i=1)	0,8×23000		
Ultimate (i=2)	0,6×23000		

Notations:

σ_s = is the strength value of the soil (MPa) given in Table A.1. (i=0,1,2)

B_B = is the length of the basement in the direction of rocking.

5.3.3. Structural Characteristics

The dynamic modelling of the structural system so also performed in three different structural limit states, as seen in Table 5.3.[9].

Table 5.3.

Moment of Inertia (m^4)	Action		
	Pure Bending	Bending+ Axial Load	Bending+ Shear
Serviceability (i=0)	$0,8\ I_o$	$1,0\ I_o$	$0,6\ I_o$
Limit (i=1)	$0,6\ I_o$	$0,8\ I_o$	$0,4\ I_o$
Ultimate (i=2)	$0,4\ I_o$	$0,6\ I_o$	$0,2\ I_o$

Note: - I_o is the second moment of area calculated with gross (uncracked) values.
- Connecting beams are assumed to be in combined bending and shear.

5.3.4. Cross-sectional Data (Figure 5.4)

Cross sectional area of ocupled sehar walls;

$$A = 15,74\ m^2$$

Cross sectional area of the foundation:

$$A_B = 12,6 \cdot 22,5 = 283,5\ m^2$$

Coordinates of
- the elastic center;

$$x_E = 10,40\ m; \qquad y_E = 6,61\ m$$

- the center of masses;

$$x_M = 10,36\ m; \qquad y_M = 5,93\ m$$

The inclination of the principal axes (Figure 5.4: $\overline{X}$; $\overline{Y}$)

$$\alpha = 0$$

(directions; vertical and horizontal).
Mass excentricities;

$$e_x = x_T - x_M = -0,06\ m; \qquad e_y = y_T - y_M = 1,01\ m$$

Equivalent second moment of area of the family of coupled shear walls calculated with gross (uncracked) values are collected in Table 5.4.
The equivalency criterion

$$\frac{M - \Sigma NL}{\Sigma I_i} = \frac{M}{(\Sigma I_i)_e}$$

provides

$$(\Sigma_i I_i)_e = \frac{1}{1 - \frac{\Sigma NL}{M}} \Sigma_i I_i$$

with ΣNL = moments absorbed by lintel rows.

Table 5.4.

i	I_{yi}	I_{xj}	j
VI.	11,0	23,3	1
V.	23,3	31,5	2
IV.	13,4	23,0	3
III.	63,6	4,9	4
II.	5,3	5,4	5
I.	33,0	4,8	6
	-	4,0	7
	-	24,0	8
I_x =	149,6 m^4;	120,9 m^4	= I_y

Warping (sectorial) moment of inertia is derived as seen in Table 5.5.
Note: Pure torsion neglected for open sections.

Table 5.5.

Wall i,j	I (m^4)	a (m)	a^2 (m^2)	I_w (m^6)
1	23,3	10,42	108,580	2529,830
2	31,5	5,02	25,200	793,812
3	23,0	0,38	0,144	3,321
4	4,9	3,08	9,486	46,483
5	5,4	3,98	15,840	85,530
6	4,8	6,68	44,622	214,187
7	4,0	8,48	71,910	287,641
8	24,0	12,08	145,926	3502,233
VI	11,0	6,61	43,092	480,613
V	23,3	4,81	23,136	593,071
IV	13,4	1,21	1,464	19,619
III	63,6	0,59	0,348	22,139
II	5,3	4,19	17,556	93,047
I	33,0	5,99	35,884	1184,043
	270,5 = $I_x + I_y$;			I_w = 9855,57

Polar radius of inertia (square)

$$i_p^2 = \frac{I_x + I_y}{A} + e_x^2 + e_y^2 = \frac{270,5}{15,74} + 0,06^2 + 1,01^2 = 18,2092 \text{ m}^2$$

5.3.5. Dynamic Characteristics

Natural frequencies (free periods) of vibration are examined with the mass $m = 10{,}89$ Mp s^2 m^{-2} in elastic range and serviceability level. Formulae to be used are given in [12]. Chapter 3.3. Fundamental (first mode) characteristics are shown in Table 5.6.

Table 5.6.

Building Type	Circular Frequencies (s^{-1})			Periods (s)			Support Condition
	ω_{y1}	ω_{x1}	$\omega_{\phi 1}$	T_{y1}	T_{x1}	$T_{\phi 1}$	
11 Stories	15,67	17,43	33,06	0,400	0,360	0,190	Clamped
	3,55	3,56	-	1,765	1,334	-	On elastic support
5 Stories	60,62	67,43	127,92	0,103	0,093	0,049	Clamped
	9,95	13,20	-	0,631	0,475	-	On elastic support

Remarks:
- the influence of the elastic support is considerable, as such provides a tool for regulation (calibration) of dynamic characteristics [4], [7], [8].
- low rise large panel buildings are relatively rigid structures, the ratio in periods from 11 to 5 stories is about 1 to 3.
- the range of fundamental periods is shown in Figure 5.5.

The analysis of coupled vibrations by using the equation ($e_x = 0$)

$$\left(1 - \frac{e_y^2}{i_p^2}\right) \omega^4 - (\omega_y^2 + \omega_\phi^2)\, \omega^2 + \omega_y^2\, \omega_\phi^2 = 0$$

proved, that

$$\omega = 15{,}54 \ s^{-1} \approx \omega_y = 15{,}67 \ s^{-1}$$

consequently, torsional vibrations do not affect the magnitude of bending vibrations. In the light of this structural properties further analysis in extended only over bending vibrations in two principal directions x and y. Higher modes than the fundamental are computed with the help of the relation

$$\omega_n = \left(\frac{2n-1}{3{,}75}\right)^2 \omega_1$$

or more detailed

$$\omega_2 = \left(\frac{4{,}694}{1{,}874}\right)^2 \omega_1 = 6{,}274\ \omega_1 = 97{,}499\ s^{-1}$$

$$\omega_3 = \left(\frac{7{,}855}{1{,}874}\right)^2 \omega_1 = 17{,}569\ \omega_1 = 273{,}022\ s^{-1}$$

Dynamic factors applied in analysis

According to the inequality (Design Guidlines, Chapt.4.3.) [12]

$$0{,}6 \leq \beta_n = \frac{1}{T_n} \leq 3{,}0$$

dynamic factors β_n are recorded on Table 5.7. and 5.8.

Table 5.7.

Building Type	Rigidly Supported Structures					
	1.mode		2.mode		3.mode	
	β_{1y}	β_{1x}	β_{2y}	β_{2x}	β_{3y}	β_{3x}
11 Stories	2,50	2,77	3,0	3,0	3,0	3,0
5 Stories	3,0	3,0	3,0	3,0	3,0	3,0

Table 5.8.

Building Type	Elastically Supported Structures					
	1.mode		2.mode		3.mode	
	β_{1y}	β_{1x}	β_{2y}	β_{2x}	β_{3y}	β_{3x}
11 Stories	0,6	0,75	3,0	3,0	3,0	3,0
5 Stories	1,58	2,10	3,0	3,0	3,0	3,0

Remark: One can essentially reduce dynamic amplification factors by making use of elastic support conditions.

Damping factor is taken for

$$\psi = 1{,}33$$

as prescribed for shear wall structures. (Design Guidlines, Chapt.4.3 - Table IV.) [12]

Regional Characteristics

Seismic coefficient: $k_g = 0{,}025$ ($I_o = 7$ seismic intensity)

Basement factor: $k_t = 1{,}0$ (shallow foundation)

Protection factor: $k_s = 1{,}0$ (III.Cathegory)

(The Design Guidline, Tables I, II, III.)

5.3.6. Equivalent Static Lateral Load

Non mode dependent part of lateral loading:

$$q_o = k_g\, k_t\, k_s\, \psi\, 10\ \text{m} = 0{,}025 \times 1{,}0 \times 1{,}0 \times 1{,}33 \times 10 \times 10{,}89 = 3{,}62\ \text{Mp/m} = 36{,}2\ \text{kN/m}$$

The mode dependent part of lateral loading calculated from η_n (modal participation) and β_n (dynamic) factors

$$k_n = \beta_n \eta_n \quad \text{with} \quad n = 1,2,3 \quad \text{and}$$
$$\eta_1 = 1,571; \quad \eta_2 = -0,712; \quad \eta_3 = 0,136$$

is collected in Table 5.9.

Table 5.9.

Building Type	Modal factors					
	k_1		k_2		k_3	
	k_{1y}	k_{1x}	k_{2y}	k_{2x}	k_{3y}	k_{3x}
11 Stories	0,943	1,18	-2,136	-2,136	0,408	0,408
5 Stories	2,482	3,299	-2,136	-2,136	0,408	0,408

The value of the loading in the u-st mode can be written as

$$q_{dn} = q_o k_n$$

The"root of sum of squares" extended over n = 1,2,3 provides the probable values of the distributed lateral loads such as:

$$p_{dy} = q_o k_{1y} \sqrt{1 + \left(\frac{k_{2y}}{k_{1y}}\right)^2 + \left(\frac{k_{3y}}{k_{1y}}\right)^2}$$

and

$$p_{dx} = q_o k_{1x} \sqrt{1 + \left(\frac{k_{2x}}{k_{1x}}\right)^2 + \left(\frac{k_{3x}}{k_{1x}}\right)^2}$$

The design values of seismic loading is calculated for three severity ranges with the help of load factors given as:

- $n_o = 1,0$ serviceability level,
- $n_1 = 1,4$ limit level,
- $n_2 = 1,8$ accidental level.

The distributed lateral load at top (z = H), taking the serviceability loading level in case of our representative, 11 storey high building are

$$p_{dy}(H) = 36,2 \times 0,943 \times 2,5 = 85,34 \text{ kN/m}$$
$$p_{dx}(H) = 36,2 \times 1,18 \times 2,1 = 89,70 \text{ kN/m}$$

and in case of a 5 storey high building

$$p_{dy}(H) = 36,2 \times 2,482 \times 1,33 = 119,50 \text{ kN/m}$$
$$p_{dx}(H) = 36,2 \times 3,299 \times 1,20 = 143,31 \text{ kN/m}$$

The distribution of these loadings with respect to the height is accordance with the first mode shape, described by the normalized displacement function [11]

$$u_1\left(\frac{z}{H}\right) = f\left(\sin \beta_1 \frac{z}{H}, \text{ sh } \beta_1 \frac{z}{H}, \cos \beta_1 \frac{z}{H}, \text{ ch } \beta_1 \frac{z}{H}\right)$$

The shear at base is calculated from

$$V(0) = \int_0^H p_d\, u_1 \left(\frac{z}{H}\right) dz$$

which provides in direction x and y (Fig.5.4)
- in case of the 11 storey high building

$$V_y(0) = 85,34 \times 0,617 \times 33,83 = 1781,31 \text{ kN}$$
$$V_x(0) = 89,70 \times 0,617 \times 33,83 = 1872,32 \text{ kN}$$

- in case of the 5 storey high building

$$V_y(0) = 119,50 \times 0,617 \times 17,20 = 1268,18 \text{ kN}$$
$$V_x(0) = 143,31 \times 0,617 \times 17,20 = 1520,86 \text{ kN}$$

The moments at base
- in case of the 11 storey high building

$$M_x(0) = 85,34 \quad 0,289 \quad 33,83^2 = 28226,33 \text{ kNm}$$
$$M_y(0) = 89,70 \quad 0,289 \quad 33,83^2 = 29668,41 \text{ kNm}$$

- in case of the 5 storey high building

$$M_x(0) = 119,50 \quad 0,289 \quad 17,20^2 = 10216,98 \text{ kNm}$$
$$M_y(0) = 143,31 \quad 0,289 \quad 17,20^2 = 12252,68 \text{ kNm}$$

Distribution of Internal Actions between Coupled Wall Members

Internal actions are to be distributed between the coacting set of coupled shear walls according to the stiffness ratia. For the sake of convenience we try to demonstrate the procedure by selecting two prototypes of wall members.

These are (see Figure 5.6)
- $I_{x5} = 5,4 \text{ m}^4$ without lintel row, but having an internal and an external (face) vertical joint,
- $I_{yII} = 5,3 \text{ m}^4$ described as a coupled shear wall, with two rows of window openings, a vertical joint at the corner of the cross section and a further external vertical joint.

Moments and shear forces absorbed by I_{x5} and I_{y11} at a service severity level (i=0) in case of an 11 storey high building are;

$$M_{x5} = \frac{I_{x5}}{I_x} M_x = \frac{5,4}{120,9} \times 28226,33 = 1260,73 \text{ kNm}$$
$$V_{y5} = \frac{I_{x5}}{I_x} V_y = \frac{5,4}{120,9} \times 1781,31 = 79,56 \text{ kN}$$
$$M_{yII} = \frac{I_{yII}}{I_y} M_y = \frac{5,3}{149,6} \times 29668,41 = 1051,08 \text{ kNm}$$
$$V_{xII} = \frac{I_{yII}}{I_y} V_x = \frac{5,3}{149,6} \times 1872,32 = 66,33 \text{ kN}$$

- in case of a 5 storey high building are:

$$M_{x5} = 456,34 \text{ kNm}$$
$$V_{y5} = 56,64 \text{ kN}$$

$$M_{yII} = 434,08 \text{ kNm}$$
$$V_{xII} = 53,88 \text{ kN}$$

Shear forces acting in joints

With regard to the uniformity of the jointing system in both (5 and 11 stories) representative large panel buildings, we shall proceed with the analysis of an 11 storey high building.

Wall X_5

Internal vertical joints at ground floor are acted on by a shear flow of intensity

$$v_x = \frac{VS}{I_x} = \frac{V}{5,4}\, 0,15 \times 1,57 \times \frac{5,40}{2} = 0,118 \text{ V}$$

Hence the shear force yields (M = pure bending)
- at service loading level (i=0), load factor = 1,0

$$V_{x5} = v_x h = 0,118 \times 2,8 \times 79,56\, \frac{1}{0,8} = 32,86 \text{ kN}$$

- at limit loading level (i=1), load factor = 1,4

$$V_{x5} = 1,4 \times 32,86 \times \frac{0,8}{0,6} = 61,34 \text{ kN}$$

- at accidental loading level (i=2), load factor = 1,8

$$V_{x5} = 1,8 \times 32,86 \times \frac{0,8}{0,4} = 118,30 \text{ kN}$$

Wall Y_{II}

External vertical joint at face is acted on by a shear flow of

$$v_y = \frac{VS_1}{I_y} = \frac{V}{5,3}\, 0,15 \times 1,85\, \frac{5,40}{3} = 0,094 \text{ V}$$

Hence the shear force yields
- at service loading level (i=0) to

$$V_{xII} = v_y h = 0,094 \times 2,8 \times \frac{66,33}{0,8} = 21,82 \text{ kN}$$

- at limit loading level (i=1) to

$$V_{xII} = 1,4 \times 21,82 \times \frac{0,8}{0,6} = 40,73 \text{ kN}$$

- at accidental loading level (i=2) to

$$V_{yII} = 1,8 \times 21,82 \times \frac{0,8}{0,4} = 78,55 \text{ kN}$$

External vertical joint at the corner is acted on by a shear flow of intensity

$$v_y = \frac{V}{5,3} \times 0,15 \times 1,20 \times 2 \times \frac{5,40}{3} = 0,122 \text{ V}$$

hence at three limit states

$$V_{yII} = 0,122 \times 2,80 \times \frac{66,33}{0,8} = 28,32 \text{ kN}$$

$$V_{yII} = 52,86 \text{ kN}; \qquad V_{yII} = 101,95 \text{ kN}$$

5.4. PROPORTIONING

5.4.1. Capacity Ranges of Keyed Vertical Joints

Load boaring capacity ranges of keyed vertical joints (Figure 5.7) in pure shear are given in Table 5.10. Values are taken from the Hungarian Building Code for Large Panel Buildings (ME-95-81). Formulae used here are basecally adapted relations of Hansen-Olsen, Pomeret, CIB/W 23 A, for static loading [2],[3]. Dynamic strength and stiffness degradation in considered on the loading side (load factor, reduced moment of inertia).

Table 5.10.

Stability Range	V_L (kN) Limit Shear Resistance		
	Internal joints	External joints	
		at face	at corner
Serviceability (i=0)	173,1	149,3	55,7
Limit (i=1)	249,3	213,3	79,6
Ultimate (i=2)	299,2	255,9	95,5

Remark: Limit shear resistances plotted here, refer to the storey height: h = 2,80 m.

5.4.2. Stability Control of Keyed Vertical Joints

The stochastic dependence of loading (X_i) and resistance (Y_i) requires that

$$X_i \leq Y_i$$

condition should be satisfied (See Part I.). The control technique is shown in Table 5.11.

Table 5.11.

Type of joint	Response Level (kN)		
	i=0	i=1	i=2
Internal	32,86 < 173,1	61,34 < 249,3	118,30 < 299,2
External at face	21,82 < 149,3	40,73 < 213,3	78,55 < 255,9
External at corner	28,32 < 55,7	52,86 < 79,6	101,95 > 95,5

<u>Remark:</u> Joints at building corners are - as expected - in the most unfavourable position when a strong (accidental) ground motion takes place.

5.4.3. <u>Energy dissipation control of lintels</u>

The rotational ductility of the cross section in pure bending is calculated from [4]

$$d_\rho = \frac{d_{c\varepsilon}}{2\rho\mu}\frac{M_{cE}}{M_u}$$

where:

$d_{c\varepsilon} = \dfrac{4\ \text{o/oo}}{1\ \text{o/oo}}$ is the strain ductility of the concrete,

$\rho = \dfrac{f_y}{f'_c}$ is the strength ratio,

$\mu = \dfrac{A_s}{A_c}$ is the steel reinforcement ratio,

M_u = ultimate (first yield) bending moment of the cross section,

$M_{cE} = 150 \times 15 \times 50^2/6 = 937500$ kpcm $= 93{,}75$ kNm the moment in elastic (uncracked) range.

With the numerical values taken from Figure 5.8. the depth of the neutral axis is

$$x_p = \frac{2400 \times 8{,}0}{150 \times 15} = 8{,}5 \text{ cm}$$

and the moment at first yield

$$M_u = 2400 \times 8{,}0\ (45 - 4{,}25) = 782400 \text{ kpcm} = 78{,}24 \text{ kNm}$$

The rotational ductility

$$d_\rho = \frac{4 \times 10^2}{2 \times 16 \times 1{,}07}\ \frac{937500}{782400} = 14{,}00$$

The displacement at first yield

$$u_E = \frac{M_u}{6EI}\ \ell_c^2$$

The plastic displacement

$$u_p = \frac{M_u}{EI}\ \ell_p\ \ell_c\ (d\rho - 1)$$

and the total displacement with

$$\ell_p = 0{,}7\ b = 0{,}7 \times 15 = 10{,}5 \text{ cm}$$

$$u_u = \frac{M_u}{6EI}\ \ell_c^2 \left[1+6\ \frac{\ell_p}{\ell_c}\ (d_\rho - 1)\right] = \frac{M_u\ \ell_c^2}{6EI}\left[1+6\ \frac{10{,}5}{100}\ (14{,}00-1)\right] = \frac{M_u\ \ell_c^2}{6EI}\ (1+8{,}19)$$

The displacement ductility factor

$$d_u = 9{,}19 > 4 \quad \text{(required)}$$

5.5. SELECTED DETAILS

Components of the Hungarian large panel system are unified all over the country. Floor alab components of standarised spans: 1800, 2700, 3600, 4500, 5400 (6300) mm are represented in Figure 5.9. The slab reinforcement is shown in Figure 5.10. Typical internal wall components with door openings, following the previous modular order, are precast according to the details given in Figure 5.11. Geometric properties of Sandwich type external walls are shown in Figure 5.12. Jointing details of different precast members are represented in Figure 5.13, 5.14, 5.15, 5.16.

5.6. CONCLUSIONS

5.6.1. A system oriented design approach may produce adequate structural solutions to resist at a limit strong ground motions. Due to the hierarchical differentiation of system components, partitioning of jointing system and reserve energy technique, achived by a high degree of ductility and redundancy, one can minimize the probability of a particular form of structural instability known as progressive collapse.

However a properly prudent level of risk should be based on a well balanced safety and economy.

5.6.2. Soil-structure interaction phenomena included in dynmic analysis, provides a more objective structural (physical) model, with more realistic response spectrum. Since surface layers may significantly modify earthquake effects on structures, which can be used to the benefit of the structural performance, soil-structure interaction technique becomes a tool in regulating seismic response.

Refering to the presented sample building a reduction in dynamic factor of 4 to 1 may be achived by introducing in the calculations the support condition.

5.6.3. Storey hight, slenderness resp. is in favour of seismic response as for as stability in inelastic deformation is preserved.

5.6.4. Needs:

a) interaction diagrams for combined axial force, bending and shear (N, M, V),
b) flexibility criteria of floor diaphragms,
c) skeleton curves of vertical and horizontal joints under cyclic (reversal) loading,
d) reduced cross sectional characteristics (A, I) in different limit states (serviceability ultimate),
e) over-all stability control of three dimensional structural models under combined action (biaxial bending, compression, torsion),
f) revised storey drift levels, local stability criteria (CEB Recommandations are conservatives.

Remark: This contribution has been prepared by B.Goschy (Geotechnical Institution, Budapest) in collaboration with J.Gyurko (Building Research Institut, Budapest)

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NOTATIONS

Report		CEB
A_c	net concrete cross sectional area	A_c
A_s	area of the steel reinforcement	A_s
C_ϕ	rocking subgrade modulus	
E_c	Young's modulus of concrete	E_c
E_s	Young's modulus of steel	E_s
G	permanent (dead) load	G
H	height of the building	h
I	second moment of area	I
M	flexural moment	M
M_t	torsional moment	M_t
N	normal (axial) load	N
P	live load	Q
V	shear force	V
T	period of free vibrations	T
d	ductility	μ
e	excentricity	e
f'_c	concrete strength	f'_c
f'_y	steel yield strength	f'_y
g	acceleration of the gravity	g
i_p	polar radius of inertia	i_p
k_s	importance factor (Table II)	
k_t	site coefficient (Table III)	S
k_g	design seismic coefficient (Table I)	C_d
ℓ	span	ℓ
m	uniformly desitributed mass	m
x	coordinates	x
y	coordinates	y
z	coordinates	z
x_p	neutral axis depht	x
v	shear flow	
V_c	shear strength of concrete	V_c
β	dynamic factor	ω
ε_c	strain in concrete	ε_c
ω	circular frequency	

Report		CEB
ψ	damping factor	η
μ	steel reinforcement ratio	ρ
ρ	strength ratio	
σ	stresses	σ

GENERAL FLOOR LAY-OUT

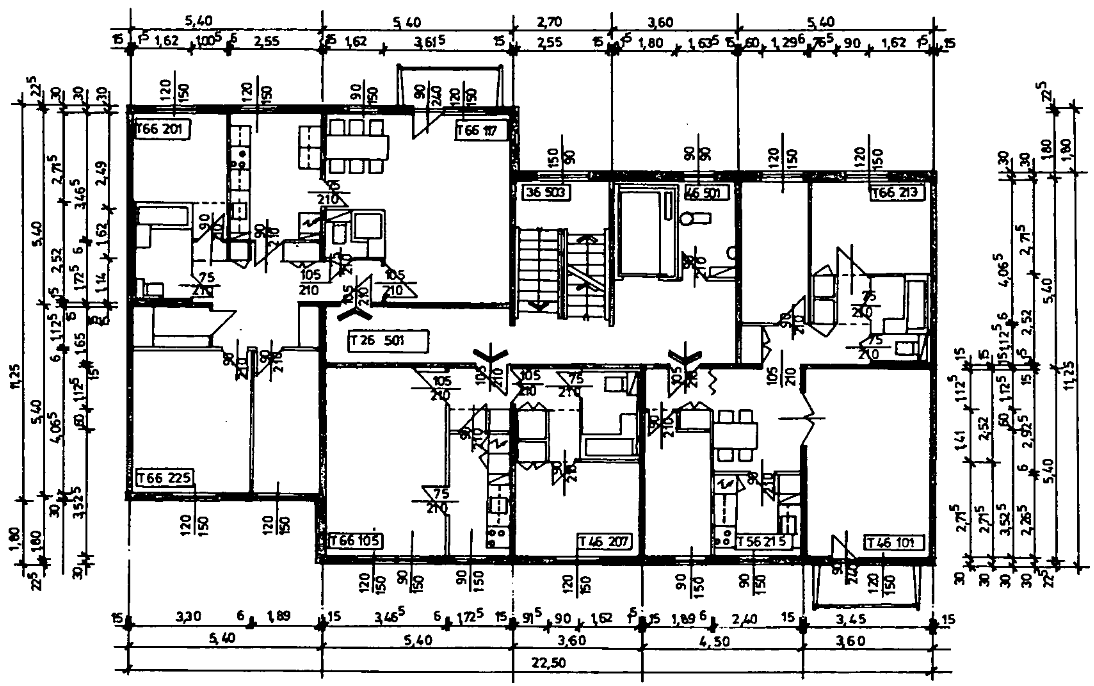


Figure 5.1

ELEVATION

11 Storey High
Large Panel Building

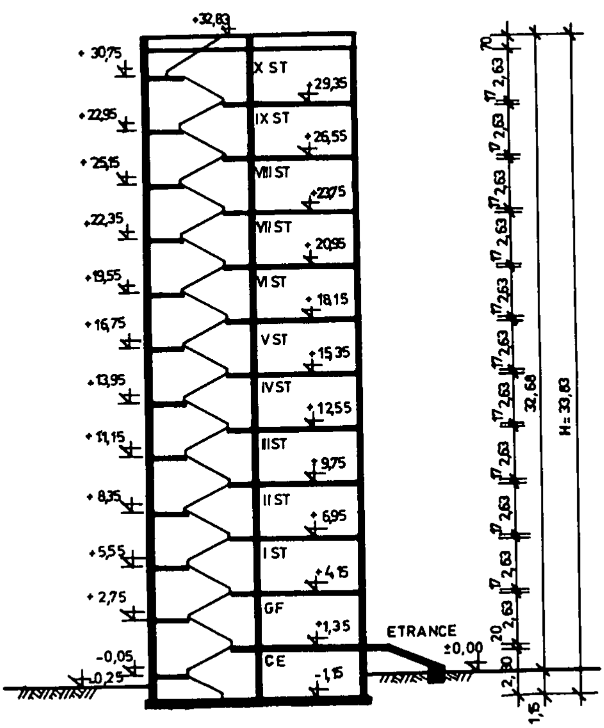


Figure 5.2

ELEVATION

5 Storey High

Large Panel Building

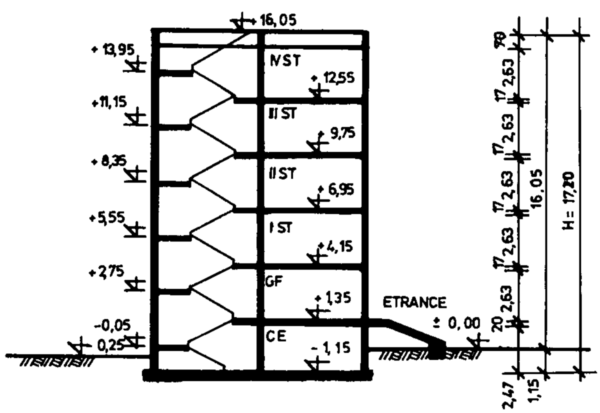
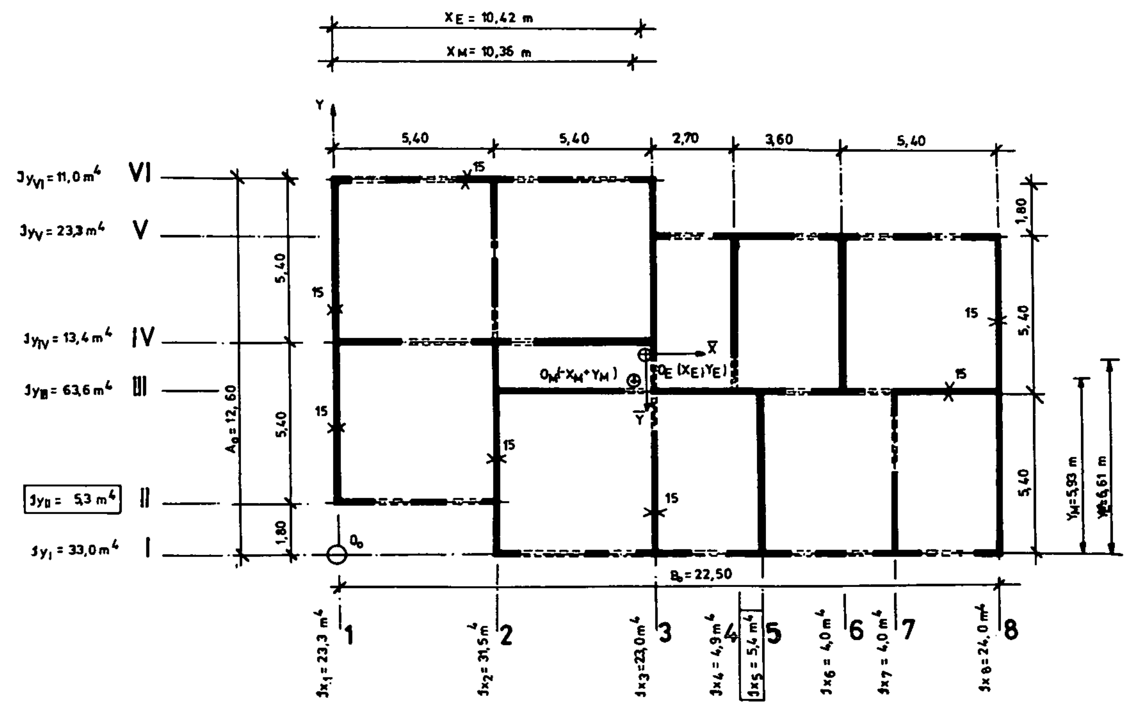


Figure 5.3

CROSS SECTIONAL CHARACTERISTICS



Remark : Subscripts x and y of wall inertial moments refer to their bending axes

Figure 5.4

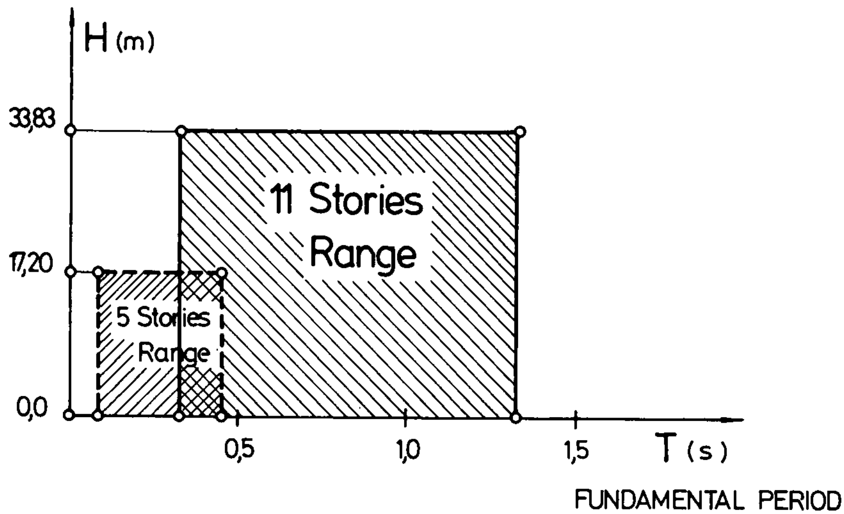

H (m)
33,83
17,20
0,0
11 Stories
Range
5 Stories
Range
0,5
1,0
1,5
T (s)
FUNDAMENTAL PERIOD

Figure 5.5

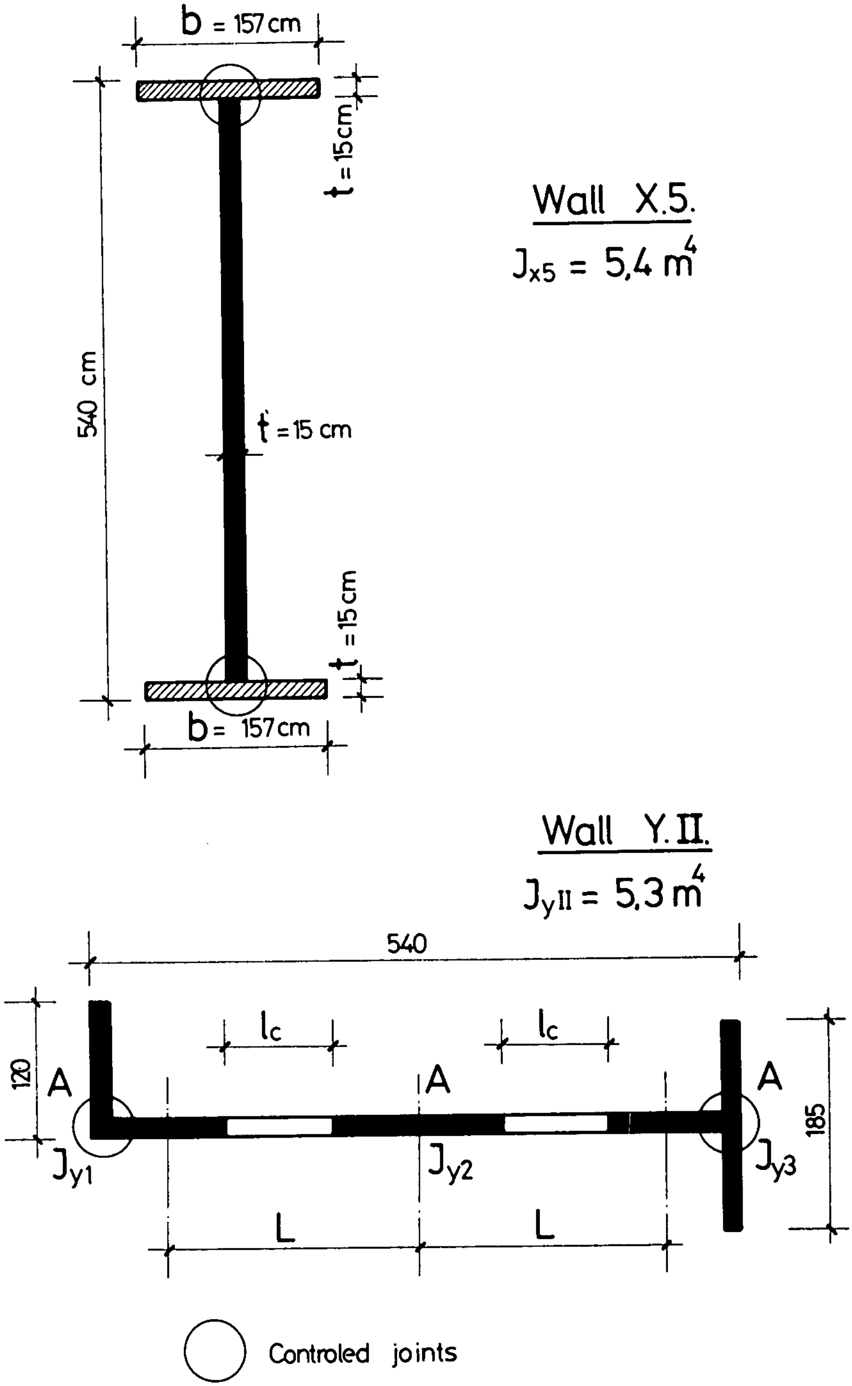
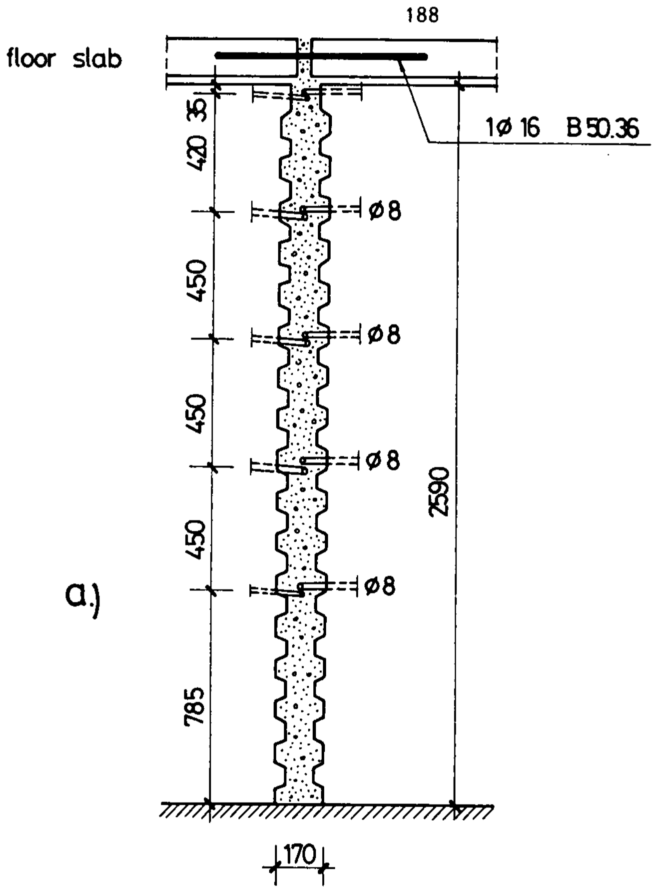
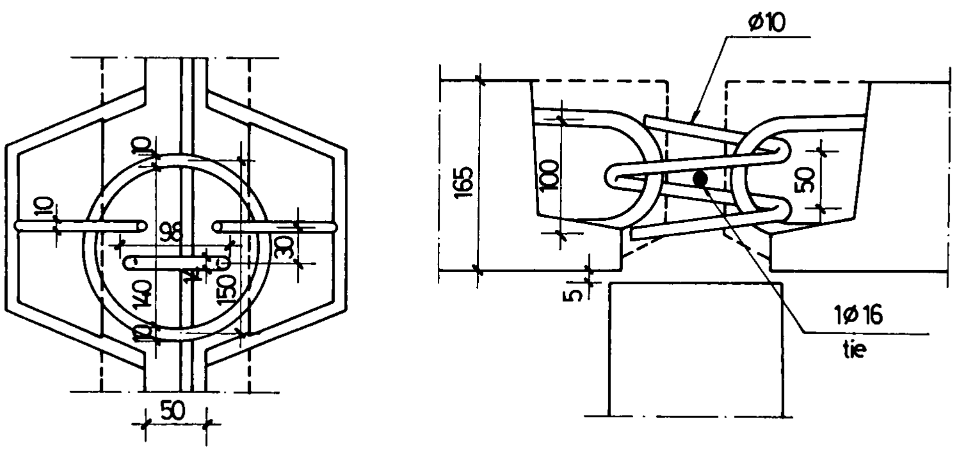

b = 157 cm
t = 15 cm
Wall X.5.
Jx5 = 5,4 m4
540 cm
t' = 15 cm
t = 15 cm
b = 157 cm
Wall Y. II.
JyII = 5,3 m4
540
120
A
lc
A
lc
A
185
Jy1
Jy2
Jy3
L
L
Controled joints

Figure 5.6

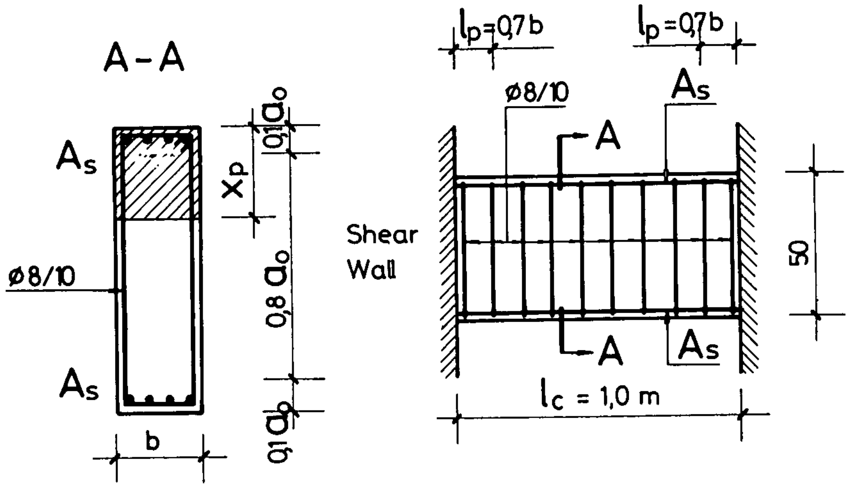


Vertical keyed joint



Jointing at floor slabs

Figure 5.7

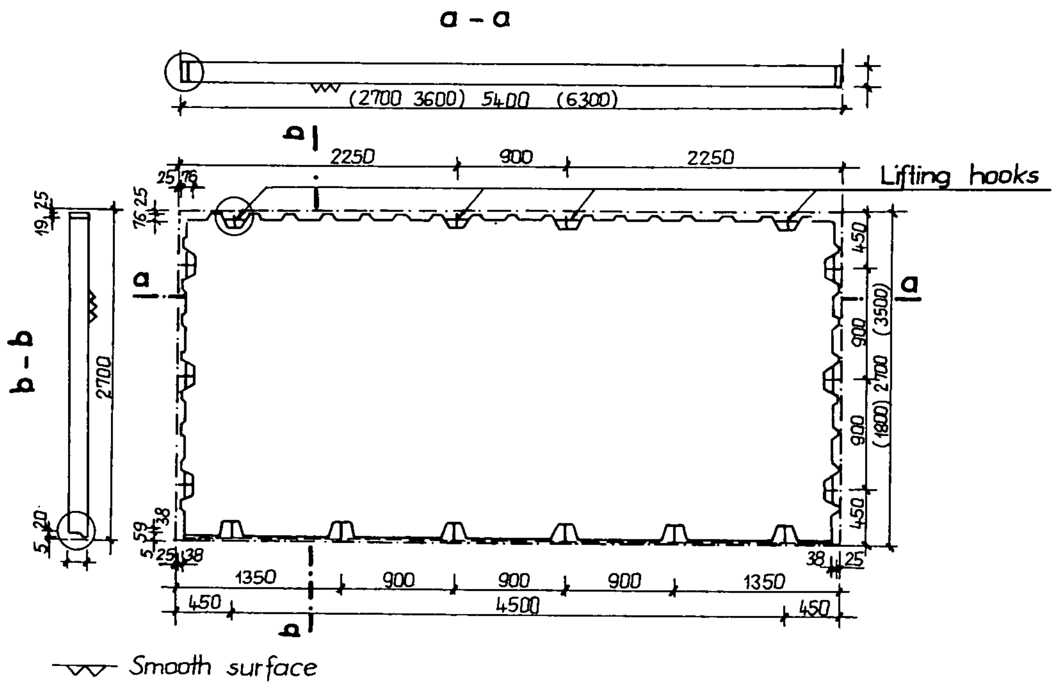


a_o = 50 cm ; b = 15cm

A_s = 8,0 cm^2 (4ϕ16)

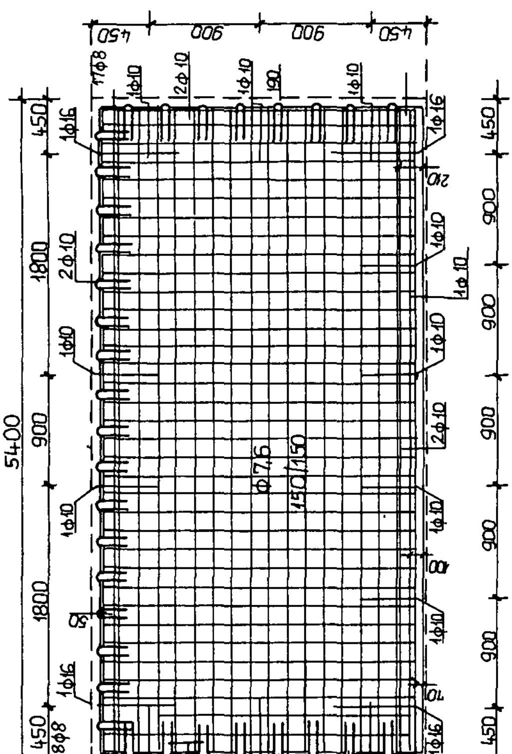
A_c = 15x150 = 2250 cm^2

Figure 5.8



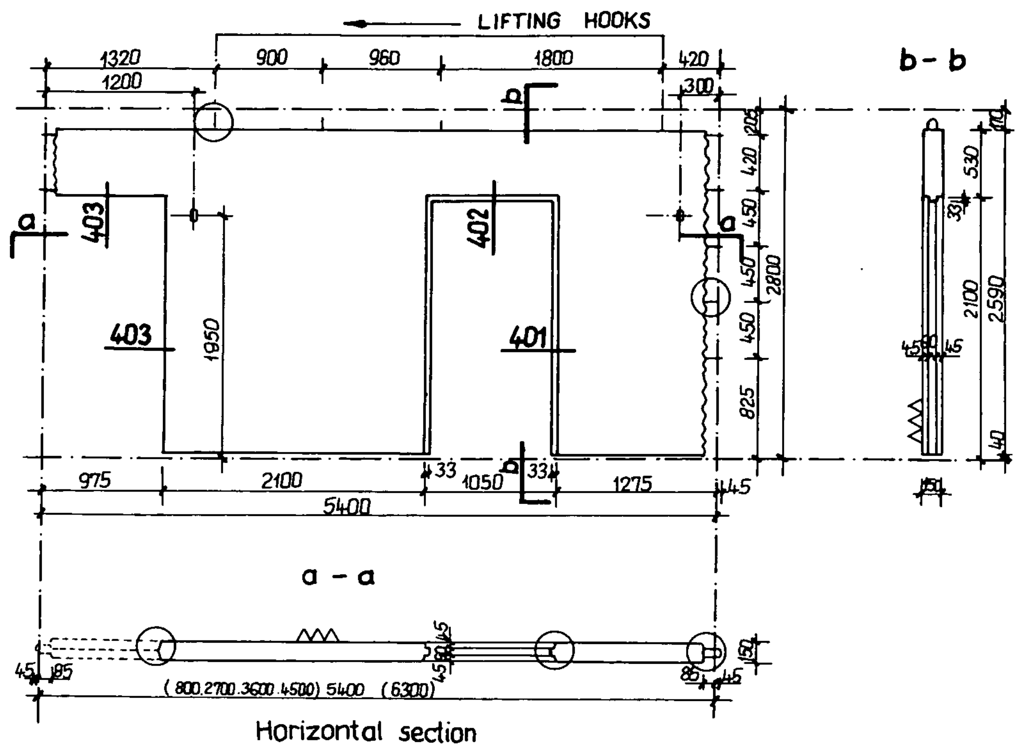
FLOOR SLAB COMPONENT

Figure 5.9



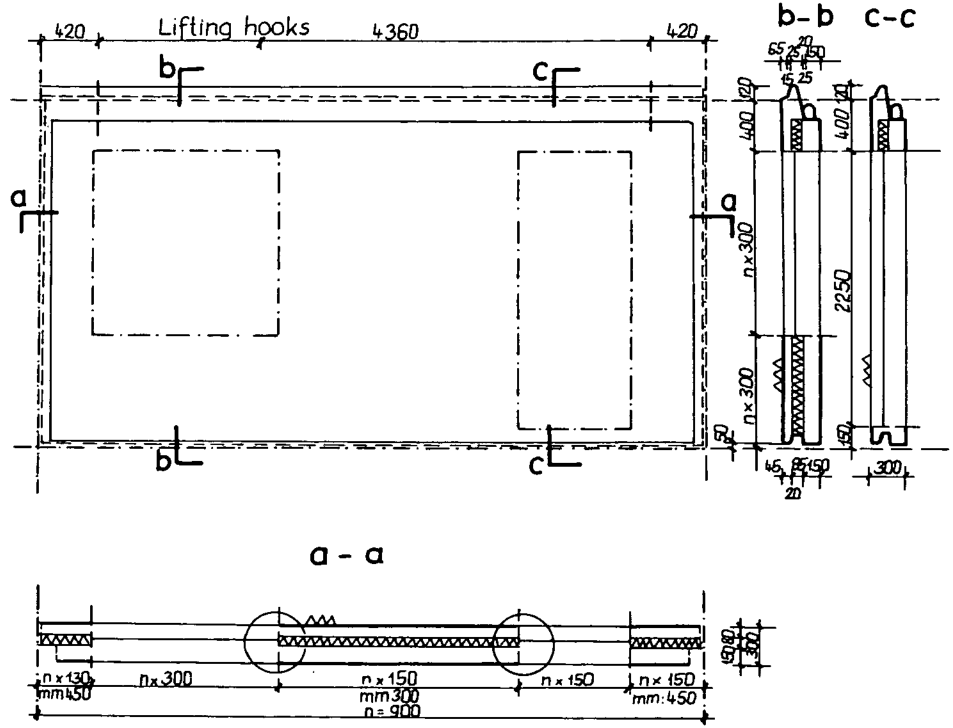
FLOOR SLAB REINFORCEMENT

Figure 5.10



INTERNAL WALL COMPONENT

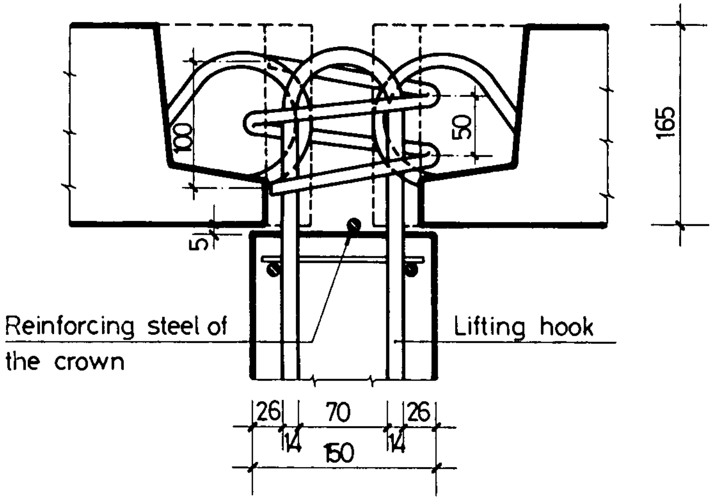
Figure 5.11



EXTERNAL WALL COMPONENT

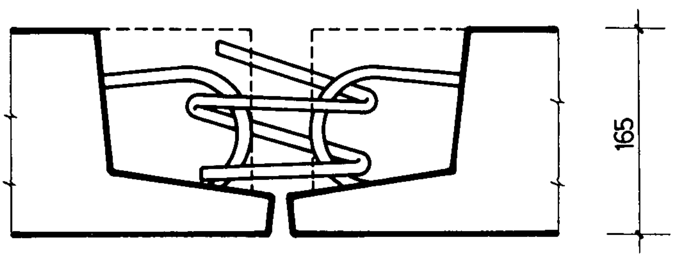
Figure 5.12

a.)



SLAB-SLAB-WALL JOINT

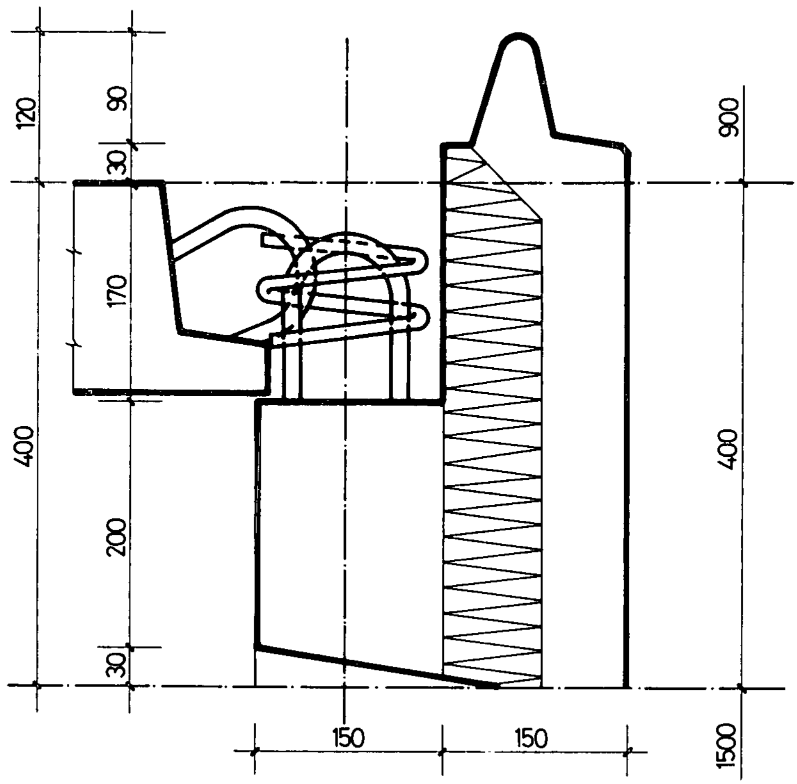
b.)



SLAB-SLAB JOINT

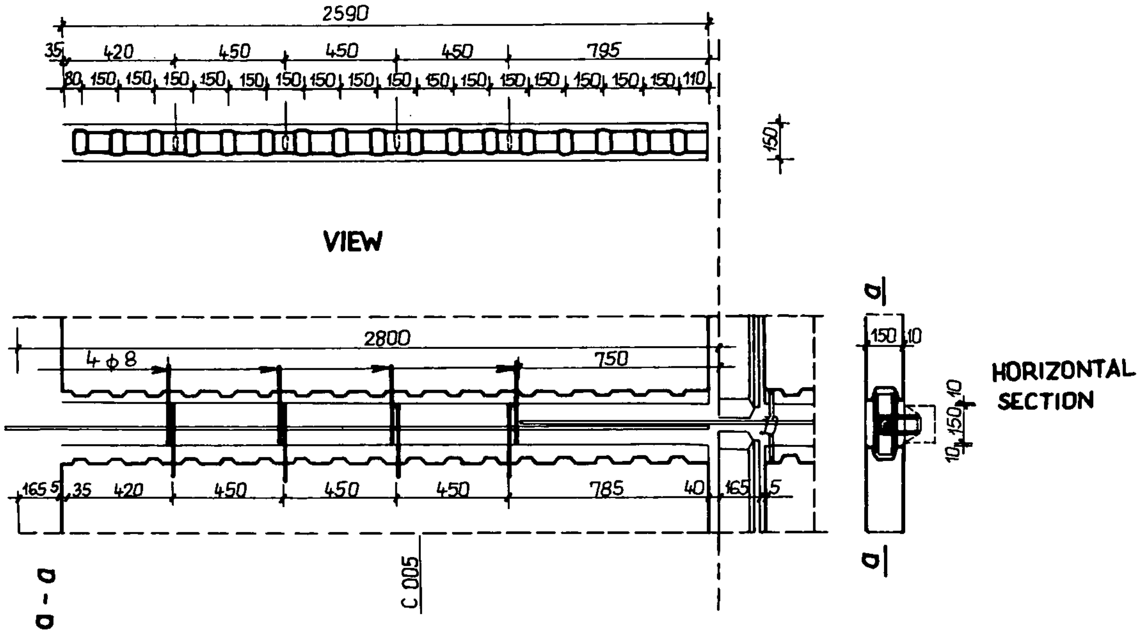
Figure 5.13

C.)



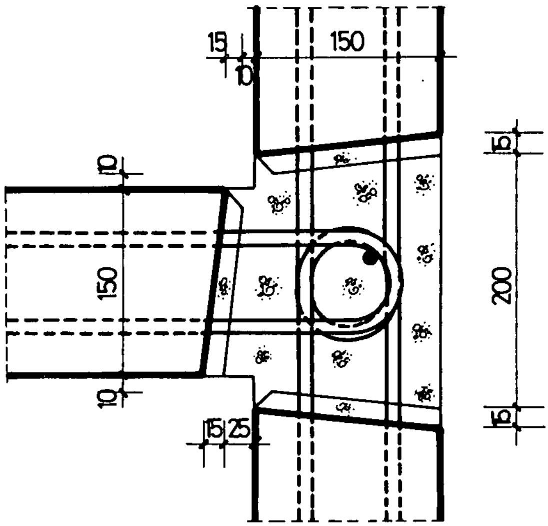
FLOOR SLAB-EXTERNAL WALL JOINT

Figure 5.13



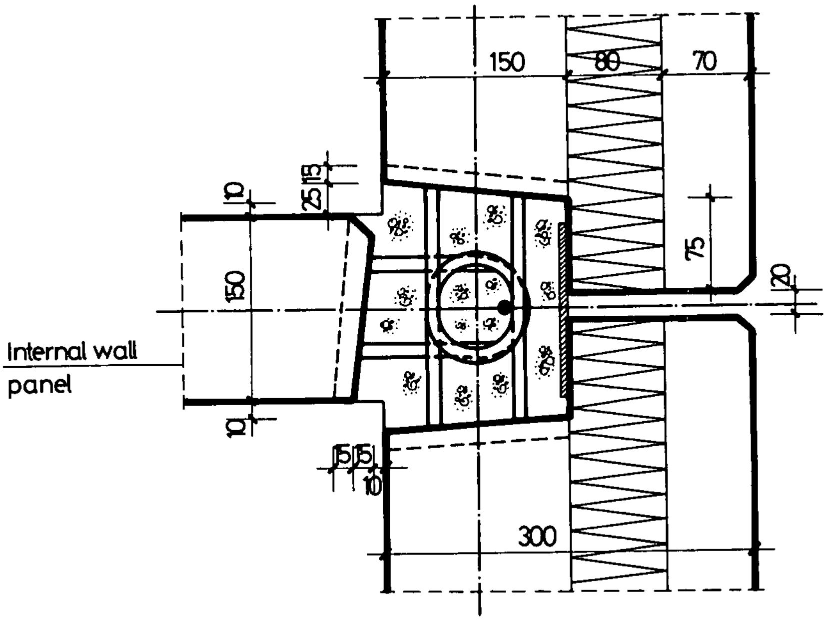
HORIZONTAL JOINT SYSTEM-INTERNAL WALL PANEL

Figure 5.14



VERTICAL JOINT OF THREE INTERNAL WALL PANELS

Figure 5.15



VERTICAL JOINT OF EXTERNAL INTERNAL WALLS

Figure 5.16

6. REPRESENTATIVE EXAMPLE OF ROMANIA

9-storey frame structures with precast elements

Prepared by DAN CONSTANTINESCU
Associate Professor Fac. of Civil Engrg.,
Bucharest, Romania

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6.1 SCOPE

The example deals with design of a RC multistorey frame structure made with precast elements. The building is a nine-storey apartment house and is to be erected in Bucharest. The plan and vertical configurations of structure are presented in Figs.6.1 and 6.2.

The example is typical for the current design practice in Romania when dealing with seismic-resistant, RC frames. The governing codes of practice are [6.9] and [6.10].

6.2 DESCRIPTION OF STRUCTURAL SYSTEM

The seismic-resistant system is made of horizontal diaphragms and space frames forming an orthogonal network. Precast panels are used for floor slabs, beams are composite and columns are cast-in-situ.

Floors are made of prefabricated semi-panels having a rectangular shape of 5.65 x 2.575 sqm and a constant depth of 0.14 m (see Fig.6.3). The semi-panels must remain fully supported till the in-situ concrete in connections has gained sufficient strength. From then on the floor slab is assumed to behave monolithically. The floor connections are detailed in Fig.6.3.

Table 6.1. Gravity loads on apartment floor

Type	Unit	Specified value	Load factor	Design value
weight of RC slab, 0.14 m depth	kN/m^2	3.5	1.1	3.85
weight of flooring	kN/m^2	0.75	1.2	0.92
partitioning walls (equivalent uniformly distributed weight)	kN/m^2	3.5	1.1	3.85
live load	kN/m^2	1.5	1.2	1.80
Total gravity load on floor slab	kN/m^2	9.25		10.42
weight of precast beam	kN/m	4.90	1.1	5.39

N.B. The required strength of floor slab is computed with the design values of gravity loads. The design value is equal to the specified value multiplied by the load factor.

The lateral faces of semi-panel are provided throughout its perimeter with:
- loop-shaped projecting bars of 8 mm diameter and spaced at 0.20 m (see Fig.6.3);

- a vertical slope to transmit the gravity loads to the supporting beams;
- sockets of 0.1 x 0.1 sqm surface area and 25 mm depth to transmit both the vertical and in-plane shear forces.

The 0.14 m depth of the floor slab is required to limit its vertical deflection (the depth to span ratio of the floor slab is about 1/40). At the same time, the resulting dead weight of about 3.5 kN/sqm ensures a good sound insulation between storeys.

The use of precast semi-panels rather than of precast panels arose from the requirement that the dead weight of precast member should not exceed the minimum load capacity of the cranes currently available in the field.

The specified and design values of gravity loads on the apartment floor are given in Table 6.1. The total gravity loads on the roof floor slab are 7.56 kN/sqm (specified value) and 8.92 kN/sqm (design value).

Multistorey Frame System is orthogonal and regular. Beams in both directions of the floor are composite: precast on a depth of 0.56 m and cast-in-situ on the floor slab depth (see Fig. 6.3). The transfer of horizontal shear at the interface between the soffit beam and the in-situ section of beam is ensured by means of two leg stirrups projecting from the soffit beam. Shear-keys are also provided on the upper face of soffit beam. The precast beam spans between columns. Each end face has shear-keys to transmit the vertical shear force to supporting column. The bottom reinforcing bars project from soffit beam at both ends whereas the top reinforcing bars are placed within the in-situ section (see Fig.6.4).

Columns and beam-column connections are cast-in-situ. Such a structure resembles a monolithic space frame in both performance and strength.

The erection of a structure storey requires the following steps:
- the clear height of columns is cast-in-situ using steel-made, ready-adaptable forms;
- the precast soffit beams in both directions and the floor semi-panels are placed and temporarily supported in position on the floor beneath;
- the connections (semi-panel to semi-panel, beam to semi-panel and beam to column) are cast-in-situ;
- the temporary supports of floor slabs and beams are removed after the in-situ concrete has gained sufficient strength.

Beams throughout the structure have the same cross-section. The 0.70 m height results from the heights of storey and doors (2.80 - 2.10 = 0.70 m) whereas the 0.35 m width is to provide good shear strength of beam and confinement of beam-column joint. The precast members are made with B 250 concrete whereas the in-situ connections are cast with B 300 concrete (see Table 6.2).

Columns have the cross-section area, A_b, required by the limitation

$$n = \frac{N}{A_b\ R_c} < \begin{cases} 0.6 \text{ for central columns} \\ 0.45 \text{ for lateral columns} \end{cases} \qquad (6.1)$$

where N is the axial force due to specified gravity load and R_c is the design compressive strength of concrete (see Table 6.2).

Table 6.2. Characteristics of concrete

Type[1)]	Design strength, MPa			Modulus of elasticity (E_c), MPa
	Compression (R_c)		Tension(R_t)	
	Beam	Column		
B 200	9.5	8.0	0.8	24,000
B 250	11.5	10.0	0.9	27,000
B 300	14.0	12.0	1.0	29,000

[1)] The type of concrete is defined in conjunction with the mean compressive strength tested on 20 cm cubes at the age of 28 days. For instance, a B 250 concrete has the above defined strength of 25 MPa.

Table 6.3. Characteristics of reinforcement

Type	Bar	Guaranteed yield strength, MPa	Design yield strength, MPa
OB 37	Plain	240	210
PC 52	Deformed	340	290
PC 60	Deformed	400	340

Eq.(6.1) provides RC cross-sections subjected to excentric compression with a minimum curvature ductility [6.2], [6.5], [6.8]. The more severe limit imposed on n - value for lateral columns accounts for the extra axial force induced by the seismic action. In order to contain the variation of column cross-section from one storey to another (which would negatively affect the vertical distribution of drift stiffness and would increase the number of span lengths of precast soffit beams), the quality of concrete cast in columns is varied along the building heigth. Thus, the first two storeys use B 300 concrete, the next four use B 250 concrete and the last three use B 200 concrete. Moreover, the dimension of column cross-section in the longitudinal direction of structure is held constant and equal to 0.60 m. On this account the columns have the cross-sections in Fig.6.2 while all longitudinal beams have the same span length (5.40 m). Transverse beams have but three different span lengths (see Table 6.4 in § 6.5).

6.3. STRUCTURAL ANALYSIS

6.3.1. Structure Response to Gravity Loads

The two-way slab transmits the gravity loads in Table 6.1 to both transverse and longitudinal frames (see Fig.6.5).

The internal forces induced by the gravity loads are computed using a linear analysis. The effects of both axial force and longitudinal reinforcement on flexural stiffness of frame members are not taken into consideration. On account of different values used for load factors when finding the required strength of structural members (see § 6.3.3), the structure response is obtained separately for the following three types of specified gravity loads:

G_1 - dead load of structure and partitioning walls;
G_2 - dead load of flooring and equalizing pad; (6.2)
P - live load.

6.3.2. Structure Response to Seismic Action

The analysis is based on the equivalent lateral force procedure [6.10]. The seismic base shear S is given by

$$S = k_s \beta \Psi \varepsilon\ G \tag{6.3}$$

where

G is the total effective weight of building;

k_s is a factor accounting for the seismic intensity of site and seismic hazard exposure. $k_s = 0.2$ for an apartment house built in Bucharest;

β is an amplification factor dependent on the fundamental period of vibration of structure, T, and on the type of foundation soil. $\beta = 2.0$ for normal soil conditions and T not greater than 1.5 sec.;

Ψ is a reduction factor which accounts for the structure capability to deform inelastically. $\Psi = 0.2$ for multibay, multi-storey RC frames;

ε is a factor which accounts for the equivalence between the mode shape of structure and that of inverted dynamic pendulum. The ε - value is determined approximately in accordance with

$$\varepsilon \simeq \frac{\Sigma G_j U_j}{G} \tag{6.4}$$

where G_j is the portion of G located at level j while U_j is the displacement amplitude at the j-th level of building (see Fig.6.6). For the multistorey frame analysed here,

$$\varepsilon \simeq 0.8 \tag{6.5}$$

On account of above values, Eq. (6.3) yields

$$S = 6.4\%\ G \simeq 2280\ \text{kN} \tag{6.6}$$

for both transverse and longitudinal directions of structure. In the previous relationship, G = 35,640 kN and corresponds to about 10.5 kN gravity load, uniformly distributed on each square meter of every floor.

The seismic base shear is assumed to occur separately on each principal direction of structure, that is transverse and longitudinal. Lateral forces equal to 0.7 S and acting simultaneonsly on both principal directions of structure should also be considered [6.10] but this situation is neglected here for the sake of simplifying the design example. It is aparent that the longitudinal reinforcement of columns would be affected by considering the oblique seismic action as a consequence of biaxial eccentric compression which occurs in such a situation.

The vertical distribution of seismic forces is determined in accordance with the following formula (see Fig.6.6)

$$S_j \simeq S \frac{G_j h_j}{\Sigma G_j h_j} \qquad (6.7)$$

where S_j is the lateral seismic shear force induced at j-th level. This force is distributed to frames on the appropriate direction with due consideration given to their relative stiffnesses. The horizontal diaphragm is assumed non-deformable and the effect of in-filled walls on frame stiffness is usually neglected. In transverse direction, for instance, all frames have the same stiffness so that the lateral seismic force S_j is equally distributed. Thus the lateral force loading the i-th transverse frame at the level of j-th floor is

$$S^t_{ij} = \frac{S_j}{6} \qquad (6.8)$$

A torsional moment

$$M_{tj} = S_j(e_1 \pm e_2) \qquad (6.9)$$

is associated to seismic force S_j, where e_1 is the eccentricity resulting from the location of mass center (see Fig.6.7) and e_2 is equal to 5% of the dimension of the building perpendicular to the direction of applied forces. Here in e_1=0 on account of the symmetry of structure (see Fig.6.1). The torsional moment induces lateral forces in all the frames (see Fig.6.8). It is conservatively considered here that only the transverse frames oppose the torsional effect of transverse seismic force so that an additional lateral force (see Fig.6.9)

$$S^T_{ij} = S_j e_2 \frac{x_i}{\Sigma x_i^2} \qquad (6.10)$$

is distributed to i-th transverse frame. Therefore, the total lateral force distributed to i-th transverse frame at the level of j-th floor results from formula

$$S_{ij} = S^t_{ij} + S^T_{ij} \qquad (6.11)$$

In view of the regularity of structure in transverse direction, Eqs.(6.8), (6.10) and (6.11) can also be written in terms of seismic base shear, S_i, of i-th transverse frame rather than in terms of S_{ij}. In such a case, S_i is distributed vertically according to Eq.(6.7) where S_{ij} and S_i should replace S_j and S, respectively. For instance, the internal transverse frame farthest located from the symmetry center of structure (i.e.with x_i = 9.0 m in Eq.(6.10) and Fig.6.9)should resist the seismic base shear

$$S_i = \frac{2280}{6} + 0.05 \times 30 \times 2280 \frac{9}{2(3^2+9^2+15^2)} = 49 \text{ kN} \qquad (6.12)$$

The frame response to lateral forces S_{ij} is determined by means of a linear analysis similar to that already used in conjunction with gravity loads.

6.3.3. Required Strength of Structure

The design strength of seismic resistant frames should at least equal the required strength prevailing from the following load combinations:

$$A = 1.1\ G_1 + 1.2\ G_2 + 1.2\ P \qquad (6.13)$$

and

$$A = G_1 + G_2 + 0.4P \pm S \qquad (6.14)$$

where G_1, G_2 and P were denoted in (6.2) and S is the seismic action.

The first combination, called "fundamental", uses the load factors given in Table 6.1. The second combination, called "special", accounts for the higher likelihood of specified seismic action to occur simultaneously with the specified values of dead loads and a reduced live load rather than with the design values of loads.

The axial forces and bending moments induced by the "fundamental" and "special" load combinations in the members of the interior transverse frame are presented in Figs.6.10 and 6.11. It is apparent that the prevailing required strength of both beam and columns ends is given by the "special" load combination.

6.3.4. Ductility Provisions

The use of reduction factor Ψ=0.20 when determining the seismic base shear and therefore the required strength of structural members assumes that structure has sufficient ductility to absorb and dissipate the energy induced by SM earthquakes. It is therefore crucial to locate the structure zones likely to yield during such earthquakes and to know how large the postelastic deformations of these zones would be. With the present state of knowledge, it is rather difficult to answer these questions in a general manner as the answer depends on the structural layout, on the level of yield forces (i.e. the Ψ - value) and on the characteristics of seismic action.

In the absence of definite criteria able to provide the ductility demands of the RC structural members, the following provisions are to be complied with in Romania in order to supply seismic-resistant RC frames with sufficient capacity to deform well into the postelastic range [6.2], [6.5], [6.6], [6.8], [6.9], [6.10]:
- the design is based on weak beam-strong column philosophy in order to ensure that plastic hinges occur primarily within the frame beams (see Eq.(6.58) in § 6.4.3);
- the occurence of plastic hinges is conducted to the end zones of beam span rather than to the middle zones;
- a minimum curvature ductility of beam and column cross-sections is ensured within the zones where plastic deformation is likely to occur during SM earthquakes (see Eq.(6.26) in § 6.4.1 and Eq.(6.1) in § 6.2);
- the brittle failure of beams, columns and joints due to shear forces is avoided by considering the values of shear force associated with the plastic mechanism of every structural member (see Eqs.(6.37), (6.52) and (6.60)).

Moreover, a limitation is imposed on the design storey drift as a general criteria of good seismic performance. Thus, the storey drift Δ induced by the lateral force S/Ψ shall comply with the limitation:

$$\Delta < \frac{H}{N^*} \tag{6.15}$$

where H is the storey height and N* = 200 or 150 depending on whether there are infilled walls or not. The design storey drift, Δ, is determined by the following formula (see Fig.6.12)

$$\Delta = \frac{1}{\Psi}\,\frac{H^2}{24}\left[\frac{M_{ac}+M_{ca}}{(EI)_{ac}} + \frac{M_{bd}+M_{db}}{(EI)_{bd}} + \frac{L}{H}\left(\frac{M_{ab}+M_{ba}}{(EI)_{ab}} + \frac{M_{cd}+M_{dc}}{(EI)_{cd}}\right)\right] \tag{6.16}$$

where M are the bending moments induced by the seismic force S and (EI) are the flexural stiffnesses of uncracked concrete cross-sections. For instance, when a bay of the third storey of transverse frame analysed here is considered, it follows that (see Figs.6.2, 6.11 and 6.13)

$$\Delta = \frac{1}{0.2}\,\frac{2.80^2}{24}\,\frac{1}{2.7\text{x}10^2}\left(\frac{86+93}{10.8} + \frac{272+275}{21.09} + \frac{6.0}{2.8}\text{ x }\frac{150+351+150+352}{20.76}\right) = 0.9\text{ cm} < \frac{280}{200} = 1.4\text{ cm}$$

It is worth mentioning that the limit imposed on storey drift is decisive in proportioning the concrete cross-sections of beams and columns of most seismic-resistant RC frames in Romania.

6.4. PROPORTIONING

6.4.1. Beam Requirements

Required Bending Strength

The maximum bending moments which every beam cross-section must resist are determined from the envelope of bending moment diagrams induced by the "fundamental" and "special" load combinations. For example, the envelope of bending moment diagrams of the 4-th floor of a interior transverse frame is depicted in Fig.6.14. The numerical example which follows refers to this very beam.

Longitudinal Reinforcement

Longitudinal reinforcement of any RC member with flexure is designed in accordance with the following relationship

$$M_{max} \leq M_u \tag{6.17}$$

where M_{max} is the required bending strength and M_u is the design bending strength. The latter strength is determined in accordance with the ultimate stress distribution in Fig. 6.15 where R_c and R_a are the design values of compressive strength of concrete and, respectively, yield strength of reinforcement (see Tables 6.2 and 6.3). Design tables [6.4] are available to aid the manual computation of M_u. The materials used herein are B 250 concrete, PC 52 steel in longitudinal bars and OB 37 steel in stirrups.

End reinforcements are designed as tensile reinforcement for M^+_{max} and M^-_{max} at the column face.

a. The area of bottom longitudinal reinforcement results as

$$A_a^{inf} = \frac{M^+_{max}}{R_a(h_o-a')} = \frac{16,570}{29x(66.5-3.5)} = 9.1 \text{ cm}^2 \tag{6.18}$$

on account that $A_a^{inf} < A_a^{sup}$ and consequently $x < 2a'$ in Fig.6.15. Three bars of 20 mm diameter (3 Ø 20) are chosen so that the effective surface area is $A^{inf}_{a,ef} = 9.42 \text{ cm}^2 > 9.1$ and the ultimate resisting moment is

$$M_u^+ = A^{inf}_{a,ef} \, R_a \, (h_o-a') = 172 \text{ kNm} \tag{6.19}$$

b. The surface area of top reinforcement results from the following relationships (see Fig.6.15 with $A_a = A_a^{sup}$ and $A'_a = A^{inf}_{a,ef}$):

$$B = \bar{\mu} \frac{R_a}{R_c} (1-0.5 \; \bar{\mu} \frac{R_a}{R_c}) = \frac{M^-_{max} - M^+_u}{bh_o^2 R_c} \tag{6.20}$$

and

$$A_a^{sup} = A^{inf}_{a,ef} + \bar{\mu} \, bh_o \tag{6.21}$$

when $x \geqslant 2a'$ and

$$A_a^{sup} = \frac{M^-_{max}}{R_a(h_o-a')} , \tag{6.22}$$

which is similar to Eq. (6.18), when $x < 2a'$

M^-_{max} = 312 kNm yields x < 2a' and thus A_a^{sup} = 17 cm^2 on account of Eq.(6.22). Four bars of 25 mm diameter (4 Ø 25) are chosen as top longitudinal reinforcement so that the effective surface area is $A^{sup}_{a,ef}$ = 19.63 cm^2 while the ultimate resisting moment is

$$M_u^- = M_u^+ + \bar{\mu}_{ef} b h_o^2 R_a (1-0.5\ \bar{\mu}_{ef} \frac{R_a}{R_c}) = 358 \text{ kNm} \qquad (6.23)$$

on account of Eq.(6.20) and of

$$\bar{\mu}_{ef} = \frac{A^{sup}_{a,ef} - A^{inf}_{a,ef}}{bh_o} = \frac{19.63-9.42}{35x66.5} = 0.00438$$

At the same time, the above values of effective surface area of longitudinal reinforcement at the beam ends comply with the following additional requirements for potential plastic zones of seismic-resistant flexural members [6.10], [6.5]:

$$A^{inf}_{a,ef} > 0.4\ A^{sup}_{a,ef} \qquad (6.24)$$

$$A^{sup}_{a,ef} > 0.4\% bh_o \qquad (6.25)$$

$$\frac{x}{h_o} = \bar{\mu}_{ef} \frac{R_a}{R_c} = 0.11 < 0.25 \qquad (6.26)$$

Mid-span reinforcement results from Eq.(6.17) in which M_{max} is the largest value of the positive bending moment in the middle zone of the span length while M_u is determined with due allowance to the T-shape of the cross-section. The effective flange width (b_p in Fig.6.15) is

$$b_p = b + 12\ h_p = 2.03 \text{ m} \simeq 2.00 \text{ m} \qquad (6.27)$$

while the necessary area, A_a, of bottom reinforcement is determined from the equilibrium equations (see Fig.6.15,c with x < h_p)

$$A_a R_a = b_p\ x\ R_c \qquad (6.28)$$

$$M_{max} = b_p x R_c (h_o - 0.5x)$$

M_{max} = 90.5 kNm in Fig.6.14, for instance, yields x=0.59 cm and A_a = 4.71 cm^2.

Since the maximum positive bending moment occurs at the beam end, the bottom 3 Ø 20 bars ($A^{inf}_{a,ef}$) are to be continuous through beam length. On this account, the positive plastic hinge will definitely occur at the end zones of beam. If the envelope of positive bending moments were as in Fig.6.16, the bottom mid-span would require additional reinforcement in order to conduct the occurrence of pozitive plastic hinge to the end zones of beam.

Transverse Reinforcement

Stirrups should ensure the shear transfer across the horizontal construction joint between the cast-in-situ and the precast parts of beam as well as prevent shear failure along any inclined crack.

Interface shear is analysed by means of the model depicted in Fig.6.17 where $\bar{L}$ is the tensioned length of the top longitudinal reinforcement, $A^{sup}_{a,ef}$. Since the compressive concrete struts are inclined about 45 deg.to the beam axis and the variation along the beam axis of tensile force of top longitudinal reinforcement is approximately linear, the force T_e which stresses a stirrup is constant along $\bar{L}$ and has the following ultimate value

$$T_e = \frac{A^{sup}_{a,ef} R_a}{\bar{L} + a_e} a_e \qquad (6.29)$$

The total surface area (e.g. $2A_e$ for a two-leg stirrup) and the spacing, a_e, of transverse reinforcement are determined from the requirement that the design strength should be not less than the required strength T_e, i.e.

$$2A_e R_{ae} \geqslant T_e \qquad (6.30)$$

where R_{ae} is the design yield strength of the steel used for stirrup bars (see Table 6.3).
Eqs.(6.29) and (6.30) with $A^{sup}_{a,ef} = 19.63\ cm^2$, $R_a = 290$ MPa, $\bar{L} = 2.55$ m, a_e = o.15 m and R_{ae} = 210 MPa yield

$$A_e = \frac{19.63 \times 290}{2 \times 210 \times 18} = 0.753\ cm^2 \qquad (6.31)$$

so that bars of 10 mm diameter can be used ($A_{e,ef} = 0.785\ cm^2$).

Shear strength of a beam is analysed in accordance with the model in Fig.6.18 and with the requirement

$$Q_{max} < Q_u = Q_b + Q_e \qquad (6.32)$$

where

Q_{max} is the required shear strength
Q_u is the design shear strength for a diagonal crack having the length s_i along the beam axis
Q_b is the shear strength provided by concrete and assumed to decrease linearly with the increase of s_i
Q_e is the shear strength provided by stirrups and assumed to increase linearly with the increase of s_i

The sum $Q_b + Q_e$ depends therefore on the length s_i and there is diagonal crack which yields a minimum value of Q^i_u. This minimum value is given by the following formula

$$Q_{eb} = \bar{Q}_{eb} b h_o R_t \qquad (6.33)$$

where R_t is the design tensile strength of concrete (see Table 6.2) and $\bar{Q}_{eb}$ is provided by design-aid tables [6.4], dependent on the qualities of concrete and transverse reinforcement, on the ratio μ of tensile reinforcement, i.e.

$$\mu = \frac{A^{sup}_{a,ef}}{bh_o} , \tag{6.34}$$

and the ratio μ_e of vertical shear reinforcement, i.e.

$$\mu_e = \frac{2A_e}{b_{ae}} \tag{6.35}$$

For the stirrups resulted from the interface shear analysis (i.e. Ø 10 mm/0.15 m of OB 37 steel) and for $A^{sup}_{a,ef}$ = 19.63 cm^2, b=0.35 m, h_o= 0.665 m and B 250 concrete, the previous relationships yield μ_e= 0.3%, μ = 0.84% so that $\overline{Q}_{eb}$=1.48. Thus Eq.(6.33) yields

$$Q_{eb} = 1.48 \times 35 \times 66.5 \times 0.09 = 310 \text{ kN} \tag{6.36}$$

Q_{max} is determined with the following relationship

$$Q_{max} = \frac{M^-_u + M^+_u}{L} + 0.5\ (G + 0.75\ P) \tag{6.37}$$

where (see Fig.6.19) M^+_u and M^-_u are computed according to Eqs. (6.19) and (6.23) while G and P are the total specified values of dead and live loads acting on the span length L. For the situation analyzed here, M^-_u = 358 kNm, M^+_u = 172 kNm, L = 5.35 m, G=162.9 kN, P=26.5 kN and therefore

$$Q_{max} = 190.5 \text{ kN} < 310 \text{ kN} \tag{6.38}$$

It is also required that [6.10]

$$\frac{Q_{max}}{bh_oR_t} = \frac{190.5}{35 \times 66.5 \times 0.09} = 0.91 < 2.0 \tag{6.39}$$

that μ_e = 0.3% > 0.2% and that a_e = 0.15 m < 0.20 m

6.4.2. Column Requirements

The design of the internal central columns is detailed herein. The third storey column is considered. Strength is dictated by the "special" load combination (see Fig.6.11 a) since no bending moments are induced by the "fundamental" load combination (see Fig.6.10). When the seismic action is assumed only in the transverse direction of structure with the torsional effect of seismic force like in Fig.6.9, the central column is subjected to compression and uniaxial bending in Fig.6.20.

Longitudinal Reinforcement

The maximum bending moment at beam face (M_{max} = 206.62 kNm in Fig.6.20)is considered for the required strength. The design strength is determined in accordance with the ultimate stress distribution in Fig.6.23, where R_c and R_a are the design strengths of concrete and steel, respectively (see Tables 6.2 and 6.3). On account of the requirement that maximum bending moment shall be not greater than the ultimate resisting moment while the axial force is constant as well as on account of the symmetry of reinforcement (A_a = A'_a), the equilibrium of stress distribution in Fig.6.21 yields

$$x = \frac{N}{bR_c} \tag{6.40}$$

$$A_a = A'_a > \frac{N.e-bxR_c(h_o-0.5x)}{R_a(h_o-a')}$$

when

$$2a' \leqslant x \leqslant 0.6\ h_o \tag{6.41}$$

where

$$e = \eta\ (\frac{M_{max}}{N} + e_{oa}) + 0.5(h_o-a') \tag{6.42}$$

$$e_{oa} = \min(20\ \text{mm},\ h/30) \tag{6.43}$$

$$\eta = \frac{1}{1 - \frac{N}{N_{cr}}} \tag{6.44}$$

$$N_{cr} = \frac{E_c bh^3}{4\ l_f^2} \tag{6.45}$$

and

$$l_f \simeq H \tag{6.46}$$

When M_{max} = 206.62 kNm, N=2,508 kNm, H=2.80 m, b=0.60 m and

h=0.75 m (see Fig.6.20) and when concrete is β 250 (see Table 6.2) and steel is PC 52 (see Table 6.13), the previous relationships yield

$$N_{cr} = \frac{2.7 \times 10^6 \times 60 \times 75^3}{4 \times 2.80^2} = 217.9 \times 10^6 N$$

$$\eta = \frac{1}{1 - \frac{2,508}{217,900}} = 1.011 < 1.2 \tag{6.47}$$

$$e = 1.011\ (\frac{206.62}{2,508} + 0.025)+0.335=0.4435\ \text{m}$$

$$x = \frac{2,508 x 10^3}{600 \times 10} = 418\ \text{mm} < 0.6\ h_o = 426\ \text{mm} \tag{6.48}$$

$$A_a = A'_a = \frac{2,508 \times 10^3\ \ (44.35-71+0.5\ 41,8)}{2.9 \times 10^4 \times (71-4)} < 0 \tag{6.49}$$

As the necessary reinforcement results negative, the gross area of column cross-section could be reduced from the strength viewpoint. However, since the 0.60 x 0.75 sqm surface area of column cross-section can not be reduced without violationg the ductility requirement (6.1), the effective longitudinal reinforcement follows the following limitations:
- the sum of reinforcement ratios shall be

$$\mu + \mu' = \frac{A_a + A'_a}{bh_o} > 0.5\% \tag{6.50}$$

- the bar diameter shall be not less than 14 mm
- the spacing between two succesive bars along the cross-section perimeter shall be not greater than 0.25 m.

These requirements yield the longitudinal reinforcement depicted in Fig.6.26. Thus the ultimate bending moment of the column cross-section when N = 2,508 kN follows from Eqs.(6.40) ... (6.46) as

$$M_u = N.e_o = 2{,}508 \text{ kN} \times 0.2018 \text{ m} = 506.1 \text{ kNm} \quad (6.51)$$

Transverse Reinforcement

The ties in the middle part of the column length $\bar{H}$ in Fig.6.22 are determined in accordance with criterion (6.32) where the design shear strength Q_u is evaluated in the same manner as in Eq.(6.33) while the required shear strength Q_{max} follows from

$$1.5Q \leq Q_{max} = \frac{2M_u}{\bar{H}} \leq 3\,Q \quad (6.52)$$

where Q is the column shear force resulting from the linear analysis of structure when subjected to "special" load combination (see Eq. (6.14) and Fig.6.20).

For the column analysed here, Q=195.35 kN, $\bar{H}$ = 2.10 m and M_u = 506.1 kNm so that relationship (6.52) yields

$$Q_{max} = 482 \text{ kN} \quad (6.53)$$

and Eqs.(6.32) and (6.33) yield

$$\bar{Q}_{eb} = \frac{Q_{max}}{bh_o R_t} = \frac{482{,}000}{600 \times 750 \times 0.9} = 1.25 < 2.0 \quad (6.54)$$

The above upper limit of $\bar{Q}_{eb}$ is required for columns in structures for which the seismicity index is not less than 7 [6.10].

On account of both $\bar{Q}_{eb}$= 1.25 and ratio of tensile reinforcement

$$\mu = \frac{A_a}{bh_o} = \frac{8.17}{60 \times 71} = 0.0019, \quad (6.55)$$

the design-aid table [6.4] gives a ratio of transverse reinforcement

$$\mu_e = \frac{A_{et}}{ba_e} = 0.0043 \quad (6.56)$$

By choosing the ties in Fig.6.26 (A_{et}=2 (0.785+0.503)=2.57 cm^2 and a_e = 0.1 m) the effective ratio of transverse reinforcement is

$$\mu_e = \frac{2.57}{60 \times 10} = 0.00429 \simeq 0.0043 \quad (6.57)$$

The limitations μ_e> 0.15% and a_e < 0.20 m are also satisfied. [6.10].

6.4.3. Requirements for Beam-Column Joints

The following provisions are required at the beam-column joint.

1. In order to control the occurence of plastic hinges

$$1.2(M_{uS} + M_{ud}) < M_{u,sup} + M_{u,inf} \quad (6.58)$$

where M_{uS} and M_{ud} are the ultimate resisting moments of beam ends at the left and right faces of joint while $M_{u,sup}$ and $M_{u,inf}$ are the ultimate resisting moments of column ends at the higher and lower faces of the joint when the axial force induced by the "special" combination of loads is considered. Both directions of bending moments in Fig.6.22 are considered.

For the central beam-column joint at fourth floor of the internal transverse frame, the above M_u - values are

$M_{uS} = M_u^+$ in Eq.(6.19)=172 kNm

$M_{ud} = M_u^-$ in Eq.(6.23)=358 kNm

$M_{u,sup} = M_u$ in Eq.(6.51)=506.1 kNm

$M_{u,inf} = 573.5$ kNm

so that requirement (6.58) is fulfilled by a large margin.

2. In order to provide a sufficient strength of joint to moment reversals

$$Q_{max} < 5A_b R_t \qquad (6.59)$$

where A_b is the horizontal effective area of joint while Q_{max} is the horizontal shear force to be resisted by the joint and is determined as in Fig.6.23, i.e.

$$Q_{max} = A_{a,ef}^{sup} R_a + A_{a,ef}^{inf} R_a \qquad (6.60)$$

For the central connection analysed here, i.e. at the fourth floor of the internal transverse frame, Eq.(6.60) yields

$$Q_{max} = (19.63 + 9.42) \times 29 = 842.45 \text{ kN}$$

and therefore

$$\frac{Q_{max}}{A_b} = \frac{842.45 \times 10^3}{600 \times 750} = 1.87 \text{ MPa} \approx 2.1 R_t \qquad (6.61)$$

where $R_t = 0.9$ MPa (see Table 6.2)

As far as the lateral reinforcement is concerned, the ties at the end zone of the bottom column shall be continued up through the beam-column joint [6.10]. When a joint is confined laterally by beams covering at least half its width, only outer ties are required in the direction of confinement. That is why inner ties are provided only in the longitudinal direction of the central joint in Fig.6.4.

6.5. SELECTED DETAILS

Details of form and reinforcement of the internal transverse beam of 4-th floor are presented in Figs. 6.24 and 6.25. Drawings in Fig.6.24 give details on the precast soffit beam while the details concerning the cast-in-situ part of the beam are presented in Fig.6.25.

The soffit beam incorporates the bottom longitudinal and the transverse reinforcement. It is worth mentioning that the stirrup diameter and spacing were decided here by the interface shear rather than by the shear strength along a diagonal crack. The bottom longitudinal bars project from the soffit beam and extend to the far face of the column (see Fig. 6.4). Additional extensions are provided to improve the anchorag of these bars during SM earthquakes.

The 25 mm depth shear keys provided at the top face of soffit beam increase the horizontal shear strength.

The shear keys at both end faces of soffit beam (see Fig.6.24) are provided in compliance with the requirement

$$1.5\ A_{bt}R_t > Q_{max} \tag{6.62}$$

where Q_{max} and R_t have the same meaning as in Eqs.(6.32) and (6.33) while A_{bt} is the vertical shear area of concrete. Indeed with A_{bt} = .35 x (2 x.16 + .14) = .161 m^2, the previous requirement becomes 217.35 kN > Q_{max} = 190.5 kN.

Prefabrication requires to reduce as much as possible the number of types of identical soffit beams. Four types of transverse soffit beams are used throughout the structure (see Table 6.4). They differ from each other function of span length, L, bottom longitudinal reinforcement, $A^{inf}_{a,ef}$, and/or stirrups.

Table 6.4. Types of precast soffit beams for transverse frames

Type	L (m)	$A^{inf}_{a,ef}$	Stirrups	Used at floor	Number of specimens
GT 1	5.25	3Ø20	Ø10/150	1 & 2	24
GT 2	5.325	3Ø20	Ø10/150	3 & 4	24
GT 3	5.425	3Ø18	Ø10/150	5 & 6	24
GT 4	5.425	3Ø16	Ø 8/150	7,8 & 9	36

Details of reinforcement of the central column designed in § 6.4.2 are presented in Fig.6.26. The tie spacing a_e = 0.10 m followed from the proportioning of transverse reinforcement. If the proportioning required a spacing a_e greater than 0.10 m, the tie spacing would have to be reduced to 0.10 m over the entire column length since the ratio between the column height 2.10 m and the cross-section height 0.75 m is not greater than 4 [6.10].

6.6. CONCLUDING REMARKS

The present design example is typical for the design philosophy in Romania when a space RC frame with precast members is used to resist seismic action on a multi-storey, non-industrial building. The design example outlines the main features of the design philosophy and the most important of them are reviewed and discussed here.

1. The structure lay out is designed as regular as possible.

2. In order to ensure that seismic response of structure resembles that of a monolithic space frame, only some of the structural members are precast. It is of common practice in Romania to precast the floor slabs and beams and to cast the columns in-situ.
The connections of precast floor panels are designed to ensure the non-deformability of horizontal diaphragm during seismic vibrations of structure whereas the floor slab is not provided with full continuity over the supporting beam when subjected

to gravity loads. Care is taken to distribute the lateral stiffness of subsystems so that the work of horizontal diaphragm over large span length is avoided.
The connection between precast soffit beam and precast floor panels is designed to ensure the flexural response of a monolithic beam.
The beam-column connections are designed to resemble a monolithic beam-column joint of a space frame.

3. On account of the above philosophy when designing connections, the seismic-resistant precast structure is designed as if it were monolithical.

4. The structure response to gravity load is determined by means of the elastic linear analysis. Plastic analysis is used solely with floor slabs.

5. The seismic response of structure may be analyzed by the folowing spproaches:
a. Equivalent lateral force procedure (i.e. static action and elastic response).
b. Elastic modal analysis procedure (i.e. dynamic action and elastic response).
c. Time-dependent, inelastic analysis procedure (i.e. dynamic action and inelastic response).

Among these approaces only the first is fit for manual computation.

Computer programs are currently available for all three approaches. With regard to the third approach it is worth mentioning that it can be currently performed in Romania only when the lateral vibration of structure is one-directional and it analyzes a structure already proportioned by one of the first two approaches. Nonetheless, the third approach is the only one capable of providing valuable information on the ductility demands. To achieve reliability, the use of many ground accelerograms is required. In view of this and of the present limits of the avaliable computer program, the third approach is mainly used in Romania for research purposes (see [6.1] and [6.7]).

The structural analysis within the present design example has been carried out by means of the first approach. The design example No.7 of the present volume (a 5 storey structure with precast floor and wall panels) uses the second approach.

6. The strength of RC structural members is designed according to limit state method. The basic concept is that required strength (determined by analysing the elastic response of structure) shall not exceed the design strength along critical cracks (determined by analysing the equilibrium of ultimate stress distribution).

The overall value of safety coefficient is divided among three factors. Thus:
- the uncertainty related to actual load amplitude is covered by the <u>load factor</u> (e.g. see Table 6.1 and § 6.3.3.);

- the uncertainty related to actual material strengths is covered by using the design values of concrete strength and steel yield strength (see Tables 6.2 and 6.3);
- a capacity reduction factor affects the design strength whenever additional safety provisions against failure must be considered (e.g. see the differences in Table 6.2 between the R_c values for beams and columns).

7. In addition to strength, ductility provisions are required to enable RC structure to deform well into the postelastic range and hence to survive SM earthquakes by absorbing and dissipating the energy induced by seismic vibrations.

The above design philosophy has proved correct during the last SM earthquake in Romania and it is in line with the recent world-wide knowledge on aseismic design. Nevertheless there are still many questions to answer before seismic - resistant RC frames are designed safely and without unnecessarily increasing cost.

Some of these questions are reminded here:
(I) are the present ductility provisions well correlated with the actual ductility demands throughout the RC structure and, indeed, with the strength level provided by the code seismic forces? [6.3].
(II) what is in terms of strength and deformability the actual response to reversed loading of those RC structural members which are decisively affected by shear or combined stresses (e.g. beams, joints, short columns)? For instance, requirement (6.1) aims to provide RC column with flexural ductility but ends quite seldom in bringing the column in the short range to avoid. (see e.g. Fig.6.20).
(III) what is the postelastic response to reversed forces of in-situ connections?
(IV) to what extent the in-filled masonry walls affect the seismic response of structure?

To find answers to these questions is the primary aim of the research in progress in Romania.

R E F E R E N C E S[1)]

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1) The present list is restricted to references published in Romania which are legal background to or provide additional information on aspects arised by the design example.

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[6.9] *** "The Romanian Code of Practice for the Structural Use of Concrete - STAS 10107/0-76" (in Romanian), 104 pp, Bucharest, 1976.

[6.10] *** "The Romanian Aseismic Code of Practice - P100/81" 63 pp, Bucharest, 1981.

NOTATION

Used		CEB
A_a	- surface area of tensile longitudinal reinforcement	A_s
A'_a	- surface area of compressive longitudinal reinforcement	A'_s
A_b	- surface area of concrete cross-section	A_c
A_e	- surface area of a leg of transverse reinforcement	-
A_{et}	- total surface area of a stirrup or tie	A_{st}
G	- total gravity load	G
G_j	- the component of G at j-th level	-
$H, \bar{H}$	- heights of column (see Fig.6.20)	L_c
I	- moment of inertia	I
L	- clear span length of beam	L_b
M	- bending moment (M^+ tensions the bottom face of beam, while M^- tensions the top face of beam)	M
M_{max}	- required bending strength	M_{act}
M_u	- ultimate bending moment (design bending strength)	M_u
N	- axial compressive force	N

P	- total live load	Q
Q	- shear force	V
Q_{max}	- required shear strength	V_{act}
Q_u	- design shear strength	V_u
Q_b	- component of Q_u provided by concrete	V_c
Q_e	- component of Q_u provided by transverse reinforcement	V_{st}
Q_{eb}	- the minimum value of sum $Q_b + Q_e$	-
R_a	- design yield strength of longitudinal reinforcement	f_s
R_{ae}	- design yield strength of transverse reinforcement	f_{st}
R_c	- design compressive strength of concrete	f_c
S	- total base shear force	E
S_j	- base shear force of j-th frame	-
S_{ij}	- component of S_j at i-th floor	-
T	- tensile force	N_{tn}
a'	- effective depth of compressive longitudinal reinforcement	d'
a_e	- spacing of transverse reinforcement	s_t
b	- width of beam web	b_w
b_p	- width of beam flange	b_f
c_s	- seismic coefficient (= $k_s\beta\varepsilon\psi$, see Eq.(6.3)).	k_{eq}
h	- height of concrete cross-section	h
h_o	- effective depth of tensile longitudinal reinforcement	d
x	- depth of compressive zone of cross-section	x
$\emptyset$	- diameter of reinforcing bar (e.g. 3$\emptyset$20 means 3 bars of 20 mm diameter and $\emptyset$ 10/15 means bars of 10 mm diameter with a 15cm spacing)	$\emptyset$
μ	- ratio of tensile longitudinal reinforcement	$\rho/100$
μ'	- ratio of compressive longitudinal reinforcement	$\rho'/100$
μ_e	- ratio of transverse reinforcement	$\rho_t/100$

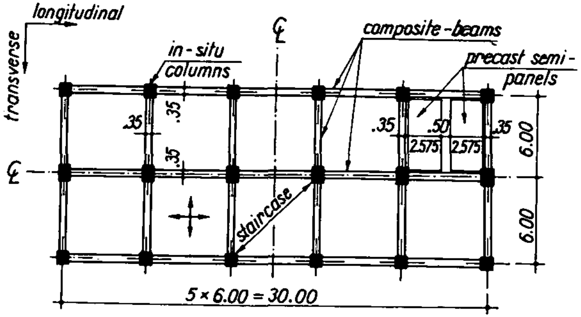


Fig.6.1-Plan configuration of structure

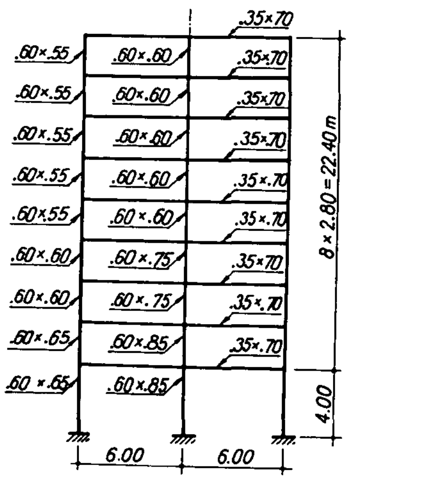


Fig. 6.2.-Vertical configuration of structure in transverse direction

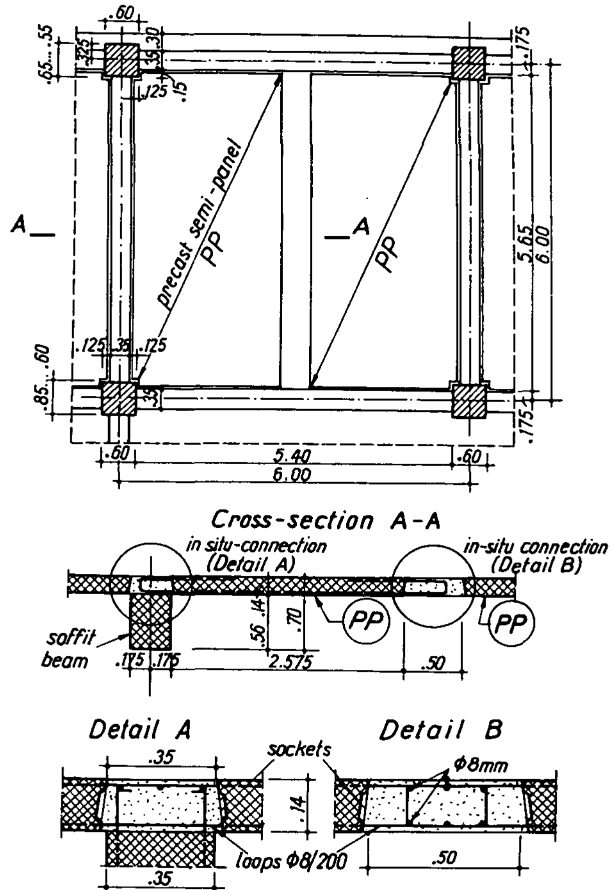


Fig. 6.3

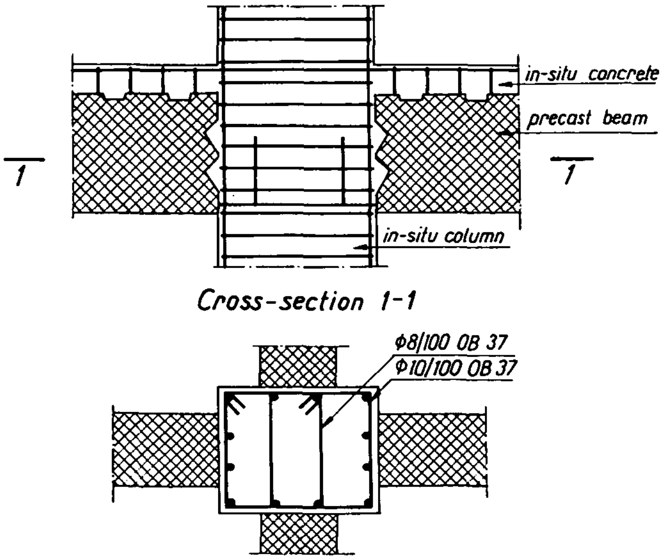


Fig. 6.4- Central beam-column connection

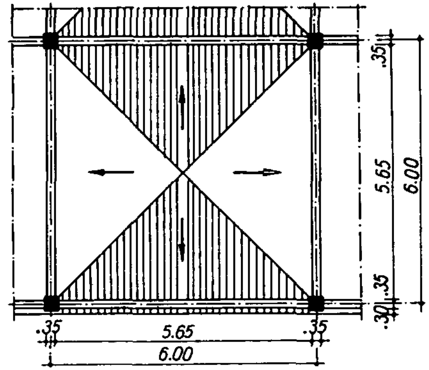


Fig. 6.5

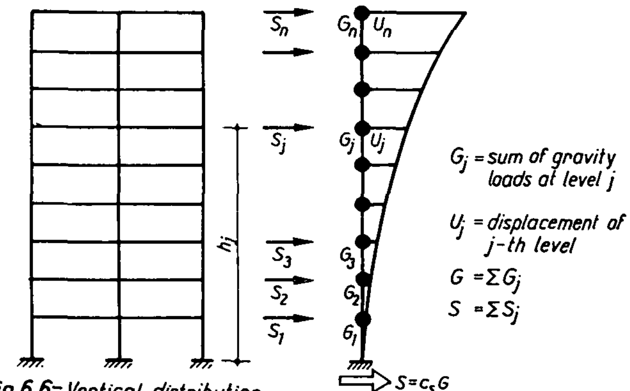


Fig. 6.6- Vertical distribution of shear base force

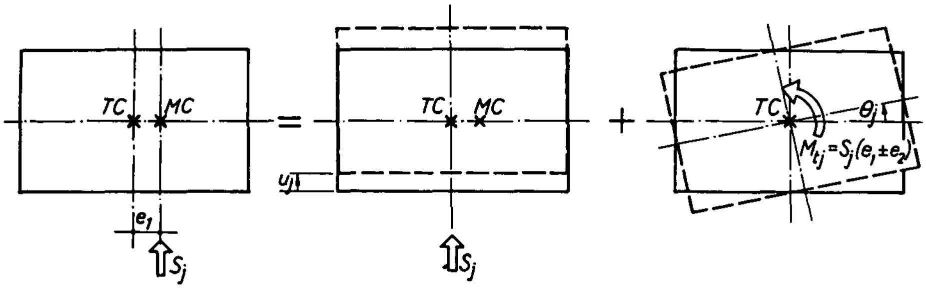


Fig.6.7 - Global effect of seismic force S_j

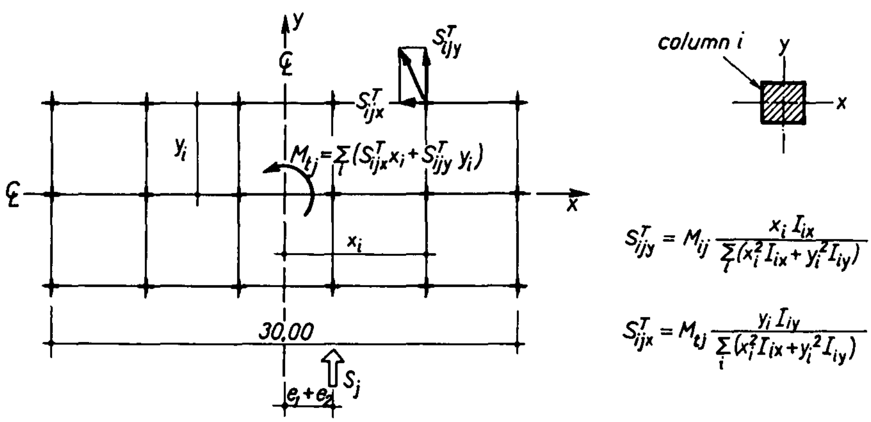


Fig.6.8 - Torsional effect of seismic force S_j

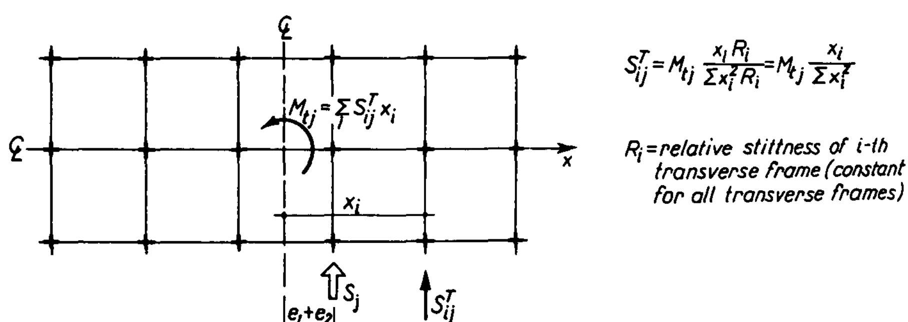


Fig. 6.9

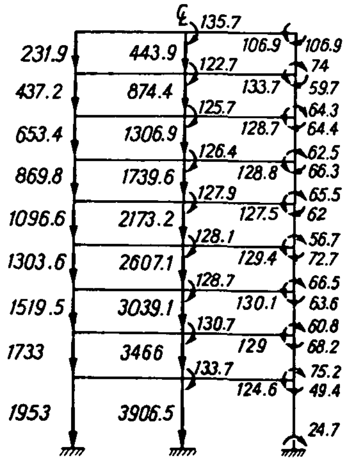


Fig. 6.10 - Axial forces (kN) and end bending moments (kNm) induced by fundamental load combination

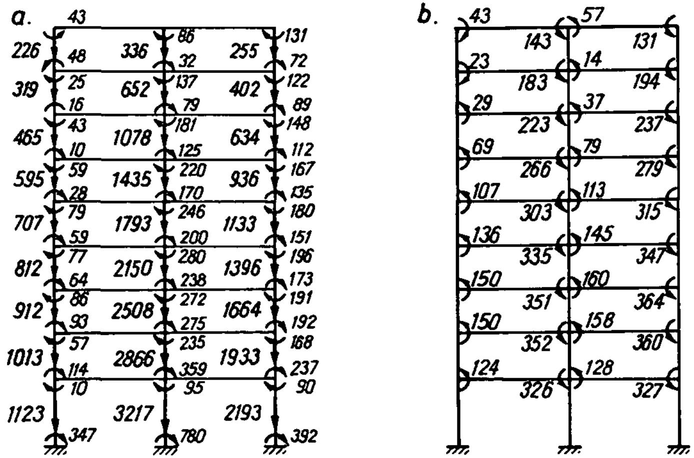


Fig. 6.11 - Axial forces (kN) and bending moments (kNm) induced by special load combination

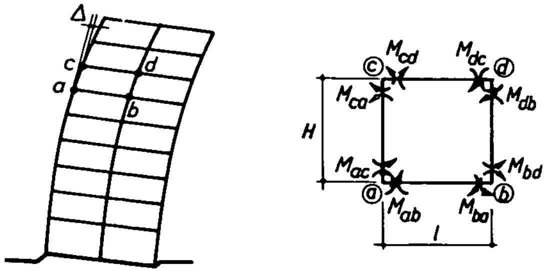


Fig. 6.12

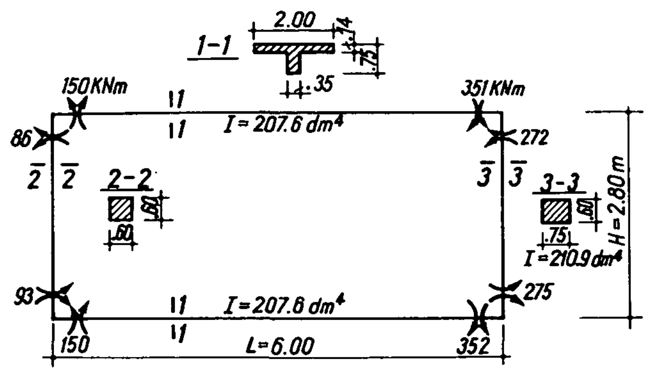


Fig. 6.13

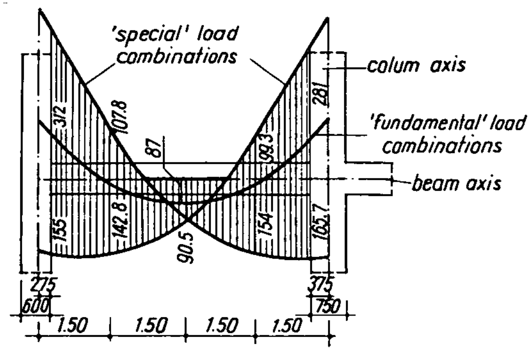


Fig.6.14-Envelope of bending moment diagrams (kNm) of transverse beam at fourth floor

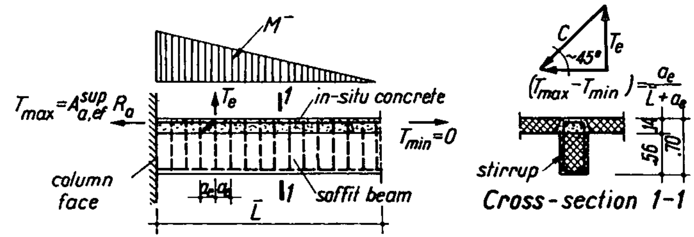


Fig.6.17-Interface shear between precast and in-situ concrete of beam

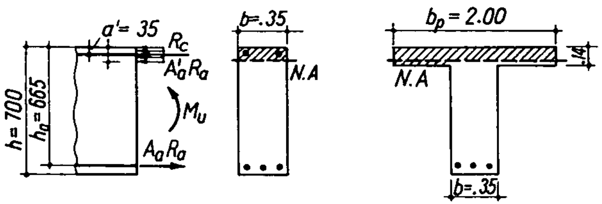


Fig.6.15-Ultimate stress distribution on a RC cross-section subjected to bending moment (when $2a' \leq x \leq 0.6\,h_o$)

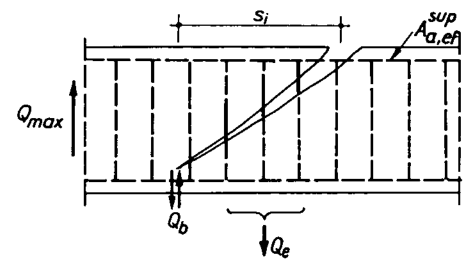


Fig.6.18-Failure mechanism along a diagonal crack

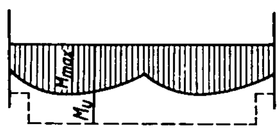
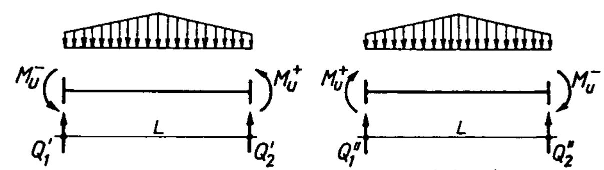


Fig. 6.16



$Q_{1max} = max(Q_1', Q_1''); Q_{2max} = max(Q_2', Q_2'')$

Fig.6.19-Evaluation of required shear strength Q_{max}

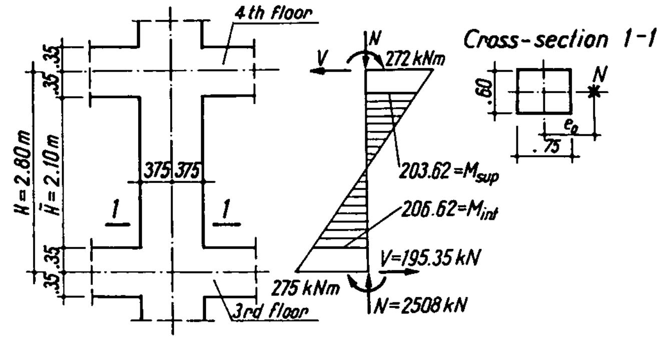


Fig.6.20-Central column

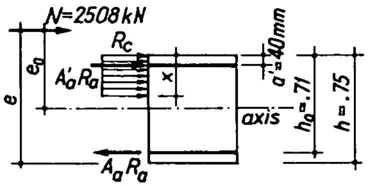


Fig.6.21-Ultimate stress distribution on a RC cross-section subjected to combined compression and uniaxial bending.

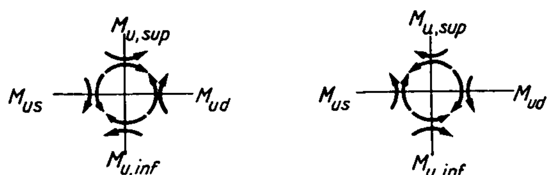


Fig.6.22

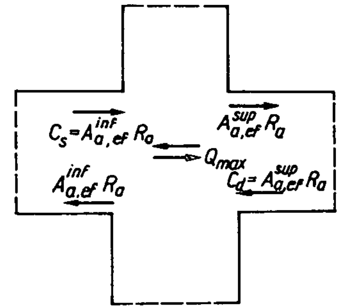


Fig.6.23

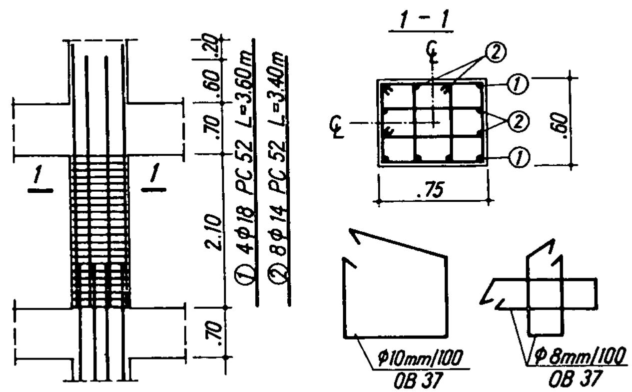


Fig.6.26

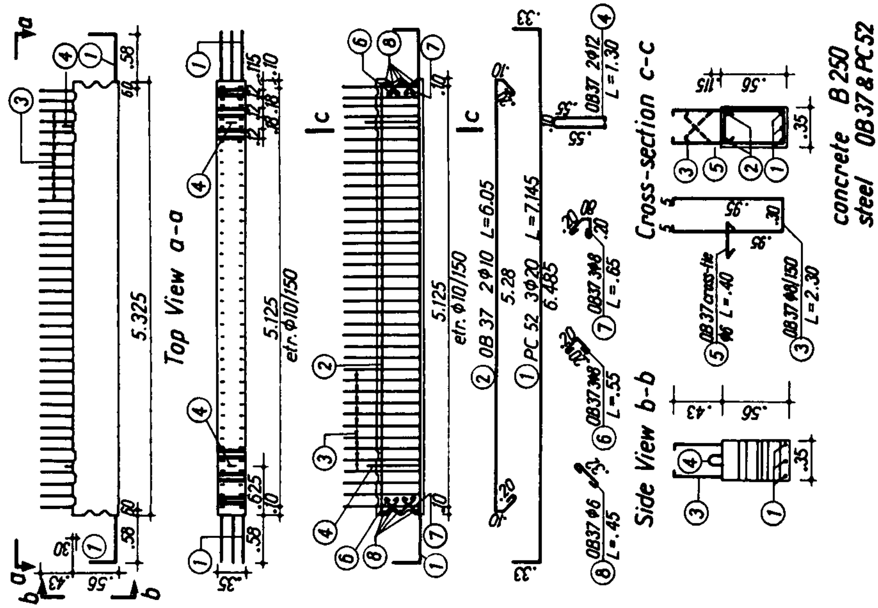
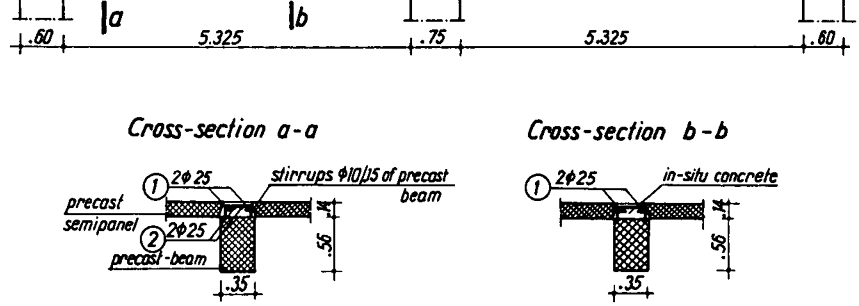


Fig.6.24 - Precast soffit beam GT 2



concrete B 300
steel PC 52

Fig. 6.25 - In-situ topping of GT 2 at the 4-th floor

7. REPRESENTATIVE EXAMPLE OF ROMANIA

5-storey structure with precast large panels

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7.1. SCOPE

The fundamentals of the aseismic design philosophy of a 5-storey apartment house built with large precast RC wall and floor panels is presented. Both structure lay out (see Fig.7.1) and design requirements are typical for the current practice in Romania. Normal soil conditions and a zone with seismicity index equal to 8 (see [7.2] or [7.3]) are considered. The detailing of large panels and connections as well as the analysis and proportioning of structure comply with the provisions of Refs. [7.1] and [7.2].

7.2. DESCRIPTION OF STRUCTURAL SYSTEM

The lay out of load bearing walls of structure is depicted in Fig.7.1. The cross walls are spaced not farther than 3.60 m and stretch across the entire width of building. All but two of cross walls are weakened by door openings. Three long walls are provided over the entire length of building. Both plan and vertical conformations of structure are regular.

Building has an overall basement with cast-in-situ RC walls beneath every structural wall in Fig.7.1. In view of the low lateral deformability of the in-situ basement (walls have 0.20 m thickness, the basement height is only 1.80 m and the door openings of basement walls are not provided on the same vertical as the apartment door openings), it is a common practice in Romania to assume that the upper 5-storey structure is practically embedded at the first floor level.

Vertical in-situ connections of precast wall panels are provided at every intersection of cross and long walls.

The floors are made of precast slab panels supported by precast wall panels on three or four sides. Horizontal in-situ connections are provided along every joint between the two superposed wall panels and the two adjacent slab panels.

Both vertical and horizontal connections are designed to fully transmit shear forces of vertical shear-walls as well as of horizontal diaphragms. Thus, after in-situ concrete of connections has gained sufficient strength, the large panel structure resembles in both performance and strength a monolothic space assembley of interconnected shear-walls and horizontal diaphragms.

<u>Precast Large Panels</u>. The external wall panel has a sandwich 300 mm depth made of the following layers:
- 100 mm of ordinary structural concrete at the internal face of wall to provide the strength;
- 150 mm of celular concrete (with a density of about 6 kN/m^3) to provide the heat insulation;
- 50 mm of ordinary structural concrete to protect the intermediate layer of celular concrete.

Ribs of ordinary concrete are provided in both directions of wall panel to link the two lateral layers.

The internal wall panel has a 140 mm depth of ordinary structural concrete. The floor panel has a 130 mm depth of ordinary structural concrete. The ordinary concrete has a density of about 25 kN/m^3 and is B 250 (see Table 6.2 in [7.4])

Each precast wall panel has 0.30 m spaced sockets or nibs evenly distributed throughout its perimeter. Reinforcing bars project in both directions of wall panel. Horizontal bars are evenly distributed over the height of wall panel. Vertical projecting bars are provided near both vertical connections and door opening. The projecting bars are placed in the median plan of panel. They are continuous over the appropriate dimension of panel and made of PC 60 steel (see Table 6.3 in [7.4]). An overall welded smooth wire fabric is also provided as shrinkage reinforcement of precast wall panel.

Connections. After welding the appropriate bars projecting from precast large panels and placing the tie bars, the connection is concreted in-situ. The concrete is B 300. Site formwork is required by the vertical connection and subsequently the in-situ concreting can be inspected after the form has been removed (see Figs.7.2 a and b). The in-situ concrete of horizontal connection is cast after the upper wall panel has been placed and levelled (see Figs.7.2 c and d).

Vertical shear between wall panels is transmitted by means of both "truss" effect within the vertical connection (compression of concrete struts and tension of horizontal projecting bars) and shear strength of the RC cross-section of horizontal connection. As for the horizontal connection a shear friction is taken into consideration in addition to the shear mechanism just described in conjunction with the vertical connection.

Vertical tensile stresses induced by lateral loading of the shear-wall are transmitted across the horizontal connection by means of the welded vertical reinforcement (projecting from wall panels and provided as tie bars of vertical connections).

7.3. STRUCTURAL ANALYSIS

The vertical seismic-resistant structure is made of cross and long cantilever shear-walls coupled by the beams above door openings as well as above and beneath window openings.

The nominal cross-sections of coupling beams are denoted in Fig.7.3 by A and B (when above door openings) and by C (when above and beneath two superposed window openings). Thus the nominal cross-section of a coupling beam is determined as if the relative shear displacement (between shear-wall and floor in Details A, B and C as well as between top wall panel and bottom wall panel in Detail C) were "free" to occur.

The nominal cross-section of cross and long shear-walls when subjected to lateral seismic forces are denoted in Fig.7.3 a and b, respectively. The effective lange width of the cross-section of a shear-wall is not greater than [7.2]:
- 1/5 of the total height of building
- the depth of the shear-wall cross-section

- 5.0 m

The flexural stiffenesses of cross-sections of shear-walls and coupling beams are equal to the moments of inertia of nominal cross-sections defined above multiplied by the modulus of elasticity, E . For a shear-wall, E is assumed equal to the concrete modulus of elasticity E_c (e.g. for a B 250 concrete E_c = 27,000 MPa - see Table 6.2 of [7.4]). For a coupling beam E is assumed equal both to 0.6 E_c and to 0.15 E_c in order to account for degrading stiffness during a SM earthquake. Thus the effects of seismic action on structure are analysed for two different stiffnesses of coupling beam. The value 0.6 E_c provides the internal forces to proportion coupling beams whereas the value of 0.15 E_c provides the internal forces to proportion shear-walls.

The seismic response of structure is analysed herein by means of the modal analysis procedure and of assumption that horizontal diaphragms at floor levels are non-deformable. The modal basic shears are determined according to Eq.(6.3) of [7.4] with the following remarks:
- the reduction factor Ψ is equal to 0.3 [7.2];
- the equivalence factors ε are provided by modal analysis;
- the gravity load is 12 kN/m^2 of floor.

The analysis is carried out by means of a computer program similar to ETABS. The seismic induced internal forces of both shear-walls and coupling beams are determined by taking the square root of the sum of squares of the first nine prevailing modal values. These internal forces are added to those induced by gravity loads in accordance with the "special" load combination (see Eq.(6.14) of [7.4]).

Gravity loads induce only axial compressive forces in shear walls. Axial forces are determined by multiplying the surface area of the nominal cross-section of shear-wall (see Fig.7.3) by the appropriate value of σ_g assumed evenly distributed over the gross surface area of shear-wall cross-section.

The design requirements associated with large panel systems in Romania are hereby presented in conjunction with the transverse and longitudinal shear-walls marked in Fig.7.3. The internal forces at the bottom end of these shear-walls are given in Table 7.1.

Table 7.1

Shear wall	Bending moment (kNm)	Shear force (kN)	Axial force (kN)
No.2 of DT1	3878	424	1279
No.3 of DL2	4193	395	2078

N.B. According to [7.2], the shear force in Table 7.1 is multiplied by 1.5 in order to obtain the required shear strength of shear-wall and its connections.

7.4. PROPORTIONING

7.4.1. Strength Requirements

The strength design of precast shear-wall is based upon the following requirements [7.1] :

(I) to prevent the shear-wall from buckling laterally, the vertical compressive stress, σ , induced by the combined bending moment and axial force shall comply with

$$\bar{c}\ \sigma < \phi\ R_c \tag{7.1}$$

where

$$\bar{c} = 1.2$$

R_c = design value of compressive strength of concrete cast in a shear-wall (herein R_c= 8.5 MPa and result from multiplying the 11.5 MPa compressive strength of B 250 concrete-see Table 6.2 of [7.4] - by a strength reduction factor equal to 0.75).

ϕ = capacity reduction factor which accounts for the lateral flexibility of shear-wall (see Fig.7.4a). The ϕ - value depends on the relative lateral eccentricity e_o/b and on the reduced slenderness $\bar{\lambda}$ (see Fig.7.5).

b = width of shear-wall web.

Provisions are given in [7.1] to locate vertical strips of shear-wall where requirement (7.1) is to be checked. The location of such a strip depends on the ratio H/ℓ of the wall panel and on whether the vertical sides of wall-panel are free or not to deform laterally (e.g. see Fig.7.4 a).

The lateral eccentricity, e_o, of the vertical compression measured from and perpendicular to the mid-plane of the shear-wall is determined by means of the following formula:

$$e_o = \sqrt{0.3(e_s^2 + e_i^2) + 0.4\ e_s e_i} + e_p + e_c \tag{7.2}$$

where

e_s, e_i = eccentricities of the vertical axial force at the top and bottom faces of large-wall panel;

e_p = 0.002 H and is an unavoidable, out-of-plane eccentricity of large wall panel;

e_c = eccentricity due to a seismic force acting perpendicularly to the large wall panel.

The eccentricities e_s and e_i are computed as

$$e_s(e_i) = e_1 + e_2 \pm 0.03\ b \tag{7.3}$$

where e_1 takes into consideration that two superposed wall panels may have not the mid-planes on the same vertical (e.g. see Fig.7.10 a) while e_2 accounts for the eccentricities of gravity loads sustained by the large wall panel (e.g. see Fig. 7.10 b).

The eccentricity e_c is given by the formula

$$e_c = \frac{M_s}{N} \tag{7.4}$$

where (see Fig.7.4 a and b)

$$N = \sigma\, b \times 1.0 \text{ m} \tag{7.5}$$

$$M_s = \frac{SH}{6} \tag{7.6}$$

$$S = c_s G \simeq 0.09\ G \tag{7.7}$$

while G, b and 1.00 m are, respectively, the weight, the depth and the width of the vertical strip associated to requirement (7.1).

The <u>reduced slenderness $\bar{\lambda}$</u> in Fig.7.5 is determined by the following formula

$$\bar{\lambda} = \frac{kH}{b\sqrt{\alpha}} \tag{7.8}$$

where

k depends on the ratio H/ℓ of the wall panel and on whether the vertical sides of wall panel are free or not to deform laterally (see Fig.7.6).

$$\alpha = \frac{E_c}{1.6R_c(1+1.2v)} \tag{7.9}$$

E_c and R_c have been defined above

v = the ratio between the long-term portion of σ and σ.

(II) <u>to prevent failure across diagonal crack A</u> in Fig.7.4 c the required shear strength of shear-wall must not be greater than the design shear strength, i.e.

$$\bar{c}\ Q < Q_{lim} \tag{7.10}$$

where

Q = 1.5 times the seismic shear force (see Table 7.1 and footnote);

Q_{lim} = design shear force across the diagonal crack and determined with the following formula

$$Q_{lim} = \frac{h}{H}(0.8A_aR_a + 0.3\ b\ HR_t) \tag{7.11}$$

where

h,b = height and web width of shear-wall cross-section;

H = 2.70 m, the storey height;

A_a = total area of horizontal reinforcement over the height of storey (the sum of horizontal bars projecting from the wall panel and of tie bars of horizontal connection);

R_t = 0.9 MPa, the design tensile strength of B 250 concrete (see Table 6.2 of [7.4]);

R_a = 340 MPa, the design yield strength of PC 60 steel (see Table 6.3 of [7.4]).

(III) <u>to prevent failure across crack B</u> in Fig.7.4 c the vertical shear force, L_e, must comply with

$$\bar{c}\ L_e < T_{max} \tag{7.12}$$

where T_{max} is the design shear force of vertical connection.

T_{max} is provided by concrete compressive struts, by shear across the concrete cross-section of horizontal connection and by dowell force of horizontal reinforcement. Both L_e and T_{max} are determined over the storey height, H, and are determined by following formulae

$$L_e = \tau\ b\ H \tag{7.13}$$

and $$T_{max} = n_v d_{sv} b_{sv} R_c + 0.6 R_t A' + 0.8\ R_a A_a \tag{7.14}$$

where b, h, H, A_a, R_a, R_c and R_t have the same meanings as above while

τ = shear stress induced by S at the location of vertical connection (see Fig.7.7);

d_{sv}, b_{sv} = depth and width of sockets on the vertical lateral face of precast wall panel (see cross-section 1-1 in Fig.7.7);

n_v = number of sockets over the height of a wall panel;

A' = area of concrete cross-section of horizontal connection (see cross-section 2-2 in Fig.7.7).

In addition to requirement (7.12), L_e must also comply with limitation

$$\bar{c}\ L_e < 1.5\ A_a R_a \tag{7.15}$$

(IV) <u>to prevent failure across crack C</u> in Fig.7.4 c the required shear strength of shear-wall must not exceed the design shear strength of horizontal connection. The design strength is provided by concrete compressive struts, by dowell force of some vertical reinforcing bars (i.e. those which are not used as tensile vertical reinforcement required by the combined bending moment and axial force on the shear-wall cross-section) and by shear friction.

The above requirement can be written as

$$\bar{c}\ L_o < L_{uh} \tag{7.16}$$

where L_o and L_{uh} are determined over the height h of the shear-wall cross-section by means of the following formulae

$$L_o = A_\tau b \tag{7.17}$$

and

$$L_{uh} = n_h d_{sh} b_{sh} R_c + 0.8 R_a A'_a \tag{7.18}$$

where h, R_c and R_a have the same meaning as sbove while A_τ = area of shear stress distribution depicted in Fig.7.7; only shear stresses τ greater than 0.35 times normal stresses σ are considered when computing A_τ to account for the shear friction along the compressed zone of shear-wall cross-section.

d_{sh}, b_{sh} = depth and width of sockets or nibs on a horizontal lateral face of precast wall panel (see cross-section 2-2 in Fig.7.7);

n_h = number of sockets or nibs along the height h;

$$A'_a = A_a - \bar{A}_a \tag{7.19}$$

A_a = total area of vertical reinforcement over the length h (the sum of vertical bars projecting from wall panels and of tie bars of vertical connections)

$$\bar{A}_a = \frac{\bar{c} b A_\sigma}{R_a} \tag{7.20}$$

[1)]

A_σ = area of tensile zone of normal stress distribution (see Fig.7.7).

---

1) [7.1] requires that $\bar{A}_a$ shall be so distributed over the depth of tensile zone as to match the triangular distribution of normal stresses. Eq.(7.20) is valid when the tensile cross-section is rectangular. $V_\sigma = \Sigma b A_\sigma$ should replace bA_σ when the cross-section is flanged.

It is apparent that the fulfilment of requirement (7.16) must be checked at both top and bottom faces of horizontal connection.

(V) <u>to prevent crushing of concrete in Fig.7.4 c</u>, the maximum compressive stress across a horizontal connection, σ'_{max}, shall comply with the following limitation

$$\bar{c}\,\sigma'_{max} \leqslant \xi R_c \tag{7.21}$$

where $\bar{c}$ and R_c have the same values as above while

$$\sigma'_{max} = \sigma_{max}\left(1 + \frac{6e_i}{b}\right) \tag{7.22}$$

takes into account the effect of lateral eccentricity e_i on the variation of σ_{max} over the width b (see cross-section 2-2 in Fig.7.7).

$$\xi = 0.7 \tag{7.23}$$

is a capacity reduction factor used when floor panels penetrate into the horizontal connection and the projecting bars are welded spliced (see Fig.7.20).

It is apparent that the fulfilment of above defined requirements (I)...(V) must be checked for both opposite signs of seismic-lateral force whenever the shear-wall cross-section is non-symmetric.

Examination of foregoing strength requirements gives rise to following conclusions:
- use of additional load factor 1.5 when designing the shear strength of shear-wall as well as of vertical and horizontal connections between wall panels aims to prevent the occurence of brittle failure mechanisms during SM earthquakes;
- the wall thickness b is controlled by requirements (7.1) and (7.21);
- the amount of horizontal reinforcement projecting from wall panels is controlled by requirements (7.10) and (7.12);
- the amount of vertical reinforcement projecting from wall panels is controlled by requirements (7.16) and (7.20).

It is equally worth adding that no specific requirement concerning ductility of shear wall structures with precast large panels is provided.

Numerical applications of above strength requirements are presented here in for shear wall No.2 of cross wall DT 1 (Fig.7.3 a) and shear wall No.3 of long wall DL 2 (Fig.7.3 b).

7.4.2. <u>Proportioning of external cross wall DT_1</u>

The first storey of the shear-wall No.2 is detailed in Fig.7.8. The normal and shear stress distribution at the bottom cross-section are determined from the internal forces in Table 7.1 (footnote included) and are depicted in Fig.7.9.

A. <u>STRENGTH TO COMBINED BENDING AND AXIAL LOAD</u> is provided by requirements (7.1), (7.21) and (7.20).

<u>Requirement (7.1)</u>. Eccentricities e_1 and e_2 in Eq.(7.3) are determined in Fig.7.10. Since

$$e_{1s} = 2.5 \text{ mm and } e_{2s} = 6.3 \text{ mm} \tag{7.24}$$

at the top face of first floor wall panel and, respectively,

$$e_{1i} = 0 \quad \text{and} \quad e_{2i} = 0 \qquad (7.25)$$

at the bottom face of the same wall panel

$$e_s = 2.5+6.3+0.03x100 = 11.8 \text{ mm} \qquad (7.26)$$

$$e_i = 0.03 \times 100 = 3 \text{ mm}$$

Taking into account the particularities of shear wall DT 1 and the provisions in Fig.7.4 a, the vertical strips to be considered by requirement (7.1) are A in Fig.7.9 a (σ=2.07MPa) and B in Fig.7.9 b (σ=3.33 MPa).

a. Vertical strip A has the weight

$$G=(0.10+0.05)x1.0x2.7x25+0.15x1.0x2.7x6=12.55 \text{ kN} \qquad (7.27)$$

and hence Eqs.(7.7) and (7.6) yield

$$S = 0.09 \times 12.55 = 1.13 \text{ kN} \qquad (7.28)$$

and, respectively,

$$M_s = \frac{1}{6} \times 1.13 \times 2.7 = 0.51 \text{ kNm} \qquad (7.29)$$

whereas Eq.(7.5) yields

$$N = 2.07 \times 10^3 \times 0.1 \times 1.0 = 2o7 \text{ kN} \qquad (7.30)$$

Eccentricity e_c follows from Eq.(7.4) as

$$e_c = \frac{0.51}{207} \times 1000 = 2.5 \text{ mm} \qquad (7.31)$$

so that the total eccentricity e_o in Eq.(7.2) results as

$$e_o = \sqrt{0.3(11.8^2+3^2)+0.4x11.8x3+0.002x2700}+2.5=15 \text{ mm} \quad (7.32)$$

while the total relative eccentricity is

$$\frac{e_o}{b} = \frac{15}{100} = 0.15 \qquad (7.33)$$

The reduced slenderness $\bar{\lambda}$ arises from Eq.(7.8) as

$$\bar{\lambda} = \frac{1.0 \times 2700}{230 \sqrt{1240}} = 0.335 \qquad (7.34)$$

on account that k=1 (see curve C of Fig.7.6 when the ratio H/ℓ of wall panel is 2.7/5.4=0.5), that b_{ech}=230 mm (the effect on slenderness of external 50 mm concrete layer is considered in here as the shear displacement between the two outside layers of external wall panel is fully restrained by the linking ribs) and that Eq.(7.9) yields

$$\alpha = \frac{27,000}{1.6x8.5(1+1.2x0.5)} = 1240 \qquad (7.35)$$

where

$$v = \frac{1.14 - 0.11}{2.07} = 0.5 \qquad (7.36)$$

in view of the fact that 0.11 MPa out of the gravity normal stress σ = 1.14 MPa is due to short-term loading.

With the values provided by Eqs.(7.33) and (7.34), the graph in Fig.7.5 yields

$$\phi = 0.53 \qquad (7.37)$$

so that the left and right hand sides of requirements (7.1) are

$$\bar{c}\,\sigma = 1.2 \times 2.07 = 2.48 \text{ MPa} \qquad (7.38)$$

and respectively,

$$\phi R_c = 0.53 \times 8.5 = 4.5 \text{ MPa} \qquad (7.39)$$

b. Vertical strip B has about the same ϕ - value as the previous strip so that the right-hand side of requirement (7.1) conserves the values provided by Eq.(7.39) while the left-hand side is

$$\bar{c}\ \sigma = 1.2 \times 3.33 = 4.5 \text{ MPa} \tag{7.40}$$

Requirement (7.21). The maximum value of normal stress distributions in Fig.7.9 is

$$\sigma_{max} = 3.88 \text{ MPa} \tag{7.41}$$

while the eccentricity e_i is - see Eqs.(7.26) -

$$e_i = 3 \text{ mm} \tag{7.42}$$

and therefore Eq.(7.22) yields

$$\sigma'_{max} = 3.88\ (1+ \frac{6x3}{100}) = 4.58 \text{ MPa} \tag{7.43}$$

while requirement (7.21) can be written as

$$1.2 \times 4.58 = 5.49 < 0.7 \times 8.5 = 5.95 \text{ MPa} \tag{7.44}$$

Requirement (7.20) Both normal stress distributions in Fig.7.9 must be considered when computing the surface area $\bar{A}_a$ of vertical reinforcement required by the combined effect of bending moment and axial force.

a. Distribution in Fig.7.9,a yields the total tensile force

$$bA_\sigma = 0.1\ \frac{2.195x1.47x10^3}{2} = 161.33 \text{ kN} \tag{7.45}$$

so that the reinforcement required at the left-hand side of shear wall (see Fig.7.8) and made of PC 60 steel shall have the surface area not less than

$$\bar{A}_a = \frac{1.2 \times 161.33}{340 \times 0.1} = 5.69 \text{ cm}^2 \tag{7.46}$$

It is apparent in Fig.7.8 that the reinforcement provided is made of 3 ϕ 16 and 1 ϕ 18 bars so that

$$\bar{A}_{a,ef} = 3 \times 2.01 + 1 \times 2.54 = 8.57 \text{ cm}^2 > 5.69 \text{ cm}^2 \tag{7.47}$$

b. Similarly, distribution in Fig.7.9,b yields

$$V_\sigma = \frac{1}{2}(1.57+1.5)x10^3x0.1x0.78+\frac{1}{2}x1.5x10^3x0.1x2.24=288.25\text{kN} \tag{7.48}$$

and the minimum amount of PC 60 reinforcement to provide at the right-hand side of shear wall (see Fig.7.8) is

$$\bar{A}_a = \frac{1.2x288.25}{340 \times 0.1} = 10.1 \text{ cm}^2 \tag{7.49}$$

The reinforcement provided is made of 3 ϕ 16 and 1 ϕ 22 bars and therefore

$$\bar{A}_{a,ef}=3x2.01+1x3.8=9.83 \text{ cm}^2 \approx 10.1 \text{ cm}^2 \tag{7.50}$$

B. SHEAR STRENGTH is designed in accordance with requirements (7.10), (7.12), (7.15) and (7.16).

Requirement (7.10). The factored value of shear force Q is (see Table 7.1).

$$Q = 1.5 \times 424 = 636 \text{ kN} \tag{7.51}$$

while the provided shear strength is (see Eq.(7.11) and Fig.7.8).

$$Q_{lim}=\frac{7.98}{2.70}(0.8x10.59x340x0.1+0.3x0.1x2.7x0.9x10^3)=1070\text{kN} \tag{7.52}$$

on account of the values of h,H,b,R_a and R_t as well as of the provided horizontal reinforcement made of 8 ϕ 12 projecting bars and 1 ϕ 14 tie bar, i.e.

$$A_a = 8 \times 1.13 + 1.54 = 10.59 \text{ cm}^2 \qquad (7.53)$$

Hence requirement (7.10) can be weitten as

$$1.2 \times 636 = 763.2 \text{ kN} < 1070 \text{ kN} \qquad (7.54)$$

Requirement (7.12). The vertical connection corresponding to point C in Fig.7.9 is considered so that the shear stress induced by factored Q is

$$\tau = 1.197 \text{ MPa} \qquad (7.55)$$

and Eq.(7.13) yields

$$L_e = 1.197 \times 10^3 \times 0.10 \times 2.70 = 323.19 \text{ kN} \qquad (7.56)$$

Since A_a= 10.59 cm^2 in Eq.(7.53) and since (see Fig.7.2a and c)

$$A' \simeq 200 \text{ cm}^2$$
$$d_{sv} = 30 \text{ mm}$$
$$b_{sv} = 60 \text{ mm}$$
$$n_v = 15$$

Eq.(7.14) yields

$$T_{max} = 15\text{x}30\text{x}60\text{x}8.5\text{x}10^{-3} + 0.6\text{x}0.9\text{x}0.1\text{x}200 + \\ + 0.8\text{x}340\text{x}0.1\text{x}10.59 = 527.8 \text{ kN} \qquad (7.57)$$

and therefore requirement (7.12) can be written as

$$1.2 \times 323.19 = 387.83 \text{ kN} < 527.8 \text{ kN} \qquad (7.58)$$

Requirement (7.15). With L_e and A_a provided by Eqs. (7.56) and (7.53), respectively, requirement (7.15) can be written as

$$1.2\text{x}323.19 = 387.83\text{kN} < 1.5\text{x}10.59\text{x}340\text{x}0.1 = 540 \text{ kN} \qquad (7.59)$$

Requirement (7.16). Both upper face and lower face of horizontal connection must be considered since the geometry of nibs provided at the bottem face of large wall panel differs from that of sockets provided at the top face of wall panel (see Fig.7.20). Also both shear stress distributions in Fig.7.9 must be equally considered in view of different values of A_τ and A_a' in Eqs.(7.17) and (7.18), respectively. In each case the computation follows the steps:

- A_τ in Eq.(7.17) is determined from the appropriate shear stress distribution;
- the required value of A_a' is determined from Eq.(7.18) and from fulfilment of requirement (7.16) at both upper and lower faces of horizontal connection;
- the larger of the above two required values of A_a' is further retained;
- the required A_a' value must be not greater than the provided value of A_a' determined in accordance with Eq.(7.19) where A_a is the total vertical reinforcement provided as projecting and tie bars while $\bar{A}_a$ is the portion of A_a already used as tensile reinforcement when checking the fulfilment of requirement (7.20).

a. Distribution in Fig.7.9, a and Eq.(7.17) yield

$$A_\tau = 31.54 \text{ kN/cm} \qquad (7.60)$$

and, respectively,

$$L_o = 31.54 \times 10 = 315.4 \text{ kN} \tag{7.61}$$

so that requirement (7.16) can be written as (see Fig.7.2, c)

$$A'_a > \frac{1.2x315.4-798/30x6x2x8.5x0.1}{0.8 \times 340 \times 0.1} = 3.94 \text{ cm}^2 \tag{7.62}$$

when the upper face of horizontal connection is considered and

$$A'_a > \frac{1.2x315.4-798/30x5x3x8.5x0.1}{0.8 \times 340 \times 0.1} = 1.45 \text{ cm}^2 \tag{7.63}$$

when the lower face of horizontal connection is considered.

When comparing the above two values required for A'_a it is apparent that limitation (7.62) prevails.

Since 3 ϕ 16 and 1 ϕ 18 bars located at the left-side of shear wall in Fig.7.8 have been used to provide $\bar{A}_a$ reinforcement (see relationship (7.47)), the remainder 5ϕ16 + 2 ϕ 18 + 1ϕ22 bars of the web are available to provide A'_a reinforcement. Hence the effective A'_a is

$$5x2.01+2x2.54+1x3.8=18.94cm^2>3.94cm^2 \tag{7.64}$$

b. Distribution in Fig.7.9, b yields, similarly to Eqs. (7.60)...(7.63),

$$A_\tau = 41.34 \text{ kN/cm} \tag{7.65}$$

$$L_o = 413.4 \text{ kN} \tag{7.66}$$

$$A'_a > \frac{1.2x413.4-798/30x6x2x8.5x0.1}{0.8 \times 340 \times 0.1} = 8.2 \text{ cm}^2 \tag{7.67}$$

and, respectively,

$$A'_a > \frac{1.2x4.13.4-798/30x5x3x8.5x0.1}{0.8 \times 340 \times 0.1} = 5.7 \text{ cm}^2 \tag{7.68}$$

The vertical reinforcement provided as A'_a is made of 6 ϕ 16 + 3 ϕ 18 bars (see relationship (7.50) and Fig.7.8) so that

$$A'_{a,ef}=6x2.01+3x2.54=19.69 \text{ cm}^2> 8.2 \text{ cm}^2 \tag{7.69}$$

It is apparent from the previous computations that detailing of shear wall No.2 of external cross wall DT 1 complies with all strength requirements specified in § 7.4.1.

In addition, it is required that area of vertical reinforcement at both lateral ends of shear wall cross-section shall be not less than 0.05% of the web surface area. The effective values of A_a in Eqs.(7.47) and (7.50) yield

$$p = \frac{8.57}{798x10} = 0.107\% > 0.05\% \tag{7.70}$$

and, respectively,

$$p = \frac{9.83}{798x10} = 0.123\% > 0.05\% \tag{7.71}$$

7.4.3 Proportioning of Central Long Wall DL2

The first storey of the shear-wall No.3 in Fig.7.3 b is detailed in Fig.7.11. The shear-wall cross-section is symmetric and therefore the sign of seismic action is irelevant. The normal and shear stress distributions at the bottom cross-section are determined from the internal forces in Table 7.1 and are depicted in Fig.7.12.

Since the same steps as in § 7.4.2 are required, only numerical computations are presented herein. To ease the reading of relationships they will be denoted by the same number (letter "a" added) as the corresponding ones in § 7.4.2.

A. STRENGTH TO COMBINED BENDING AND AXIAL LOAD

Requirement (7.1). Fig.7.13 yields

$$e_{1s} = 5 \text{ mm and } e_{2s} = 0.2 \text{ mm} \tag{7.24a}$$

and

$$e_{1i} = 0 \quad \text{and} \quad e_{2i} = 0 \tag{7.25a}$$

so that

$$e_s = 5 + 0.2 + 0.03 \times 140 = 9.2 \text{ mm}$$

$$e_i = 0.03 \times 140 = 4 \text{ mm} \tag{7.26a}$$

Vertical strip A in Fig.7.12 is considered and thus

$$G = 0.14 \times 1.0 \times 2.7 \times 25 = 9.45 \text{ kN} \tag{7.27a}$$

$$S = 0.09 \times 9.45 = 0.85 \text{ kN} \tag{7.28a}$$

$$M_s = \frac{1}{6} \times 0.85 \times 2.7 = 0.38 \text{ kNm} \tag{7.29a}$$

$$N = 1.96 \times 10^3 \times 0.14 \times 1.0 = 274.4 \text{ kN} \tag{7.30a}$$

$$e_c = \frac{0.38}{274.4} \times 1000 = 1.3 \text{ mm} \tag{7.31a}$$

$$e_o = \sqrt{0.3(9.2^2+4^2)+0.4 \times 9.2 \times 4}+0.002 \times 2700+1.3 = 13.4 \text{ mm} \tag{7.32a}$$

$$\frac{e_o}{b} = \frac{13.4}{140} = 0.1 \tag{7.33a}$$

$$v = \frac{1.17 - 0.13}{1.96} = 0.53 \tag{7.36a}$$

$$\alpha = \frac{27{,}000}{1.6 \times 8.5 \times (1+1.2 \times 0.53)} = 1220 \tag{7.35a}$$

$$\bar{\lambda} = \frac{1.0 \times 2700}{140\sqrt{1220}} = 0.552 \tag{7.34a}$$

The above values e_o/b and $\bar{\lambda}$ yield (see Fig.7.5)

$$\phi \simeq 0.5 \tag{7.37a}$$

so that

$$\phi R_c = 0.5 \times 8.5 = 4.25 \text{ MPa} \tag{7.39a}$$

and

$$\bar{c}\sigma = 1.2 \times 1.96 = 2.35 \text{ MPa} < 4.25 \text{ MPa} \tag{7.38a}$$

Requirement (7.21)

$$\sigma_{max} = 2.79 \text{ MPa} \tag{7.41a}$$

$$e_i = 4 \text{ mm} \tag{7.42a}$$

$$\sigma'_{max} = 2.79\ (1+ \frac{6 \times 4}{140}) = 3.27 \text{ MPa} \tag{7.43a}$$

and

$$1.2 \times 3.27 = 3.29 \text{ MPa} < 0.7 \times 8.5 = 5.95 \text{ MPa} \tag{7.44a}$$

Requirement (7.20)

$$V_\sigma = \frac{1}{2}(0.44+0.378) \times 10^3 \times 0.14 \times 1.5 + \frac{1}{2} \times 0.378 \times \times 10^3 \times 0.14 \times 0.852 = 108.43 \text{ kN} \tag{7.48a}$$

$$\bar{A}_a = \frac{1.2 \times 108.43}{340 \times 0.1} = 3.82 \text{ cm}^2 \tag{7.49}$$

The effective area is made of 3 ϕ 16 + 1 ϕ 20 bars and thus

$$\bar{A}_{a,ef} = 3 \times 2.01+1 \times 3.14 = 9.17 \text{ cm}^2 > 3.82 \text{ cm}^2 \tag{7.50a}$$

B. SHEAR STRENGTH

Requirement (7.10)

$$Q = 1.5 \times 395 = 592.5 \text{ kN} \tag{7.51a}$$

The horizontal bars are 8 ϕ 12 + 1 ϕ 16 so that

$$A_a = 8 \times 1.13 + 1 \times 2.01 = 11.06 \text{ cm}^2 \tag{7.53a}$$

and

$$Q_{lim} = \frac{7.34}{2.70}(0.8x11.06x340x0.1+0.3x0.14x2.7x0.9x10^3) = = 1095 \text{ kN} \tag{7.52a}$$

$$1.2 \times 592.5 = 711 \text{ kN} < 1095 \text{ kN} \tag{7.54a}$$

Requirement (7.12)

$$\tau = 0.72 \text{ MPa} \tag{7.55a}$$

$$L_e = 0.72 \times 10^3 \times 0.14 \times 2.7 = 272.5 \text{ kN} \tag{7.56a}$$

From Figs.7.2b and d

$A^{v} \approx 200 \text{ cm}^2$
$d_{sv} = 30$ mm
$b_{sv} = 60$ mm
$n_v = 15$

and with due account to Eq.(7.53a)

$$T_{max} = 15x30x60x8.5x10^{-3}+0.6x0.9x0.1x200+0.8x340x0.1x11.06 = = 540.8 \text{ kN} \tag{7.57a}$$

so that

$$1.2 \times 272.5 = 327 \text{ kN} < 540.8 \text{ kN} \tag{7.58a}$$

Requirement (7.15)

$$1.2x272.5=327 \text{ kN} < 1.5x11.06x340x0.1= 564 \text{ kN} \tag{7.59a}$$

Requirement (7.16)

$$A_{\tau} = 18.43 \text{ kN/cm} \tag{7.60a}$$

$$L_o^{\tau} = 18.43 \times 14 = 258 \text{ kN} \tag{7.61a}$$

$$A_a^{v} > \frac{1.2x258-734/30x6x2x8.5x0.1}{0.8x340x0.1} < 0 \tag{7.62a}$$

$$A_a^{v} > \frac{1.2x258-734/30x5x3x8.5x0.1}{0.8x340x0.1} < 0 \tag{7.63a}$$

$$\bar{A}_{a,ef} = 3 \times 2.01+1 \times 3.14 = 9.17 \text{ cm}^2 \tag{7.47a}$$

$$p = \frac{9.17}{734x14} = 0.089\% > 0.05\% \tag{7.70a}$$

7.5. CONCLUDING REMARKS

The design example is typical for the current design philosophy and structural analysis of low-rise apartment houses built with precast large panels. The most important of the features outlined within the design example are reviewed and discussed herein.

1. The structure conformation in both horizontal and vertical direction is as regular as possible.

2. All structural members but footings and basement walls are precast. The connections between precast large panels are designed to ensure that structure resembles in strength a monolithic one.

3. The seismic-resistant structure is analyzed as if it were monolithic.

4. Gravity loads induce only axial compression of shear-wall cross-section.

5. Two approaches are available to analyze the seismic response of structure:
a. Equivalent lateral force procedure.
b. Elastic modal analysis procedure.

The latter has been employed here in.

Structure is virtually supposed to withstand lateral seismic action by flexural stiffness. Correcting factors are sometimes used to account for the shear stiffness of cantilever shear-wall [7.5]. Multiplication of required strength by $\bar{c} = 1.2$ aims at covering this very weakness of analysis.

6. Strength of RC structural members follows from condition that maximum elastic stress does not exceed the appropriate design strength. However, formulae used to compute the design strength of structure are not always consistent with the development of required strength (e.g. see strength requirements associated with connections).

7. There is no mention of any mechanism associated with postelastic deformation of structure. However, a tentative is made to avoid shear failure by multiplying required shear strengths by 1.5.

More than 60,000 flats are built every year in Romania using the large panel system presented here in. In addition, a different system is currently used for 9-storey structures [7.3]. Both systems performed quite well during the 1977 earthquake and most likely that that was due more to the sound structural layout and careful detailing of large panels and connections than to proper analysis and proportioning. The fact is generally agreed upon in Romania, and a tentative is in progress to improve the current code of practice for large panel structures which pays more tribute to old-fashioned aseismic design criteria than any other type of RC structure. Nevertheless the task is most difficult since there are still many questions to answer. The most important among them are worth mentioning here in :

(i) which are the strength and deformability of currently used connections when subjected to seismic action ?

(ii) is the actual seismic performance of large panel low-rise structure similar to that of the monolithic structure ? If not, what is the effect of currently used connections on the inelastic seismic response of precast structure and which is the best way to consider it ?

REFERENCES[1)]

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[7.2] *** "The Romanian Aseismic Code of Practice - P100", 1981, Bucharest.

[7.3] *** "Prefabricated Concrete Buildings in Romania. State-of-the-Art Report", Input Report for WGB of UNIDO Regional Projekt RER/79/015, 70 pp, 1982.

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[7.5] Agent, R. and Postelnicu, T. "Analysis of RC Shear-Wall Structures" (in Romanian), Ed.Tehnica, 196 pp, Bucharest.

[7.6] Diaconu, D. e.a. "Experimental Investigation of the Dynamic Seismic Response of a Large Panel Structure with Non-Bearing Facade Walls" (in Romanian), ICCPDC, Yassy, 1982.

[7.7] Irescu, R. e.a. "Theoretical and Experimental Investigation of the Inelastic Response of a Large Panel Shear Wall" (in Romanian),INCERC, Bucharest, 1982

[7.8] Stanescu, M. and Stefanescu, M. "Investigation of Inelastic Response of Vertical Connection of large Panel Structures" (in Romanian), INCERC, Bucharest, 1982.

NOTATION

Used		CEB
A'	- surface area of concrete cross-section of horizontal connection	-
A_a	- surface area of longitudinal reinforcement	A_s
H	- storey height	-
L_e	- shear force acting across vertical connection	$V_{act,v}$
L_o	- shear force acting across horizontal connection	$V_{act,h}$
L_{uh}	- shear resistance of horizontal connection	$V_{res,h}$
N	- axial force	N
Q	- required shear strength across a diagonal crack	V_{act}
Q_{lim}	- design shear strength across a diagonal crack	V_u

1) The present list is restricted to references published in Romanian which are legal background to or provide further information on aspects arised by the design example.

R_a	- design yield strength of reinforcement	f_s
R_c	- design compressive strength of concrete	f_c
S	- seismic force	E
T_{max}	- shear resistance of vertical connection	$V_{res,v}$
b	- thickness of shear-wall web	b
$\bar{c}$	- coefficient	-
e	- vertical eccentricity	e
h	- length of shear-wall	h
p	- percentage of longitudinal reinforcement	
Ø	- diameter of reinforcing bar (e.g. 3Ø12 means 3 bars of 12 mm diameter)	Ø
$\bar{\lambda}$	- reduced slenderness	-
σ	- normal stress	σ
σ_g	- part of σ due to gravity loads	σ_g
τ	- shear stress	τ
ϕ	- stength reduction factor related to slenderness	-

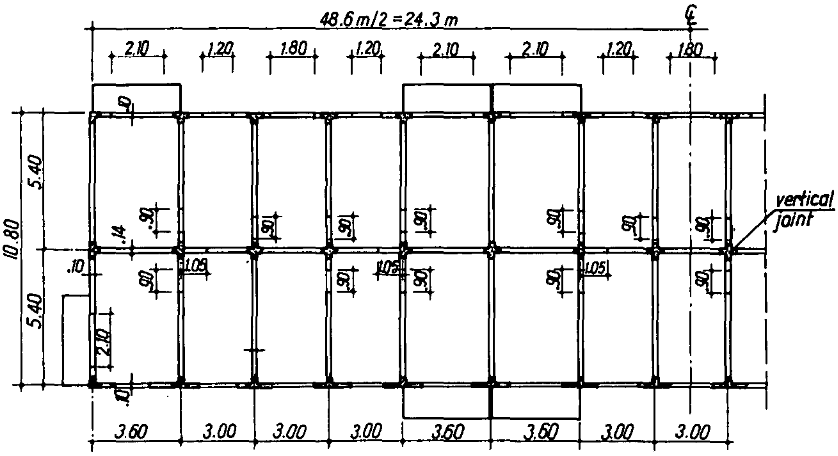


Fig. 7.1 – Structural lay-out

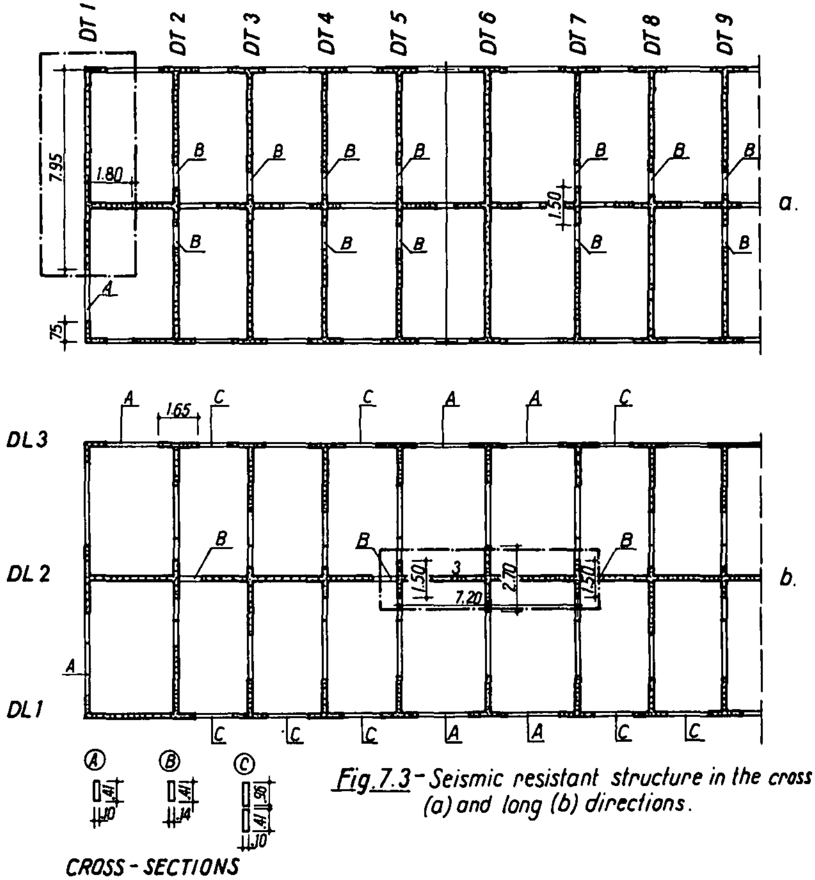


Fig. 7.3 – Seismic resistant structure in the cross (a) and long (b) directions.

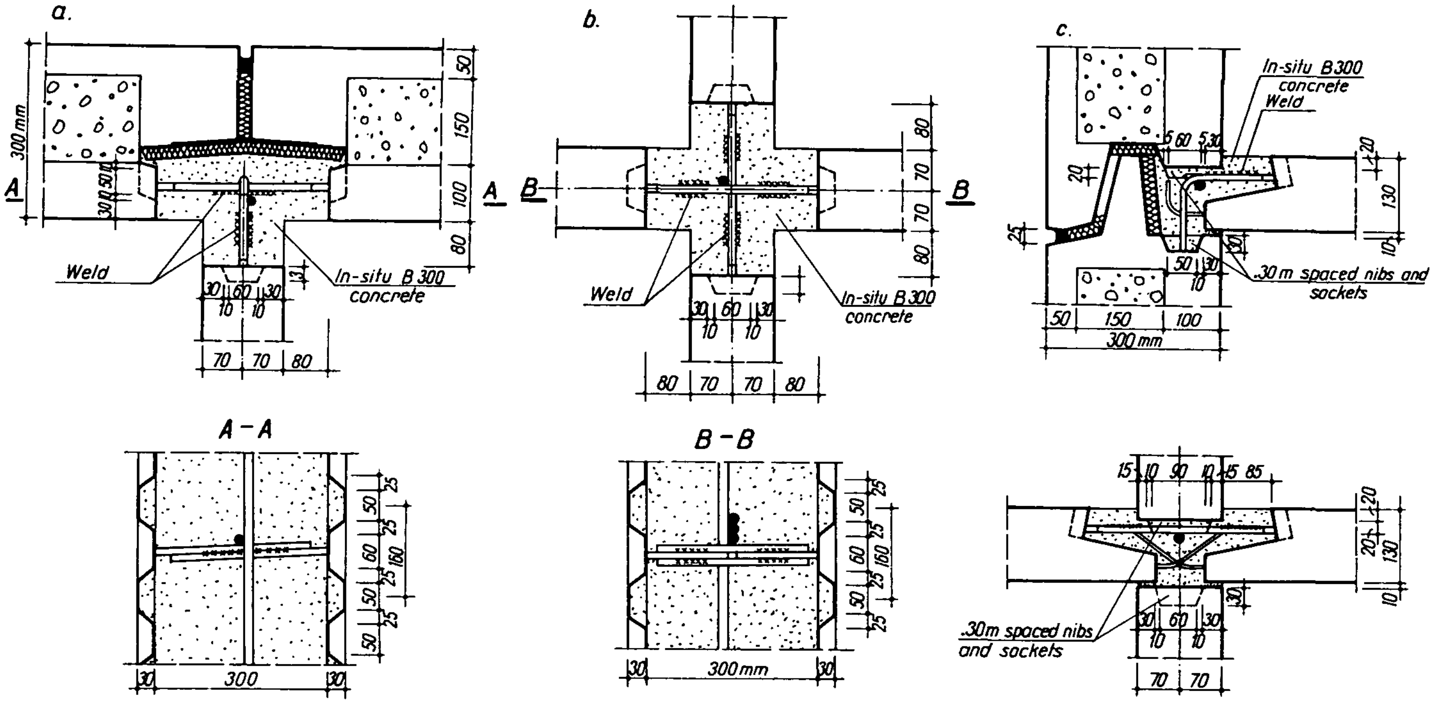


Fig. 7.2- Connections. (a) External vertical, (b) Internal vertical, (c) External horizontal, (d) Internal horizontal

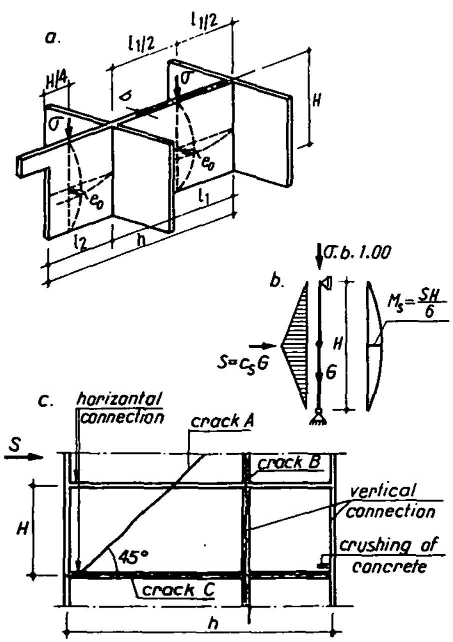


Fig. 7.4

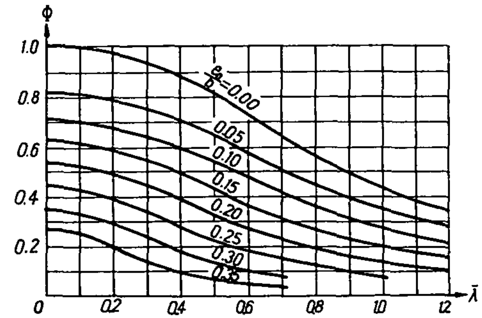


Fig. 7.5- Φ-Values [7.1]

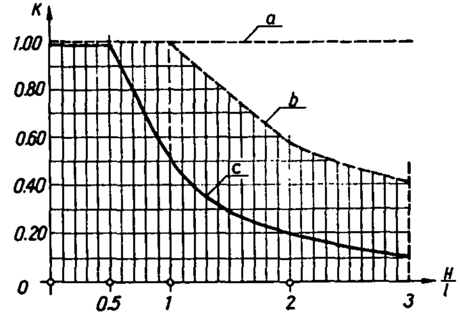


Fig. 7.6- K-Values [7.1]

a. lateral deformation of both vertical sides is free to occur
b. lateral deformation of one vertical side is free to occur while the other is restrained
c. lateral deformation of both vertical sides is restrained

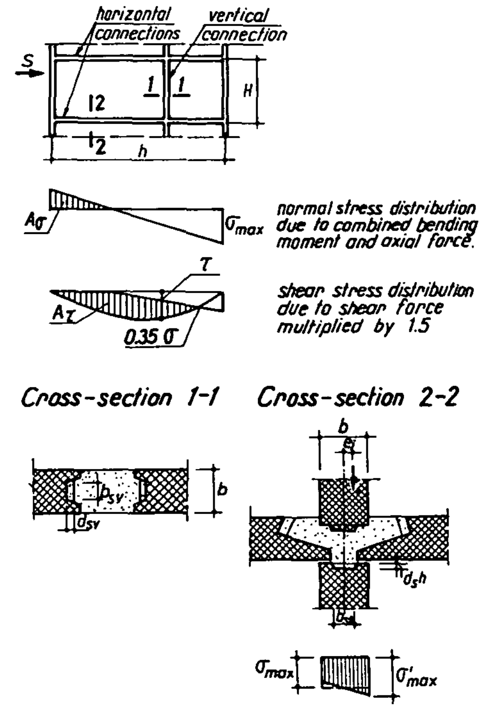


Fig. 7.7

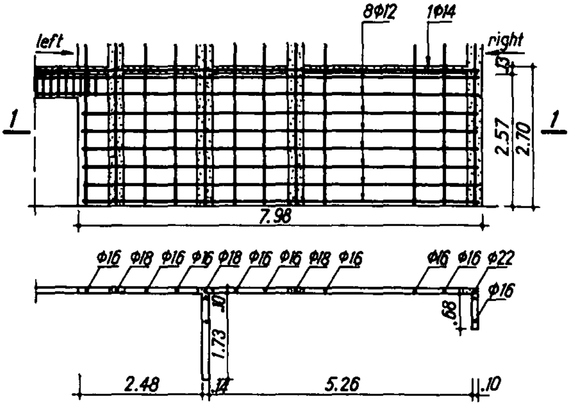


Fig.7.8-Detail of first storey of shear-wall No.2 of cross wall DT 1 (see Fig.7.3a). Only tie and projecting bars are depicted.

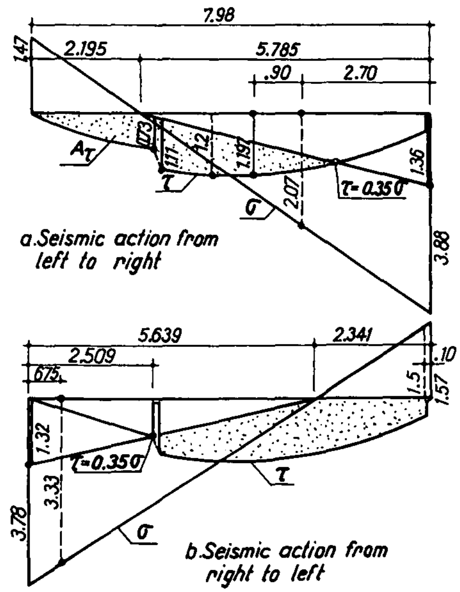
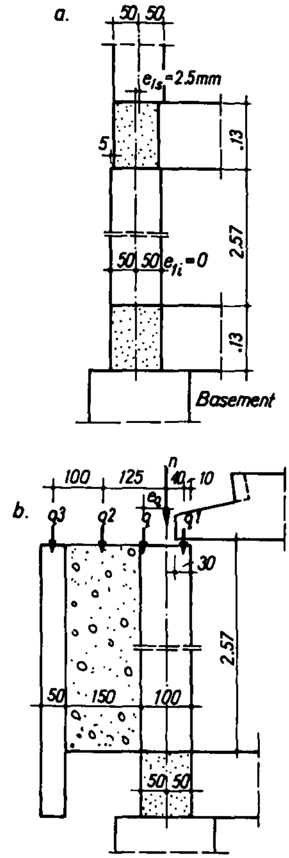


Fig.7.9- Normal and shear stress distributions (in MPa)



$q_1 = 9.90\,kN/m$

$q_2 = 2.40\,kN/m$

$q_3 = 3.40\,kN/m$

$q = q_1+q_2+q_3 = 15.70\,kN/m$

$e_q = 43\,mm$

$\sigma_g = 1.14\,MPa$

$n = \frac{4}{5}\times 1.14\times 100 = 91.2\ kN/m$

$e_{2s} = \frac{q e_q}{n+q} = \frac{15.7\times 43}{91.2+15.7} = 63\,mm$

$e_{2i} = 0$

Fig.7.10-Compution of eccentricities e_1(a) and e_2(b) of shear wall DT_1

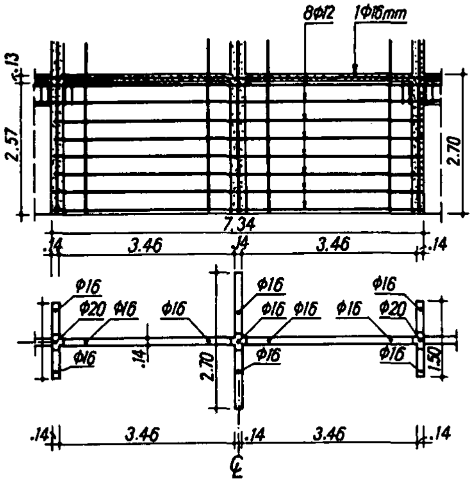


Fig. 7.11 - Detail of first storey of shear wall No.3 of long wall DL2 (see Fig. 7.3 b). Only tie and projecting bars are depicted.

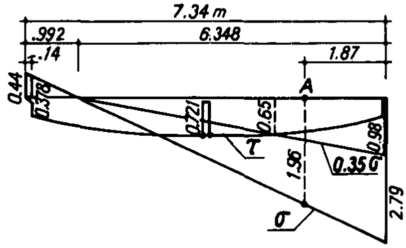
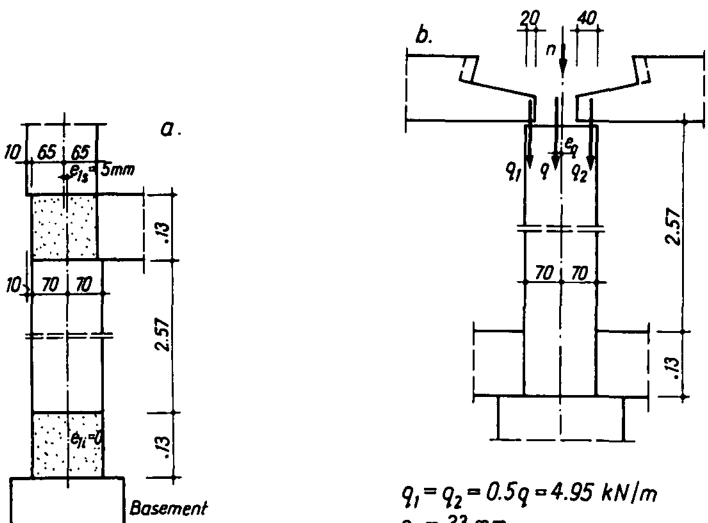


Fig. 7.12 - Normal and shear stress distributions (in MPa)



$q_1 = q_2 = 0.5q = 4.95$ kN/m

$e_q = 33$ mm.

$\sigma_g = 1.176$ MPa

$$n = \frac{4}{5} \times 1.176 \times 140 = 131.71 \text{ kN}$$

$$e_{2s} = \frac{q e_q}{n+q} = \frac{9.9 \times 3.3}{131.71 + 9.9} = 0.2 \text{mm}$$

Fig. 7.13 - Computation of eccentricities e_1 (a) and e_2 (b) of shear - wall D2

Appendix : EXPERIMENTAL RESULTS

The seismic response of a structure made with precast large panels is crucially dependent on the particularities of connections which may well differ from country to country as their design has to consider aspects as various as technical expertise, workmanship, costs, existing facilities etc.

It is therefore less possible to use other countries experience when dealing with the design of large panel structures than it is the case with cast-in-situ structures.

Every new large panel system need thorough investigation before using it safely and economically. Experimental results provide invaluable information on the validity of a large panel system and no such system is practically implemented in Romania without experimental support.

Although experimental evidence on the current large panel systems in Romania has increased every year over the last decade, it is far from being satisfactory. This situation is partly due to the limits imposed by the available experiment facilities and partly due to the rapid progress of aseismic design philosophy.

Some of the major questions still to answer are mentioned at the end of § 7.5. The experimental investigations carried out in Romania in 1982 in conjunction with those questions are briefly presented and discussed here in.

1. Investigation [7.8] analyzes the postelastic shear response of current vertical connections. Full scale models of vertical connections between large wall panels were subjected to static shear forces and the force-displacement relationship was recorded. The investigation aimed at finding evidence on the descending branch of shear force-shear displacement relationship, on the accuracy of Eq.(7.14) as well as on the effects of parameters such as the vertical distribution and type of horizontal projecting bars on the shear strength of vertical connection.

The main conclusions provided are:

- after the cracking of concrete the shear force which vertical connection can carry decrease to about 0.7 ... 0.8 of the maximum shear force before cracking;

- the overlapped loop-shaped projecting bars provide a lower decay of the post-cracking shear resistance than the welded bars projecting in the median plane of the large wall panel;

- the post-cracking shear resistance is practically constant while shear displacement increases well into the inelastic range;

- the post-cracking shear resistance is about 40% higher than the design shear strength provided by Eq.(7.14).

2. Investigation [7.7] analyzes the inelastic response of a 4-storey shear wall made of 2x4 large panels. The shear wall is 6.14 m long, 10.82 m high and 0.14 m thick. Two 1/2 - scale models of the shear wall were subjected to static, reversed lateral forces applied at the level of third and fifth floor. Transverse members to model the effects of perpendicular shear walls and floor slabs were included and the effect of gravity loads was considered. The investigation aimed at finding evidence on the hysteretic behavior of shear wall, on its capacity to absorb and dissipate energy as well as on the cracking and failure mechanisms of the precast shear wall. The experimental value of ultimate lateral deformation of shear wall is compared with a theoretical estimate based upon considering the effect of both flexural and shear deformations of a two-degree-of-freedom equivalent system subjected to monotonously increased, static loading.

The main conclusions provided are:

- the crack pattern is similar to that of a cast-in-situ shear wall;

- the failure mechanism develops from the normal crack placed at the level of first floor; this crack crosses the entire length of shear wall as result of reversed inelastic bending and widens progressively as the shear wall deforms well into the inelastic range;

- the theoretical estimate of ultimate lateral deformation of shear wall is in good agreement with the experimental evidence.

3. Investigation [7.6] analyzes the dynamic response of a new large panel structure having a median long shear wall and non-bearing facade walls when subjected to longitudinal vibrations. The structure has 5-storey, is 15.14 m long and 11.06 m wide and is designed for a zone with seismicity degree equal to 7 on a MSK scale.

A 1/5 - scale model of the structure was subjected to a series of artificial and recorded accelerograms on a shaking table. Static loadings applied at various stages of structure (uncracked, cracked and before collapse) were intercalated. The experimental investigation aimed at providing evidence on the seismic response of the structure as well as on the dynamic characteristics and relative stiffnesses of structural subassembleges at various stages of degradation.

The experimental results indicated the good seismic performance of structure and recommended the validation of the project.

8. REPRESENTATIVE EXAMPLE OF BUILDING SYSTEMS TURKEY

Prepared by K. ÖZDEN

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8.1. SCOPE

Outline of design principles of reinforced concrete multi-storey earthquake resistant buildings of prefabricated large wall and floor panels is illustrated on design example which is for an 8-storey residantial building shown in figure (8.1) in Chapter-Figures. The structure is in a seismic region and rests on a fairly thick sand and gravel layer. The structural analysis, proportioning of structural elements, and detailing of panels and joints are carried out in accordance with Ref [8.1], [8.2] and [8.4].

8.2. DESCRIPTION OF STRUCTURAL SYSTEMS

8.2.1. General Description

Typical floor plan and panelization drawing are given in Figure (8.1). All wall panels in both directions are load-bearing, except external walls on 1-1 and 11-11 axes parallel to Y direction and the thin walls shown on the plan. The shape of the building is regular. The foundation of the building is of continuous footing in both directions in such a way as to have beams under all load-bearing walls and building's periphery. The height of foundation beams is 1.90 m and the top of beam is 30 cm above ground level. In the first floor the prefabricated walls are supported on these beams. As the foundation system consisting of orthogonal beams is rigid enough in its plane, the 8-storey building is assumed to be fixed supported at the top level of foundation.

The floor slabs are made of precast panels. During erection the panels are supported temporarily on four points of two steel I profiles cast in two parallel edges of the panels during fabrication. They are turned into panels supported on three or four edges by means of cast-in-place reinforced concrete joints between slabs and walls. The continuity of reinforcing bars in the adjacent slabs are maintained by looped connections to provide the required bond length. The joints are also provided with tensile reinforcements of peripheral tensile elements in order to form horizontal tensile elements. These joints also connect horizontally the walls above and below the slab. The keys and castellations on all sides of floor panels and on top and bottom sides of wall panels secure the horizontal connection of floor and wall panels so as to transfer shear forces the floor panels can transfer fully the horizontal loads to the load-bearing walls.

In the joints of load-bearing walls both in their plane and perpendicular to their plane, the column-like elements of 0,24 to 0,44 m in width are constructed by using vertical reinforcements and cast-in-place concrete. The shear keys on vertical edges of wall panels provide the transfer of shear forces fully. The tensile forces in the joints are resisted by hooks placed horizontally during fabrication and additional loops placed during erection.

The tensile forces resulting from the cantilever behaviour of load-bearing walls under horizontal loads are resisted by bolts connected each other by reinforcing bars running through along the height of wall. There are at least two bolts in each wall panel to maintain ease in erection and proper connection. At the places, which the bolts don't suffice additional vertical reinforcement bars be used.

Therefore, the system formed thus by wall and floor panels behaves as monolithic structures under horizontal and vertical loads by use of the above described joint details and tie elements.

The tie elements are detailed in such a minimum size that in case a load bearing wall panel loses its carrying capacity due to an abnormal use, the panel above will either act as a horizontal cantilever or a deep beam on two supports thus the remaining structural elements will not suffer any damage. An important weakness of prefabricated buildings has been removed completely in this way.

8.2.2. Precast Large Panels

The vertical and horizontal sections of load-bearing and non-loadbearing facade panels and load bearing wall panels are shown in Chapter 8.5. Selected Details. The facade and internal load-bearing wall panels subjected to large shear forces have castellations of different sizes on horizontal and vertical edges. The load-bearing walls subjected to comparatively smaller shear forces may not contain keys on top and bottom edges, but reinforcing hooks and loops. Their vertical edges only will have castellations. The wall panels have at least two erection bolts on their top edges and their reservations on the bottom edges. Where additional vertical reinforcements and the erection bolts are not sufficient to resist the tensile forces on horizontal section of wall panels under horizontal loads, special steel box reservations are used and the number of bolted connection is increased accordingly. The tensile forces on vertical sides due to loads perpendicular to wall planes and due to eccentricity are resisted by reinforcements placed on these sides. In the mid-plane of wall panels, mesh steel of 4,5 mm diameter with 150 mm spacing bothways is placed.

Floor panels are 160 mm thick. On two parallel sides, they have two IPN 80 steel profiles on which they are supported during erection. Floor panels have castellations on all four sides and hooks to provide splicing of bars for bond and moment transfer. The top view and section of floor panels are shown in Chapter 8.5.

All panels are produced with B300 concrete. The density of reinforced concrete is about 25 kN/m^3. The reinforcing bars are deformed BÇ III (St IIIb) with a characteristic yield stress of $f_{yk} = 400$ MPa.

8.2.3. Connections

The floor panels are supported and aligned on the walls by the use of steel profiles cast-in-place during fabrication. The horizontal joints between floor panels are made by placing loops between hooks and longitudinal bars. Afterwards the wall panel above is erected by tne use the two bolted connections and the temporary erection struts are placed for vertical alignment of the wall plane. The alignment of wall is adjusted by the bolted connections which also provide the splicing of tensile reinforcements. Then the reinforcing loops on the sides are placed to splice the hooks on the panels, and concrete of B300 quality is cast-in-place to complete the connections. The connections of the hooks at the vertical joints along the vertical edges of the wall panels are carried out with loops and the vertical joint is cast in concrete after the longitudinal reinforcement is placed. It is necessary to use side formwork to cast concrete in the joints (See chapter 8.5).

8.3. STRUCTURAL ANALYSIS

As seen in the previous Chapter, the load bearing structure is a space system formed by interconnection of shearwalls in two perpendicular directions x-x and y-y together with lintels above and below openings, and by construction of floor panels exhibiting diaphragm action. The connections of shear walls with walls perpendicular and in their own plane, and with diaphragms are made rigid against shear forces. Consequently, the shear walls connected with perpendicular walls will have flanges as parts of walls joining perpendicular to them.

The sections of a shear wall parallel to y-y direction and coupling beams are shown in Figure (8.2). The effective flange width for two-sided flanges is the smallest of the following values [8.3]:

- 16 b_w (b_w being width of flange)
- height of shear wall cross section
- 4 m
- 1/5 of height of building

As seen in the cross-section of coupling beams, the lintels below and above in the panels are assumed to act independently during deformation of the structural system.

In the structural desing, the linear elastic theory has been used. And in the design of structures for earthquake loads the stiffness of lintels are taken as 0,65 $E_c I_c$ and 0.15 $E_c I_c$ and two analysis are made. The first design results are used for proportioning of lintels, and the second design results for shear walls.

Analysis of the structural system is carried our for

(a) Vertical loads,
(b) earthquake loads in two directions.

The internal stresses due to imposed deformations are neglected when the dimensions of the building are less than 25 m, the settlement of soil under foundation is small, and the building is not too high.

8.3.1. Design Loads

The following load combinations shall be taken as design loads, G being the specific value of vertical permanent load, and Q being the nominal value of live load as in [8.4]. In the analysis for vertical loads only:

$$1.4\ G + 1.6\ Q \qquad (8.1a)$$
$$1.0\ G + 1.2\ Q + 1.2\ T \qquad (8.1b)$$

In cases where earthquake is involved:

$$1.0\ G + 1.0\ Q + 1.0\ E \qquad (8.2a)$$
or $$1.4\ G + 1.6\ Q \qquad (8.2b)$$
or $$0.9\ G + 1.0\ E \qquad (8.2c)$$

where,
T = Loads due to imposed deformations
E = earthquake loads

In the computation of E, the weight w_i at the i th floor is:

$$w_i = G_i + 0.30\ Q_i \qquad (8.3)$$

where, G_i = sum of dead loads of floor slabs and half weights of walls above and below the slabs.

Q_i = total live load on floor slabs.

The floor loads used in earthquake design are computed using (8.3)

For roof floor $W_8 = 6043.9$ kN

For typical floor $W_i = 5521.0$ kN

and the total of floor loads W is:

$$W = 7\ W_i + W_8 = 44690.9 \text{ kN}$$

The total horizontal earthquake shear forces in the base of building using a semi-modal method in accordance with (8.3) is:

$$E = C.W \tag{8.4}$$

$$C = C_o K\ S\ I \tag{8.5}$$

where,

C_o = earthquake zone coefficient
K = structural type factor
S = dynamic coefficient for the structure (spectral coefficient)
I = building importance coefficient

where,

$$S = \frac{1}{|0.8 + T - T_o|} \leq 1 \tag{8.6}$$

and

T = Natural period of the building, sec.
T_o = Predominant period of the underlying soil stratum, sec.

T can be approximated by using one of the following expressions which gives larger value for S:

$$T = 0.09 \frac{H}{\sqrt{D}} \quad , \quad T = (0.07 \sim 0.10).N \tag{8.7}$$

where,

H = height of building above foundation in m,
D = width of building plan in earthquake direction
N = number of storeys.

For the subject building:

from (8.7) $T = 0.09 \dfrac{8 \times 2.7}{\sqrt{21}} = 0.43$ sec , $T = 0.56 \sim 0.8$ sec

According to [8.4] taking $T_o = 0.42$ sec , from (8.6) S is calculated as 0.85.

In the structural analysis S is taken as 1.

Thus,

$C_o = 0.08$, $K = 1$, $S = 1$, $I = 1$

From (8.5)

$$C = 0.08 \times 1 \times 1 \times 1 = 0.08$$

Total shear force E at the base of building from (8.4) is:

$$E = C.W = 0.08 \times 44690.9 = 3575.3 \text{ kN}$$

Equivalent static loads at each floor, E_i can be calculated using the expression below:

$$E_i = E.(w_i h_i / \sum_{i=1}^{8} w_i h_i) \quad \text{(kN)} \qquad (8.8)$$

where ,

W_i = floor load for i th floor
h_i = height from top of foundation to the i th floor.
(The computed values are summarized in the Table 8.1).

Table 8.1

E_i Statical Equivalent Floor Loads

i	W_i	h_i	$W_i h_i$	$\frac{E}{\sum W_i h_i}$	E_i
	kN	m	kN.m	10^{-4}	kN
8	6043.9	21.6	130548.2	6525.03	851.8
7	5521.0	18.9	104346.9	"	680.9
6	"	16.2	89440.2	"	583.6
5	"	13.5	74533.5	"	483.6
4	"	10.8	59626.8	"	389.1
3	"	8.10	44720.1	"	291.8
2	"	5.40	29813.4	"	194.5
1	"	2.70	14906.7	"	97.3

8.3.2. Computation Techniques

The structural analysis of floor panels during erection is carried out as two beams of 0.25 m width supported on two I profiles on two parallel edges under its own dead weight, and after erection as plates supported-mostly on 4 edges, sometimes on 3 and rarely on 2, under dead load plus covering and superimposed loads.

Plate support reactions thus obtained are transformed into equivalent uniform loads acting on the width of shear walls including the portions distributed to lintels above door and windows.

Axial forces N at each floor are obtained by adding the dead weight of the wall panels onto these uniform loads. These axial forces N are used in the load combinations mentioned in paragraph 8.3.1. In the subject example, analysis using load combinations (8.1a) and (8.2a) are found sufficient.

Bending moments due to vertical loads whose moment vectors are parallel to horizontal edges of panel, are obtained using initial eccentricity e_o

$$M_n = e_o N \tag{8.9}$$

Structural analysis for horizontal earthquake loads are made separately in x-x and in y-y directions under earthquake design loads at each floor (see Table 8.1). Although approximate methods giving satisfactory results are known (e.g. [8.5]), this example analysis is performed by using a computer-program named GØS161 [8.6], in computing center of Istanbul Technical University.

8.3.3. Internal Forces

8.3.3.1. Reactions Due to Earthquake

It is sufficed to state in Table 8.2 the internal forces in the most unfavourable shearwall sections and coupling beams (lintels).

Table 8.2

Internal Forces of the Most Unfavourable Elements

Element	Name	Number of Storey	Place of Section	Bending Moment M_y	Shear Force V_y	Axial Force N	Section
				kNm	kN	kN	
Shear-wall	P_6	1	bottom	2843.0	362.	1766.	$A_c = 1.325\ m^2$ $I_c = 2.416\ m^4$ $y_1 = 2.41$ m $y_2 = 2.27$ m See Fig. 8.3.
Shear-wall	P_6	2	bottom	1867.0	375.	1345.	
Lintel	L_{4a}	3	Above window	41.5	82.	-	0.16 x 0.50 m^2
Lintel	L_{4b}	3	Below window	72.0	142.	-	0.16 x 0.60 m^2

8.3.3.2. Effects on Shear-walls for Loading without Earthquake

The sections of subject shear-walls for loading case without earthquake loads under load combination (8.1a) are subjected to the following axial forces:

1st floor bottom section N = 2527.0 kN
2nd floor bottom section N = 2266.0 kN

However, shear walls may be subjected to bending moment M_n whose moment vectors are parallel to horizontal edges of panels without having external forces (such as wind and earthquake) perpendicular to their own planes due to a lateral initial eccentricity e_o as a result of their normal forces or support reactions being off-center and irregulaties in midplanes. Also, earthquake loads perpendicular to their plane cause additional moments.

Furthermore, slenderness of shear-walls in lateral direction result in second order moments which are dependent upon slenderness ratio and lateral eccentricity e_o.

Lateral eccentricity e_o is defined as [8.2]:

$$e_o = \sqrt{0.3(e_s^2 + e_i^2) + 0.4\, e_s e_i} + e_h + e_E \qquad (8.10)$$

where e_s, e_i = algebraic sum of eccentricities at the top and bottom end section of the panel, these, being due to all causes other than wind and earthquake effects.

e_h = 0.002 ℓ and is an unavoidable, out of plane eccentricity of shear-wall panel

e_E = eccentricity at mid-height of panel due to earthquake acting percendicularly to the large wall panel.

ℓ = height of panel

The eccentricities e_s and e_i can be calculated as:

$$e_s,\ e_i = e_1 + e_2 + 0.03\, b_w \qquad (8.11)$$

where,

e_1 = eccentricity due to wall panels with mid-planes not in the same vertical plane.

e_2 = eccentricity due to vertical loads acting on shear-wall panel (see Fig.8.4).

The eccentricity e_E is calculated using moment M_E due to earthquake load E_1 acting on a vertical strip 1 m width of wall panel

$$e_E = M_E/N_1 \qquad (8.12)$$

Considering a vertical panel strip of 1 m width of weight G_1 the total earthquake load is:

$$E_1 = C_o G_1 = 0.08\, G_1 \qquad (8.13)$$

Assuming that this load is distributed as second degree parabola, the moment M_E can be defined as:

$$M_E = \frac{\ell}{6.4} E_1 \qquad (8.14)$$

and N_1 can be expressed in terms of σ_v which is the stress due to axial force N.

$$N_1 = \sigma_v . b_w . 1 \qquad (8.15)$$

In the first storey, the eccentricities at panel P_6 as seen in figure 8.4a and 8.4b are;

e_{1s} = 5 mm (assumption), $e_{1i} = 0$

$e_{2s} = 0$, $e_{2i} = 0$

Using the above values, the eccentricities e_s, e_i are calculated as:

e_s = 5.0 + 0.03 x 160 = 9.8 mm
e_i = 0 + 0.03 x 160 = 4.8 mm

The eccentricity e_h;

$$e_h = 0.002 \times 2700 = 5.400 \text{ mm}$$

Eccentricity e_E at 1 m vertical strip adjacent to external edge without a flange:

$$G_1 = (0.16 + 0.02) \times 1 \times 2.7 \times 25 = 10.8 \text{ kN}$$

From (8.13)

$$E_1 = 0.08 \times 10.8 = 0.87 \text{ kN}$$

From (8.14)

$$M_E = \frac{2.7}{6.4} \times 0.87 = 0.37 \text{ kNm}$$

Using N and A_c values given in Table 8.2 normal stress σ_v on panel P_6 is:

without earthquake $\sigma_v = 2527/1.325 = 1.907 \text{ kN/m}^2$

with earthquake $\sigma_v = 1766/1.325 = 1.333 \text{ kN/m}^2$

Using these values and (8.15) for N_1;

without earthquake $N_1 = 0.16 \times 1 \times 1907 = 305.1$ kN
with earthquake $N_1 = 0.16 \times 1 \times 1333 = 213.3$ kN

e_E using (8.12);

without earthquake $e_E = 0.37/305.1 = 0.0012 \text{ m} = 1.2 \text{ mm}$

with earthquake $e_E = 0.37/213.3 = 0.0017 \text{ m} = 1.7 \text{ mm}$

e_o, using (8.10);

without earthquake $e_o = 12.7$ mm
with earthquake $e_o = 14.4$ mm

The moment M_n in 1 m vertical strip can be calculated with the following expression;

$$M_n = e_o N_1 \tag{8.16}$$

without earthquake $M_n = 0.0127 \times 305 = 3.9$ kNm/m
with earthquake $M_n = 0.0144 \times 213.3 = 3.1$ kNm/m

Second order effects:
In part I of the panel P_6 as shown in Fig. 8.3, using the coefficient k in accordance with 8.2 :

$$k = 1 - (1 - \frac{\sqrt{3}}{3})(\frac{\ell}{b} - 1) = 1 - (1 - 0.577)(\frac{2.70}{1.52} - 1) = 0.71 \tag{8.17}$$

buckling length and slenderness λ are obtained as;

$$k\ell = 0.71 \times 2.70 = 1.92 \text{ m}$$

$$\lambda = \frac{k\ell}{i} = \frac{1.92}{0.289\ b_2} = \frac{1.92}{0.046} = 41.5 > 22$$

Since $\lambda > 22$ second order stresses are present and taking the first order moment as M_n, its value together with the second order moment is M_{nt};

$$M_{nt} = \beta M_n \tag{8.18}$$

where the maximum value of β is:

$$\beta = 1/(1-N_1/N_k) \tag{8.19}$$

$N_k = \pi^2(EI)/(k\ell)^2$ Euler critical load

$EI = E_c I_c/2.5\left[1 + N(\text{permanent})/N_1(\text{total})\right]$

For the subject 1 m strip:

$E_c = 30250$ MPa

N(permanent)/N(total) = 1490,6/1766 = 0.84

$$EI = 3025000\,\frac{1 \times 0.16^3}{12 \times 2.5(1 + 0.84)} = 2238.5 \text{ kNm}^2$$

$$N_1 = 10 \times 2238.5/(0.71 \times 2.70)^2 = 6099.5 \text{ kN}$$

Using these values the value of β is found;

$$\beta = \frac{1}{1 - 305/6099.5} = 1.05$$

M_{nt} for loading case without earthquake can be obtained from (8.18);

$$M_{nt} = 1.05 \times 3.9 = 4.10 \text{ kNm/m}$$

The axial force in this vertical strip is:

$$N_1 = 305.1 \text{ kN/m}$$

8.4. PROPORTIONING

8.4.1. Shear-Wall

In case where earthquake loads are applied in positive and negative direction of y-y axis as shown in Fig.8.2, using elastic theory normal and shear stresses (computed by 1,5 time shear force) acting at the floor levels of the first and second storeys to the shear-wall P_6, which selected as example are stated in Figure 8.5.

8.4.1.1. In Shear-Wall Safety against Buckling should be maintained

In shear-wall safety against buckling should be maintained in such a way:

(a) $\gamma_{n1}\, \sigma_c \leq \phi(e_o/b_w, \bar{\lambda}) f_{cd}$ (8.20)

(b) Pertinent reinforcement should be placed on the surfaces of the panel as required for internal forces (N, M_{nt}).

In (8.20) the factors are as follows:

γ_{n1} = 1,2 modifying factor against brittle breaking

σ_c = normal stresses on horizontal section of panel due to vertical design loads.

ϕ = $\phi(e_o/b_w, \bar{\lambda})$, buckling function of reduced slenderness $\bar{\lambda}$ and e_o/b_w

f_{cd} = design concrete stress of shear-wall panel wehre 0.75 f_{ck}/γ_{cm}

f_{ck} = characteristic streght of panel concrete

γ_{cm} = 1.4

$\bar{\lambda}$ = the reduced slenderness

$$\bar{\lambda} = k / b_w \sqrt{\alpha} \tag{8.21}$$

where

$$\alpha = E_c / 1.6 f_{cm} (1 + \beta\xi) \tag{8.22}$$

E_c = longitudinal elasticity modulus of concrete

f_{cm} = average compressive strength of concrete

ξ = N(permanent)/N(total)

β = creep coefficient

For B300

E_c = 30250 MPa, f_{ck} = 25.0 MPa , f_{cm} = 22.0 MPa

ξ = 0.84 as calculated in parag. 8.3.3.2. Thus α and $\bar{\lambda}$ using (8.21) and (8.22) respectively are as follows:

$$\alpha = \frac{32250}{1.6x22(1+1.2x0.84)} = 425, \quad \sqrt{\alpha} = 20.6$$

$$\bar{\lambda} = 0.71x2.70/(0.16x20.6) = 0.582$$

e_o/b_w is obtained as:

with sideway earthquake 14.4/160 = 0.09
without sideway earthquake 12.7/160 = 0.07

ϕ is from Fig. 8.6 with the above computed values of e_o/b_w and $\bar{\lambda}$:

with sideway earthquake = 0.50
without sideway earthquake = 0.53

σ_c using value of N_1

$$\sigma_c = \frac{N}{1xb_w} = \frac{305.1 \times 1000}{1000 \times 160} = 1.97 \text{ MPa}$$

The condition (8.20) is satisfied with the above values:

$$1.2x1.97 = 2.288 \text{ MPa} < 0.50x0.75 \frac{25}{1.4} = 6.70 \text{ MPa}$$

The reinforcement required for (M_{nt}, N):

$$\max e_o = 14.5 \text{ mm} < b_w/6 = 160/6 = 26.6 \text{ mm}$$

Therefore, it is not necessary to reinforce the side surfaces of panel.

8.4.1.2. <u>In Shear-Wall no Diagonal Crack should exist</u>

In order to prevent diagonal cracks in shear-wall panels, tension stress σ_1 at center of gravity of panel section should be less than design value of concrete tension strength also when the amplification factor for shear force is taken as 1,5, which can be expressed as follows:

$$\gamma_{n1}\, \sigma_1 \leq \frac{0.75\ f_{ck}}{1.4 \times 12.5} \tag{8.23}$$

σ_1 = normal stress σ_c and tensile stress due to τ_{gf} shear stress calculated using 1,5 times shear force at center of gravity of section.

σ_c = vertical stress due to permanent loads.

γ_{n1}, f_{ck} are defined in the preceeding paragraph.

From Table 8.2. using N=1545 kN at second storey:

$$\sigma_c = 1545/1.325 = 1.144 \text{ MPa}$$

And from τ_f diagram drawn using an amplifying factor of 1.5 for shear force as in Fig.8.5;

$$\tau_{gf} = 1.125 \text{ MPa}$$

and σ_1;

$$\sigma_1 = \frac{1}{2}\sigma_c - \sqrt{\left(\frac{\sigma_c}{2}\right)^2 + \tau_{gf}^2} = \frac{1.144}{2} - \sqrt{\frac{1.144^2}{4} + 1.125^2} = -0.69 \text{ MPa} \tag{8.24}$$

The condition (8.23) is satisfied as follows:

$$1.2\text{x}0.69 = 0.828 < \frac{0.75 \text{ x } 250}{1.4 \text{ x } 12} = 1.12 \text{ MPa}$$

8.4.1.3. Tensile Stresses due to Earthquake Effects parallel to Shear-Wall Plane should be resisted by Steel Reinforcement

In the shear-wall panel P6, selected as example, from the diagrams in Fig. 8.5 for vertical stresses due to earthquake loads in (y-y) axes at floor level in the first floor; when earthquake is in the direction of (y-y) axis at the left side on a width of 1.247 m,

$$T_\ell = \frac{1}{2} 1.497 \text{ x } 1.274 \text{ x } 160 = 152.5 \text{ kN}$$

and when earthquake is in the direction of -(y-y) axis at the right side on a width of 1.135 m,

$$T_r = \frac{1}{2} \text{ x } 1.332 \text{ x } 1.135 \text{ x } 160 = 120.9 \text{ kN}$$

tensile forces act on the above section. Application of 1.2 coefficient:

$$1.2\ T_\ell = 183 \text{ kN},\ 1.2\ T_r = 145 \text{ kN}$$

Each forces can be resisted by a bolt of a diameter d = 26 mm, f_{sd} = 400/1.15 = 347 MPa

Since;

$$A_s f_{sd} = 530.7 \text{ x } 347 = 184.1 \text{ kN} > 1.2\ T_\ell > 1.2\ T_r$$

The bolts are placed closer to the edge than the center of gravity of tension zone. In this case, it is apparent that the load on the bolts will be smaller in reality.

8.4.1.4. In a Shear-Wall Panel, the Ratio of the Area of vertical Reinforcement, placed at two Edges, to the Area of Panel Cross-Section should not be less than 0.05 %

The panel cross-section is:

$$4680 \times 160 = 748800 \text{ mm}^2$$

And the condition is satisfied as follows:

$$5 \times 10^{-4} \times 748800 = 374.4 \text{ mm}^2 < \frac{\pi d^2}{4} = \frac{\pi}{4} \times 26^2 = 530.7 \text{ mm}^2$$

8.4.2. Connections

8.4.2.1. Horizontal Connections

The horizontal connection of bottom edge of shear-wall panel P_6 will be calculated as an example. This connection is subconcreted with bottom of panel sections having castellations at 150 mm intervals. The depth of castellation is 13 mm with a 75 mm length in panel plane and 130 mm length along thickness of panel (See Fig.8.7).

(a) Safety against shear should be maintained, thus the condition stated below should be satisfied:

$$\gamma_{nl} \cdot V_p \leq n.c.a. \frac{0.75\, f_{ck}}{1.4} + 0.8\, A^*_{sv} \frac{f_{yk}}{1.15} \tag{8.25}$$

where

$V_p = A_{\tau p}\, b_w$

$A_{\tau p}$ = The resultant value obtained, when the area of the shearing stress diagram, τ_f (drawn with an amplification factor 1.5) is decreased by substituting ($\tau_f - 0.3\, \sigma_c \geq 0$) for τ_f. This substraction, due to friction is possible in the region of the graph where σ_c is compression only.

b_w = thickness of panel

n = number of ribs or sockets at the bottom edge of panel at the connection.

a, c = depth and width of rib or socket (see Fig.8.7)

f_{yk} = characteristic strength of steel

At bottom section of Panel P_6:

$A_{\tau p} = 271.5$ N/mm, $b_w = 160$ mm

$n = 4680/150 \simeq 30$, $c \simeq 13$ mm, $a = 130$ mm

$f_{ck} = 25$ MPa, $A^*_{sv} = 0$.

V_p using (8.65);

$$V_p = 271.5 \times 160 = 434.4 \text{ kN}$$

The condition of (8.25) is satisfied as;

$$1.2 \times 434.4 = 512.3 \text{ kN} < 30 \times 13 \times 130 \times \frac{0.75 \times 25}{1.4} = 679.0 \text{ kN}$$

The sum of vertical components of compressive forces acting on the sloping edges of castellations;

$$1.5\ V = 1.5 \times 362 = 543\ \text{kN}$$

which is in equilibrium with vertical compressive force N = 1766 kN in the panel.

(b) Safety of connection against compressive stresses should be maintained; therefore the following condition should be satisfied;

$$\gamma_{nl}\ \sigma^*_{cmax} \leq k_v \frac{0.75\ f_{ck}}{1.4} \qquad (8.26)$$

where

σ^*_{cmax} = maximum concrete compressive stress due to initial eccentricities under vertical load and earthquake load parallel to wall panel.

k_v = coefficient dependent on slab support type and width of peripheral tie element. Its value is 0.7 when the floors penetrate into the connection as a normal feature along their length, and the mechanical continuity between panels and floors is established by appropriate arrangement of reinforcement subject to the condition that the properties of connection concrete is comparable to those of concrete in panels.

e_t being total initial eccentricity and σ_{cmax} maximum concrete strength due to vertical and earthquake loads, than σ_{cmax} can be expressed as;

$$\sigma^*_{cmax} = \sigma^*_{cmax}\ (1 + \frac{6\ e_t}{b_w}) \qquad (8.27)$$

From Fig. 8.5, in the subject.connection

$$\sigma^*_{cmax} = 4.16\ \text{MPa}$$

From (8.29) e_t = 12.7 mm

From (8.27) σ^*_{cmax} can be calculated as;

$$\sigma^*_{cmax} = 4.16\ (1 + \frac{6 \times 12.7}{160}) = 6.14\ \text{MPa}$$

And the condition (8.26) is checked to be satisfied as follows:

$$1.2 \times 6.14 = 7.37 < 0.70 \times 0.75 \times 25 / 1.40 = 9.37\ \text{MPa}$$

8.4.2.2. Vertical Connections

The vertical connection between I. and II. part of shear-wall Panel P_6 at the first floor will be calculated as an example.

(a) Safety against shear should be maintained and thus the panels at the connection should have castellations and the following condition should be satisfied:

$$\gamma_{nl}\tau_{jf} < \frac{A_k}{b_w \ell} \cdot \frac{0.1\ f_{ck}}{1.5} + \mu\ \frac{N_d + As\ f_{yk}/1.15}{b_w \ell} \qquad (8.28)$$

where

τ_{jf} = shear stress computed at point (j) of vertical connection at the horizontal bottom section of panel using horizontal shear force with a factor $\gamma_{n2} = 1,5$

$A_k = \Sigma b_c \ell_n$

ℓ_n = length of socket of castellations (see Fig.8.7)

b_c = width of socket of castellations

μ = friction coefficient (= 0.8)

N_d = compressive force acting on Connection

A_s = total area of reinforcement passing through perpendicular to the connection.

ℓ, b_w = length and thickness of panel

(b) Tensile forces acting on the joint should be resisted by horizontal reinforcement. This reinforcement as mentioned in (8.28) can be calculated with the following expression:

$$A_s = \frac{b_w \ell \tau_{jf}}{f_{yk}/1.15} \tag{8.29}$$

In plastic state:

$$b_w \ell \tau_{jf} \leq 1.1 A_s \frac{f_{jk}}{1.15} \tag{8.30}$$

In the subject connection, τ_{jf} shall be calculated using the following expression:

$$\tau_{jf} = \frac{1.5 V S_j}{b_w I_c} \tag{8.31}$$

From Table 8.2 and Fig.8.3:

$$V = 362 \text{ kN}, \; I_c = 2.4216 \text{ m}^4$$

$$S_j = 0.16 \times 1.52 \, (2.41 - \frac{1.52}{2}) = 0.4013 \text{ m}^3$$

And τ_{jf} from (8.31):

$$\tau_{jf} = \frac{1.5 \times 362 \times 0.4013}{0.16 \times 2.4216 \times 10^3} = 0.562 \text{ MPa}$$

Using $\ell = 2.70$ m (See Fig.8.3) A_s from (8.29):

$$A_s = \frac{160 \times 2700 \times 0.562}{400/1.15} = 698.4 \text{ mm}^2 = 6.98 \text{ cm}^2$$

This will mean placing 12 hooks each ø6 bar.

In Fig.8.7, it can be seen that the edges of the panels at the vertical connection have 12 sockets. The lengths of each socket is $\ell_n = 70$ mm and the width in the direction of panel thickness is $b_c = 90$ mm.

The condition (8.28) with the above values is checked to be satisfied as follows:

$$1.2x0.259 = 0.31 \text{ MPa} < \frac{12x70x30x0.1x25}{160 \text{ x } 2700 \text{ x } 1.5} + 0.8 \frac{322x400}{160x2700x1.15} \text{ MPa}$$

$$0.31 \text{ MPa} < 0.29 + 0.21 = 0.49 \text{ MPa}$$

In this vertical connection minimum longitudinal reinforcement of 4ø14 bars shall be placed.

8.5. SELECTED DETAILS

In Fig. 8.8 the horizontal and vertical sections and castellations of a slab panel, in Fig.8.9 the same sections of a wall panel with door openings and its border castellations are shown.

Accordingly in Fig.8.10 and Fig.8.11 vertical connections of two outside and one inside bearing walls and two outside and two inside bearing walls, in Fig. 8.12 and Fig.8.13 vertical section of horizontal joints of two outside bearing walls and finally in Fig. 8.14 the different types of castellations in wall panels have been shown.

8.6. REMARKS

The outline of current design principles, and structural analysis of RC multi-storey earthquake resistant apartement houses built with precast large panels is illustrated in this design example.

1- The structure is in a seismic region with a final seismic coefficient of 0.08, and rests on a fairly thick sand and gravel layer. The foundation consisting of basement walls and footings is of continuous footing in both directions, and is rigid enough in its plane.

2- The connections between large panels are designed to ensure that the system formed by walls and floor panels behaves as a monolithic structure under horizontal and vertical loods.

3- Due to the rigid connections between the continuous shear-walls in two directions, gravity loads induce only axial compression on the shear-wall cross-section.

4- The total value of the horizontal earthquake shear force in the base of the building is obtained by using a semi-modal method.

5- In the structural design, the linear elastic theory has been used. So, there are no references to any mechanism associated with the postelastic deformation of the structure. To account for the reversibility of seismic loads and the increase in the forces, the shear forces have been multiplied by a factor of 1.5 in the proportioning of shear-walls and connections. The amplifications of flexural moments of shear-walls are obtained by substituting 0.15 E_c I_c for lintels flexural rigidity.

6- The strength of RC structural wall components originated from the condition that the maximum elastic stress does not exceed the appropriate design strength. Whether the coefficients and methods of amplification and the conservative assumptions will account for the internal forces during postelastic deformation, has not been firmly established yet. In addition, the actual strength of the connections when subjected to reversed loading cycles is another point to be clarified.

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NOTATIONS

Report		CEB
A_c	cross sectional area of concrete	A_c
A_s	cross sectional area of reinforcement	A_s
C	design seismic coefficient	C_d
C_o	earthquake zone coefficient	
E_c	longitudinal strain modulus of concrete	E_c
E_s	longitudinal strain modulus of reinforcement	E_s
E	earthquake load	F
G	permanent load	G
H	height of the building	h
I	a) second moment of area b) building importance factor	a,b)I
K	structural type factor	
M	flexural moment	M
M_t	torsional moment	M_t
N	a) normal (axial) force b) number of storeys	a)N
N_u	Euler critical load	N_k
P	vertical design load	P

Q	live load	Q
S	a) dynamic coefficient for the structure(spectral coefficient) b) first moment of area	b)S
T	natural period of the building	T
V	shear force	V
W_i	gravity load on floor i	W_i
a	depth of rib or socket	a
b	horizontal length of the panel	b
b_w	width of the panel	b_w
c	width of rib or socket	c
d	effective height	d
e	excentricity	e
f_c	concrete strength	f_c
f_y	steel yield strength	f_y
i	radius of gyration	i
k_v	coefficient without dimension	
l	height of the panel	l
l_n	length of the one shear key	l_k
m	uniformly distributed mass	m
x,y,z	coordinates	x,y,z
β	creep coefficient	ϕ
γ	safety factor	γ
λ	slenderness ratio	γ
μ	coefficient of friction	μ
σ	axial stress	σ
τ	shear stress	τ

SHEAR WALL P6 AND CROSS SECTIONS OF COUPLING BEAMS

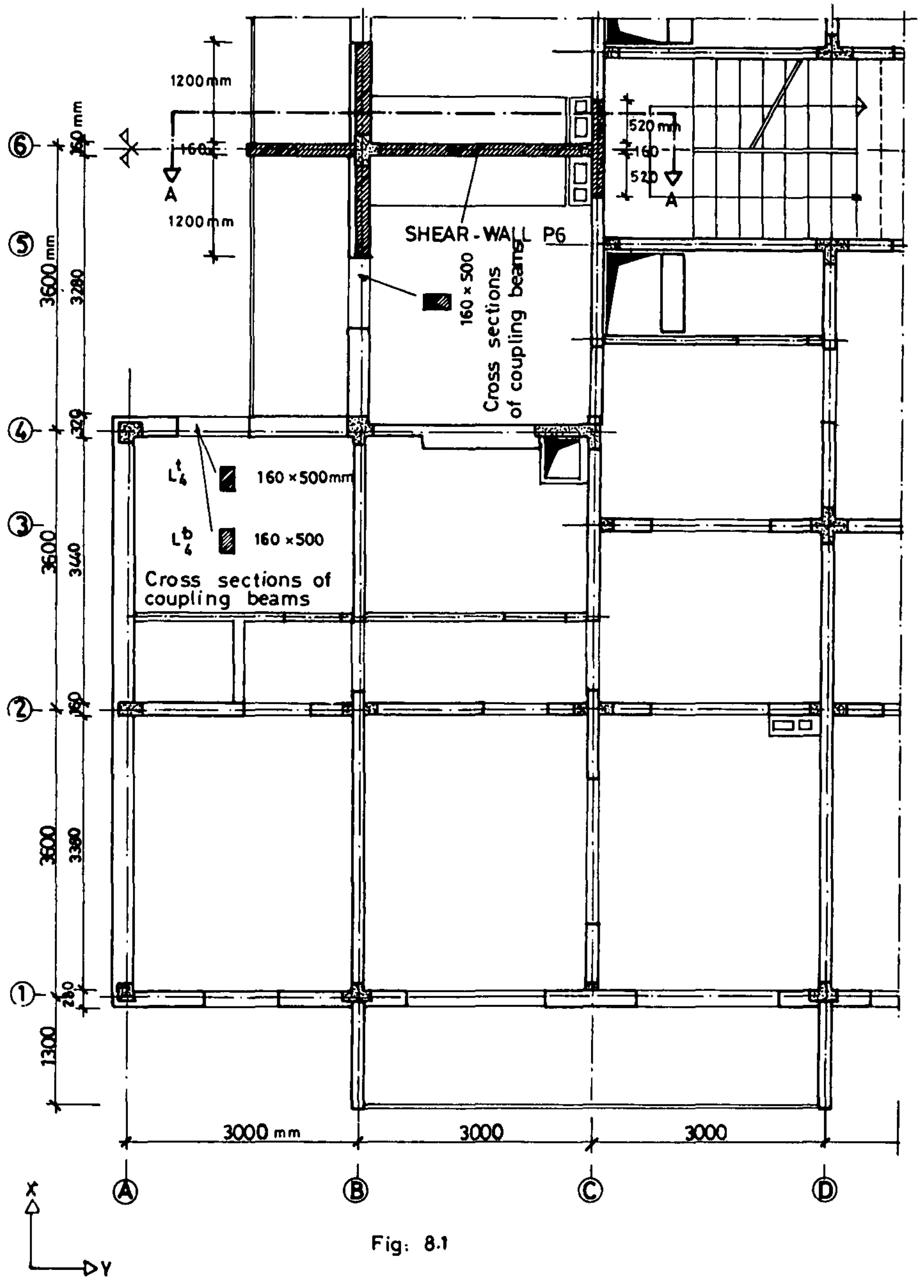
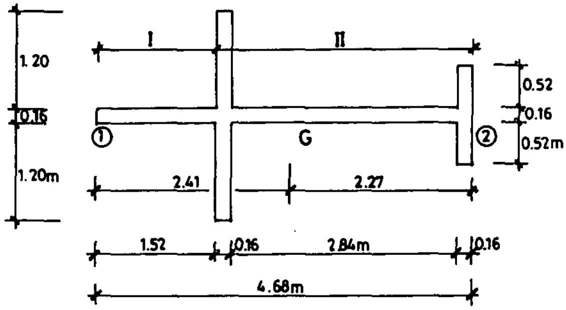
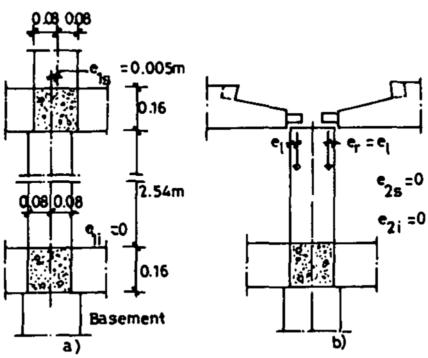


Fig: 8.1



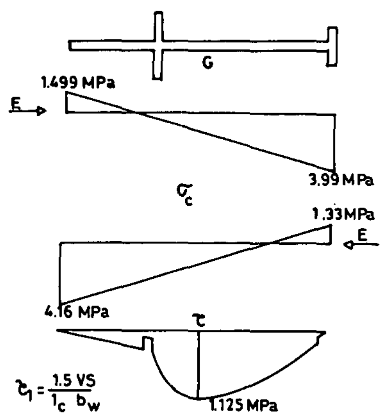
CROSS-SECTION OF SHEAR-WALL P6

Fig. 8.2



COMPUTATION OF ECCENTRICITIES e_1, e_2 OF SHEAR WALL P6

Fig. 8.3



NORMAL AND SHEAR STRESS DISTRIBUTIONS

Fig. 8.4

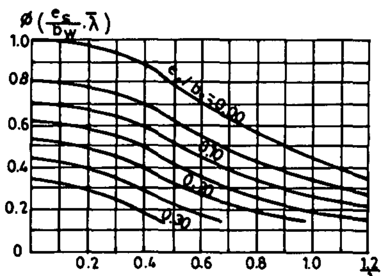
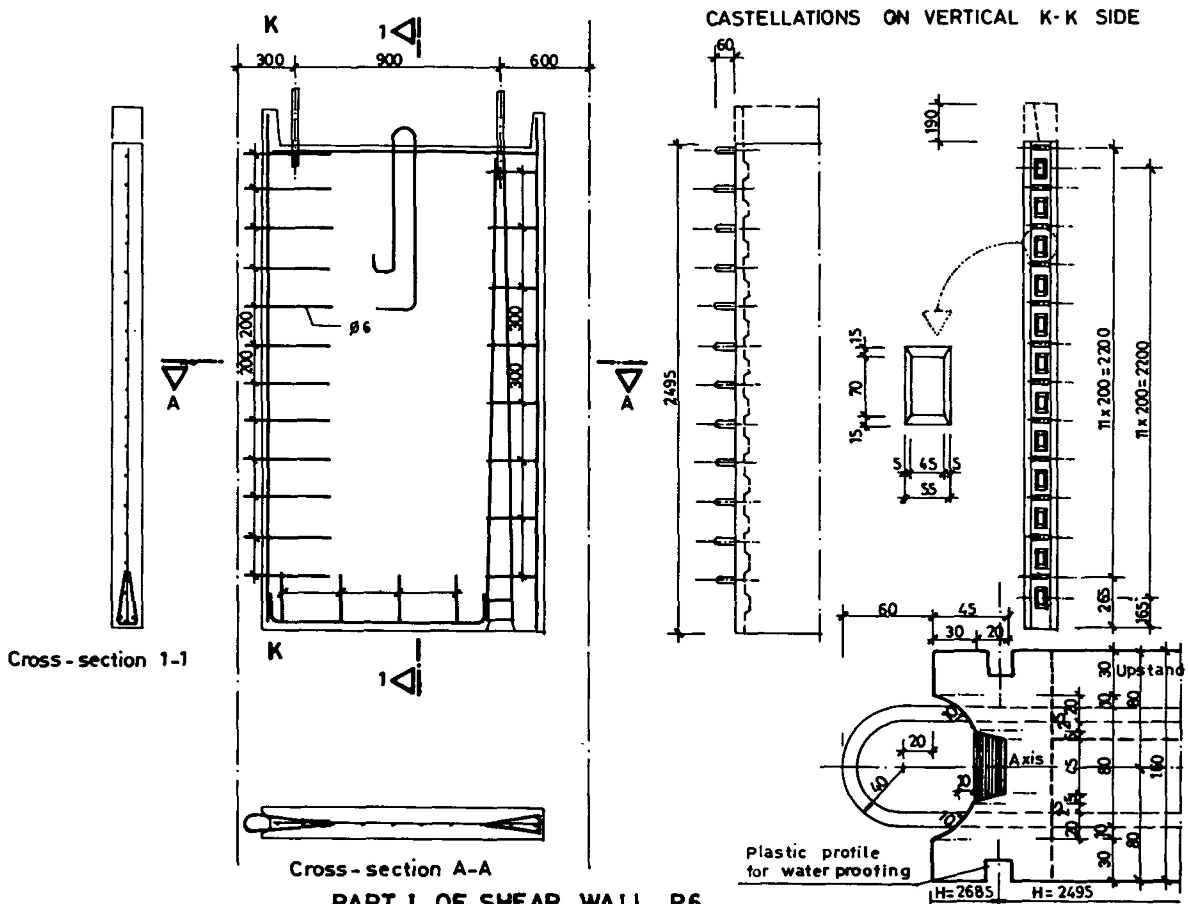
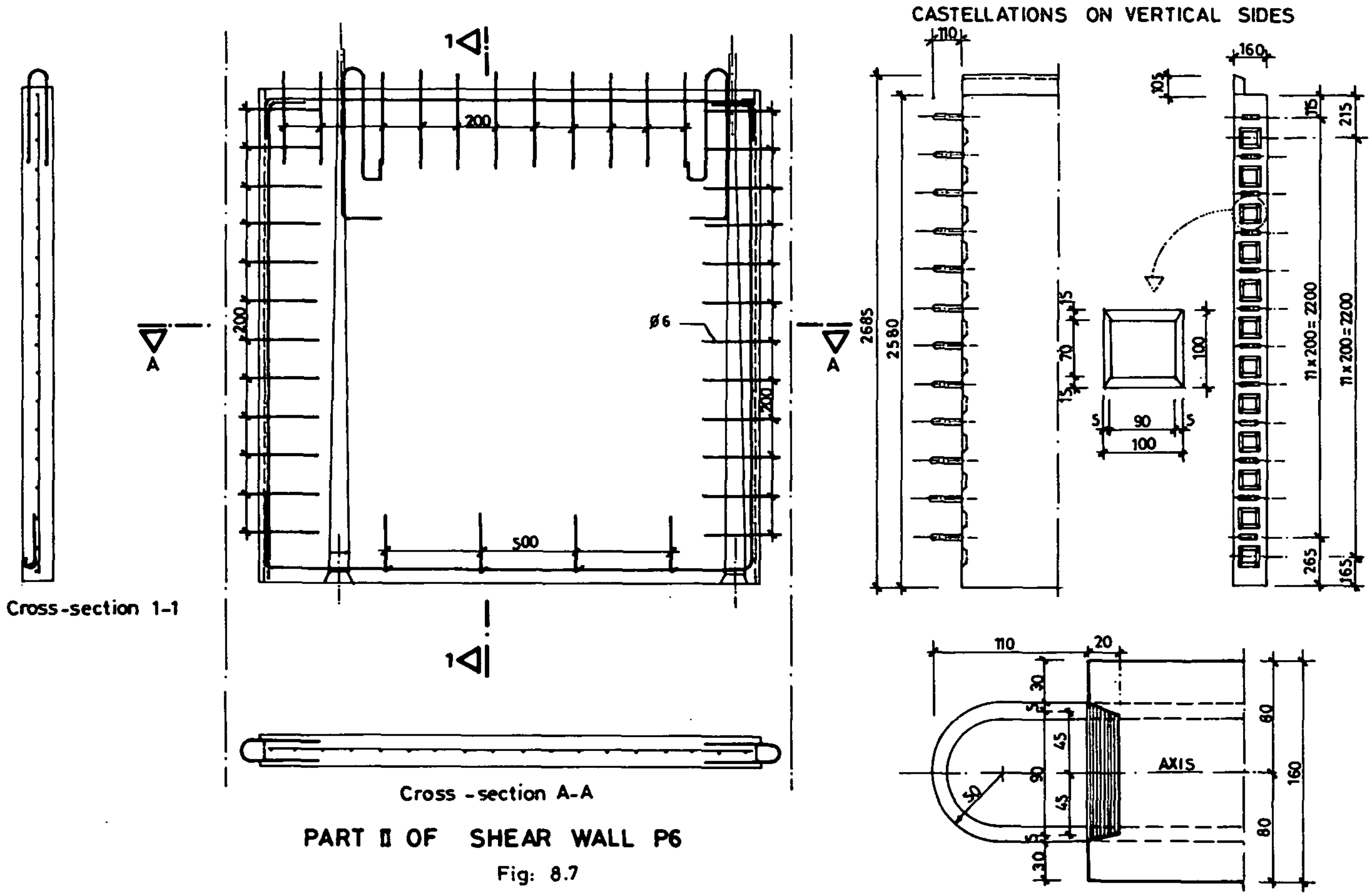


Fig. 8.5

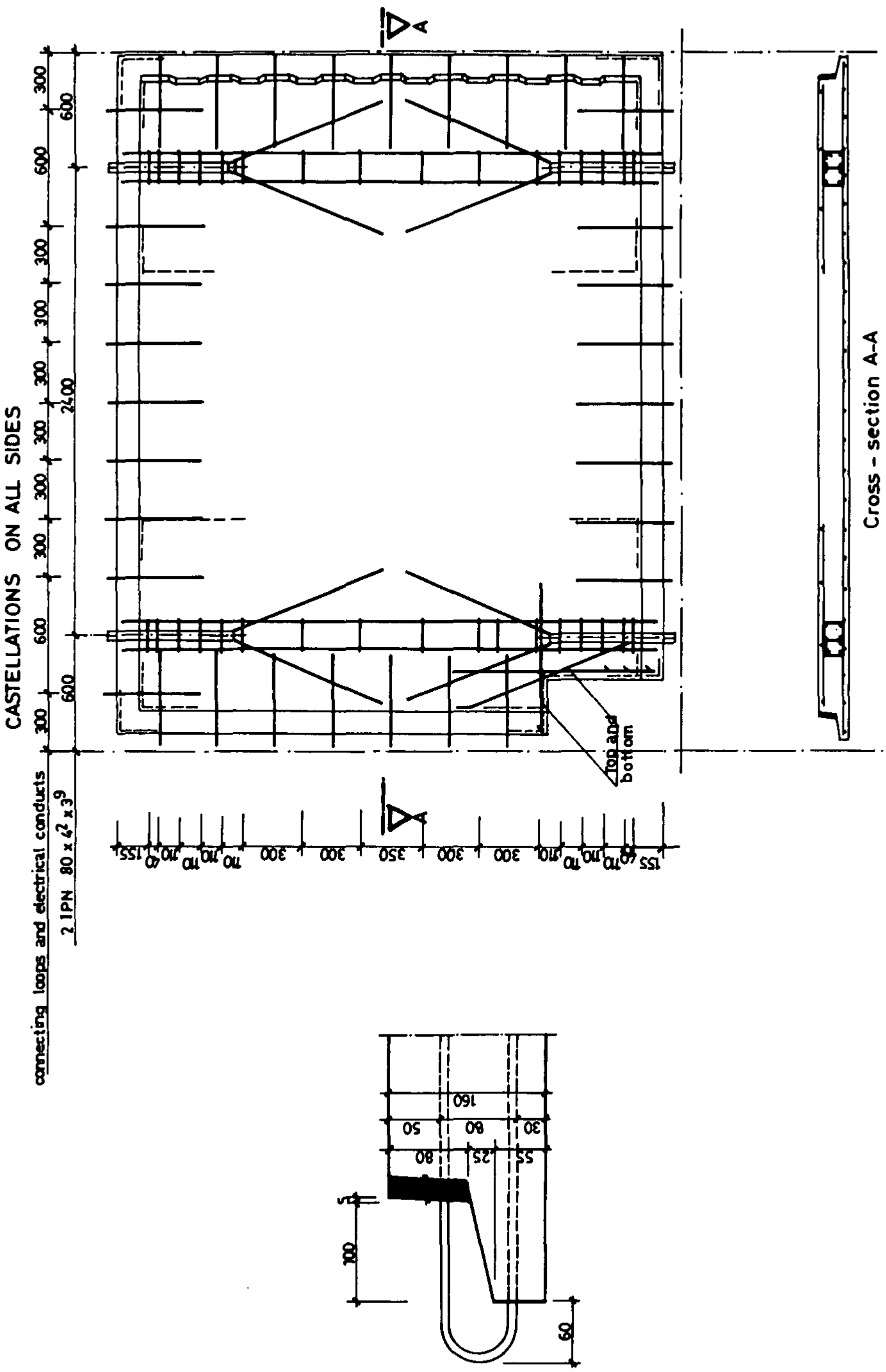


PART I OF SHEAR WALL P6
Fig: 8.6

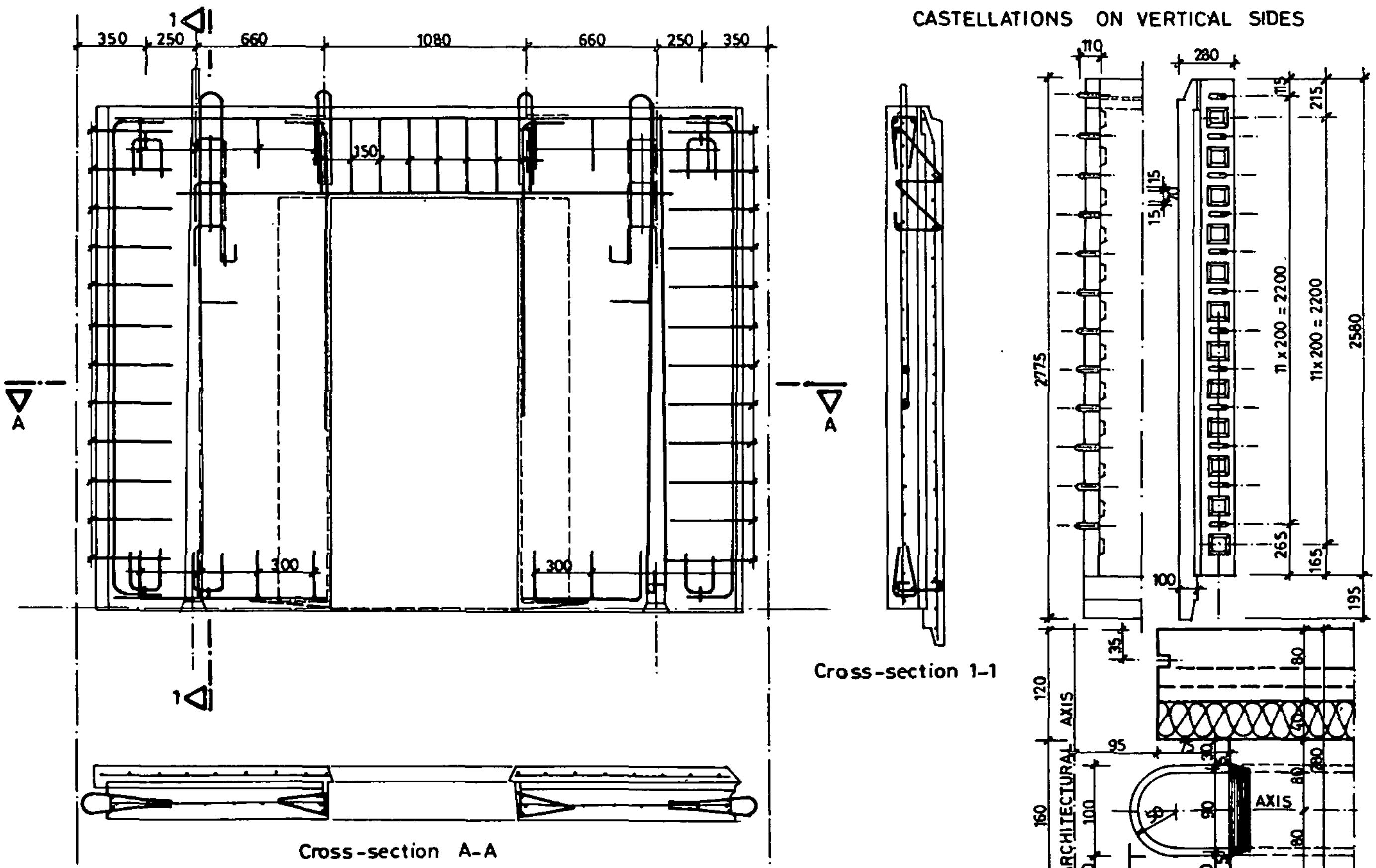


PART II OF SHEAR WALL P6

Fig: 8.7



A FLOOR PANEL
Fig: 8.8



A SHEAR WALL PANEL
Fig: 8.9

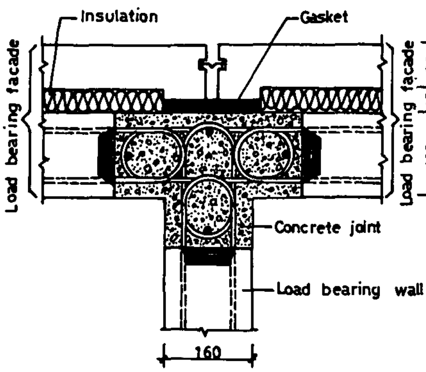


Fig. 8.10

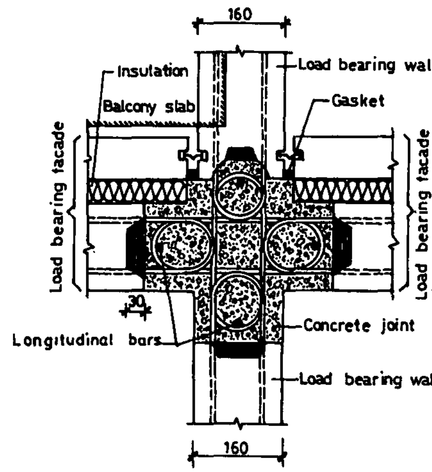


Fig. 8.11

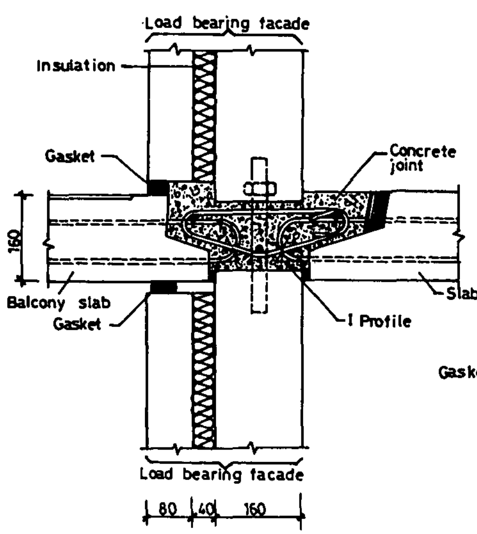


Fig. 8.12

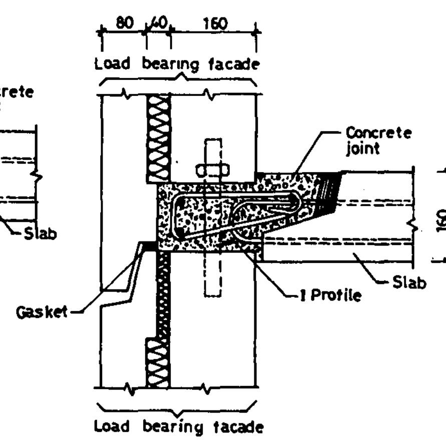
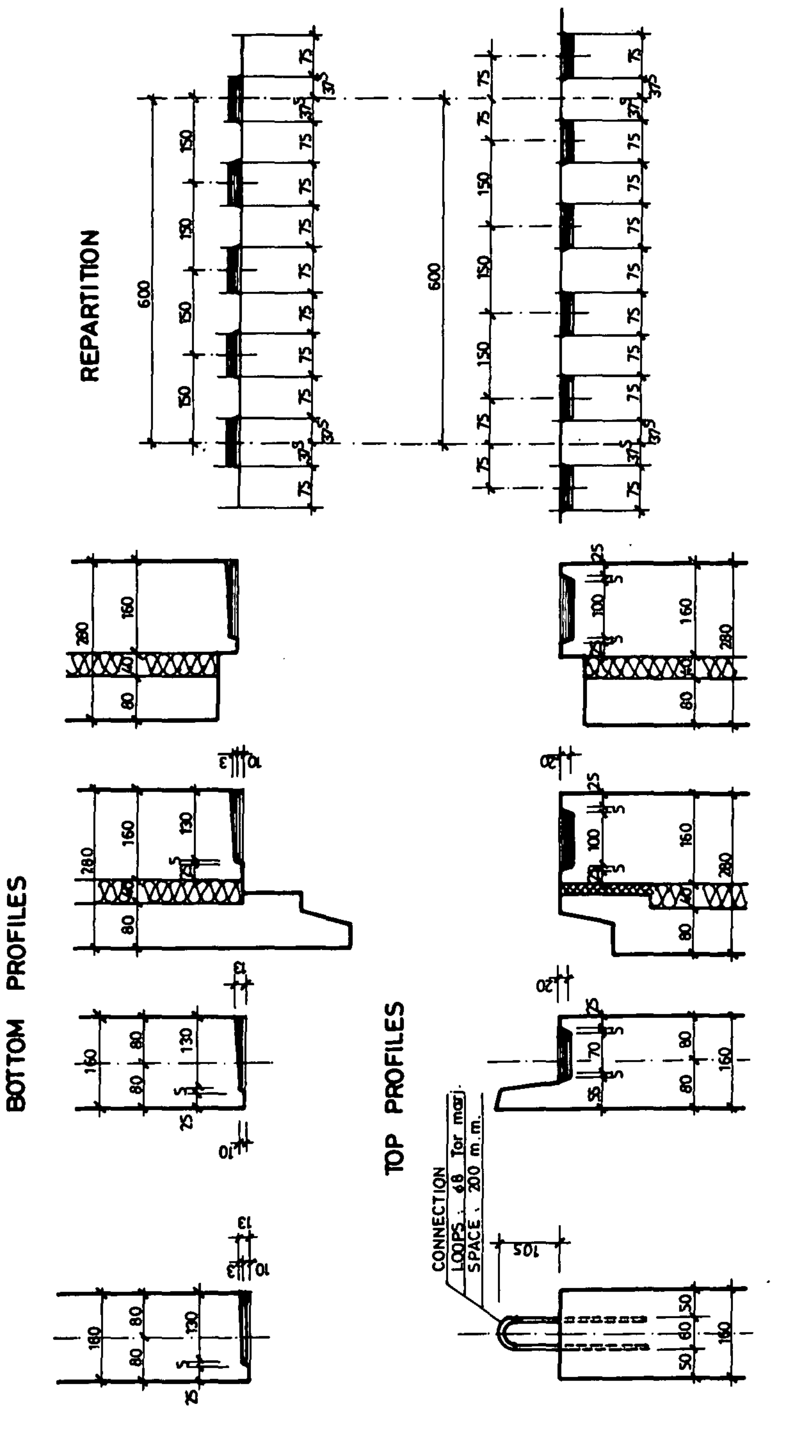


Fig. 8.13



CASTELLATIONS TYPE OF WALL PANELS
Fig: 8.14

9. REPRESENTATIVE EXAMPLE BUILDING SYSTEM YUGOSLAVIA

TABLE OF CONTENTS

9.1. SCOPE

The IMS System - frame structure consisting of prefabricated elements is the most widely applied building technology in Yugoslavia where industrialized construction is concerned. It is applied in the construction of residential buildings, schools, hospitals, office buildings, garages and light industry plants. The IMS System was developed at the Materials Testing Institute of the Socialist Republic of Serbia.from the idea of Prof.Branko Zezelj. In Yugoslavia the IMS System is applied by some fifteen building contractors.

The same technology is applied, to a greater or lesser extent, in Italy, Hungary, Austria, Cuba, Egypt, Angola, the P.R. of China, and the USSR; preparations are under way for introducing the technology in some other countries as well.

The descriptions provided here cover structural features of the system, architectural design considerations, the production of prefabricated units and erection procedures. The comparative advantages of the IMS System as comapared to conventional and panel systems are shown. The numerical example analyzes a 12-storey building with shear walls for earthquake force action, and lists examples of column, beam and wall proportioning. Sketches and photographs illustrate the concepts of the IMS System, and the appearance of a number of completed structures. The supplement includes a detailed description of results obtained by testing joints and structural units.

9.2. DESCRIPTION OF STRUCTURAL SYSTEM

9.2.1. Introduction

For the purpose of easier understanding of the concept of the IMS System, the elements can be divided into three categories:

- primary elements - load-bearing parts of the structure: columns,floor slabs, shear walls, etc.;
- secondary elements - nonload-bearing parts: cladding panels, partition walls, kitchen and bathroom units, etc.;
- tertiary elements: finishes, woodwork etc.

All the primary elements are exclusively typical of the IMS System; they are not fabricated for other systems, and the elements and joints are protected by patent and licence rights. The secondary and tertiary elements are not exclusively related to the IMS System, and can also be used in other building technologies.

The originality of the IMS System is to be found in the unique features of the prefabricated elements of the primary structure and their joints; i.e., namely, columns, shear walls and floor slabs are prestressed into a monolithic whole at each storey level. Such a system of element jointing gives IMS building technology a very high degree of safety and stability which is of particular importance for the construction of buildings in seismically active areas and areas subjected to the action of very strong winds - hurricanes. This has been proved not only by experimental and theoretical check-ups, but also by the behaviour of numerous buildings during the 1969 and 1981 earthquakes in Banja Luka.

The salient feature of the IMS building technology is that a relatively small number of industrially manufactured elements can be used in order to build a frame structure for buildings of different purposes. The versatility of the system is also evinced by the possible use of a variety of subsystems - both with regard to technology and to materials. Thus, different types of facades, partition walls, kitchen and bathroom units, etc., can be applied.

Element transport is economical up to no less than 100 km, and can be effected by standard facilities.

The most widely applied span in residential construction is 4.20 x 4.20 m.

9.2.2. IMS System - Design Considerations

The prefabricated prestressed concrete frame system presents all features of a monolithic frame structure; as such, it offers designers a very broad scope of freedom in selecting the plan of the building, a point which is rather exceptional when prefabricated structural systems are concerned. The arrangement of structural elements is mainly controlled by architectural and functional considerations. The system involves very few constraints due to either the structural concept or to building technology. The basic consideration to be followed when designing and IMS building is adjustment of the configuration to design and zoning ordinance requirements.In structural terms, symetry in both directions is desirable but not an imperative. The arrangement of partition walls and stairacases may be asymmetrical; however, in seismic activity zones the distribution of slabs and shear walls should be as symmetrical as possible.

The height of the building is not specifically limited by the prefabricated system; the height of buildings built by applying the IMS prefabricated frame system is governed by the same parameters ruling buildings erected with a monolithic reinforced concrete frame. So far buildings of 18 to 20 storeys have been built with standard columns and spans.The highest project built by applying the IMS System to date is a 26-storey building in Pecs, Hungary.

In the system under consideration, joints are governed by the same basic rules applied in the construction of conventional, reinforced concrete frame buildings. The positioning of expansion joints depends on the soil, the form of the building, and its length. The longest building erected to date runs to about 80 m. The building can by supported on individual foundations, slabs of piles - depending on soil features. So far numerous buildings for a variety of purposes and having a varying number of storeys have been built over the 25 years of IMS System application. Of particular importance is the high flexibility of the system in living space design, in line with the latest trends of functional layout, made possible by the lightweight and easily movable partition walls.

9.2.3. The Structural System

The IMS structural system consists of the following elements:

- columns (one-, two- and three-storey height);
- floor slabs (standard, with openings and cantilever);
- edge girders;
- shear walls for horizontal forces;
- staircases.

Columns and floor slabs are connected into a monolithic whole by pretensioning cables passed through openings in the columns (at floor slab level) and through the duct formed by two adjacent slabs. Every storey level is prestressed by a cable system in two orthogonal directions, pretensioned straight or polygonally for purposes of greater steel economy. After prestressing, all the cable ducts between the slabs are filled with concrete, and the interstorey structure thus becomes adhesively prestressed for live load.

Whereas the columns only take vertical loads, the influences due to horizontal forces are partly taken by the frame, and mainly by special shear walls positioned at optimum points in the plan, between columns, in order to obtain the wanted stiffness of the structure. Shear walls are precast with the exception of special situations (very high structures or zones of a high seismic zone) when they are cast in situ.

The IMS System offers an extensive range of structural spans (from 3 to 9 m either way in 0.60 m steps). Depending on weight and size, floor slabs are fabricated in three thicknesses 0.22, 0.30 and 0.36 m. Moreover, slabs of greater spans are fabricated in two or three segments connected by pretensioning cables. The slabs are provided with or without ceiling finishes; slabs with adequate openings for installations, stairwells or elevators are also produced.

Cantilever floor slabs are of standard overhang (up to 3 m, depending on column spacing); edge slabs with no cantilever require edge girders, which also from the ducts for pretensioning cables. In most cases the columns are three-storey ones of square cross section and standard dimensions, depending on the span and height of the project. Columns are continued by inserting anchoring reinforcement from one column into openings left in the other column. Edge girders are of solid cross section and carry the cladding panels.

Shear walls are precast reinforced concrete panels, 0.15 m thick, with vertical openings, for the reinforcement taking horizontal forces, which are subsequently concreted. The wall thus becomes a monolithic foundation-to-roof cantilever.

Standard IMS staircases are single, double or triple-flight, and consist of girders and stairs (mounted sebsequently).

Like any frame system, the IMS System lends itself to the use of an almost unlimited variety of precast cladding elements. Facade walls can also be of traditional materials, e.g., brick etc. Concrete sandwich panels - two concrete layers with a thermal insulator in-between- have been the most popular so far. Panels which fully fill the space between the columns and floor slabs have also been used; in the early days of IMS System application, horizontal parapet-type facade elements were very popular. Full-face elements can be provided with any type of opening. An extensive variety of balconies, loggias and combinations thereof is also applied. Cladding panels and edge girders are joind by steel anchor bolts poured in situ.

Partition walls are available in numerous variants, and their positioning is not governed by any structural considerations. Shear walls usually also serve as dividing walls between apartments or between apartments and the stairwell. At an ever increasing rate the system uses prefabricated walls incorporating all instalations. Complete kitchen and bathroom units, with walls consisting (usually) of thin concrete slabs, are also used.

9.2.4. Building Construction

The safety and strength of joints of an industrialized building system are certainly the main factors determining its quality. The IMS System is rendered monolithic by prestressing:the floor slabs and columns are held together by forces induced at contact surfaces. The IMS building system applies the IMS Prestressing System, which uses cables made mostly of ϕ 5 or 7 mm steel wire. The pretensioned cables run in two orthogonal directions through the columns and ducts formed by two adjoining floor slabs. The number of cables (and wires) depends on the structural span, loading and cable trajectories. Linear cable configurations are economical up to a span of 5 meters, while polygonal configurations are in principle more rational. The number of cables varies from 2 to 4, a linear pattern requiring more cables.

9.2.5. Building Behaviour During Earthquakes

It should be noted that 17 IMS System buildings underwent two severe earthquakes in Banja Luka (Yugoslavia) in 1969, assessed at 8 degrees on the MCS Scale. The buildings ranged from five to thirteen storeys. As this region was not considered to be a seismic area up to then, the buildings did not have any special design features for withstanding unusually strong horizontal forces. Nevertheless, they all survived the earthquakes without any severe damage. This "full-scale experiment" opened the way to other IMS System applications in seismic areas of Yugoslavia and many other countries. Damage occured on secondary elements (partition walls) and shear walls, mainly in zones of poorly poured working joints.

9.3. STRUCTURAL ANALYSIS

9.3.1. Design Loads and Computation Technique

The influence of an earthquake on an idealized IMS-type building with 3x5 bays is analysed (Fig.9.1). Only the y-direction is considered here. The columns are spaced by 4.20x4.20 m and the building has 12 storeys. Four shear walls were designed in each direction. The building was designed for the IX seismic zone according to Yugoslav 1981. earthquake regulations.

Total base shear

$S = K \cdot Q$

$K = K_o \cdot K_s \cdot K_p \cdot K_d$

$K_o = K_p = 1.0 \qquad K_d = 0.70/T = 0.70/0.854 = 0.820$

$K_s = 0.10$ (see Code, Vol.7)

Category factor $K_o=1.0$ for residential buildings. Plasticity and damping factor $K_p=1.0$ for RC buildings. Seismic factor $K_s=0.1$ for the IX^o MCS scale. Dynamic factor $K_d=0.70/T$, for IInd category of soil. The first mode period was calculated in the frame of computer program for the analytical model represented in Fig. 9.2.

$S= 1.0 \cdot 0.1 \cdot 1.0 \cdot 0.820\ Q = 0.082\ Q$

The mass was calculated and reaches m=0.7 t/m^2 or 185 t/storey.

$S= 0.082 \cdot 12 \cdot 185.22 = 1822$ kN

The modulus of elasticity taken into account was

$E= 3.8 \cdot 10^7$ kN/m^2

9.3.2. Computation Technique

Distribution of seismic force along the building height.
Concentrated force at the building top: $S_t = 0.15\ S = 273$ kN. The remaining portion (85 percent) of the seismic force should be distributed according to the formula:

$$S_i = 0.85 \cdot S \cdot \frac{Q_i\ h_i}{\sum_{i=1}^{12} Q_i h_i}$$

Calculation results are given in table 1.

TABLE 1. Total earthquake forces

Storey	Force in kN
12	513
11	220
10	200
9	179
8	159
7	139
6	119
5	99
4	79
3	58
2	38
1	18

Internal forces are then calculated by computer program for the analytical model shown in Fig. 9.2. Both pairs of shear walls are represented by one pair having the stiffness equal to the sum of two pairs (2xA). Four frames are represented by a single frame having the total stiffness equal to 4xB. Shear wall and frame are interconnected by a dummy (zero stiffness) beam.

Computer output gives all internal forces for all members. Only extreme and selected values are given here.

TABLE 2. Bending moments in beams (kN.m)

Storey	Row	Moment left	Moment right
12	1	114.6	114.6
12	2	0	0
12	3	23.0	21.8
12	4	20.6	20.6
12	5	21.8	23.0
6	1	99.3	99.3
6	2	0	0
6	3	22.6	22.1
6	4	21.6	21.6
6	5	22.1	22.6
1	1	22.8	22.8
1	2	0	0
1	3	5.2	5.1
1	4	5.0	5.0
1	5	5.1	5.2

TABLE 3. Bending moments in walls and columns (kN.m)

Storey	Row	Top moment	Bottom moment
12	1,2	- 114.6	- 19.9
12	3,6	- 23.0	- 16.0
12	4,5	- 42.3	- 30.6
6	1,2	2451	- 3393
4	1,2	4245	- 5355
2	1,2	6402	- 7623
1	1,2	7600	- 8720
1	3,6	4.8	- 8.5
1	4,5	3.1	- 9.4

TABLE 4. Shear forces in beams (kN)

Storey	Row	Force
12	1	- 27.3
12	2	0.0
12	3,5	- 10.7
12	4	- 9.8
6	1	- 23.6
6	3,4,5	- 10.6
1	1	- 5.4
1	3,4,5	- 2.5

TABLE 5. Shear forces in columns (kN)

Storey	Row	Force
12	1,2	48.0
12	3,6	13.9
12	4,5	26.1
6	1,2	336.3
6	3,6	8.0
6	4,5	15.2
1	1,2	448.0
1	3,6	1.5
1	4,5	2.5

TABLE 6. Normal forces in columns (kN)

Storey	Row	Force
1	1	255.2
1	2	-255.2
1	3	113.5
1	4	- 4.4
1	5	4.4
1	6	-113.5

TABLE 7. Horizontal deflections in mm

Storey	Deflection
12	33.6
10	26.0
8	18.5
6	11.5
4	5.6
2	1.5
1	0.3

The building deflection reaches H/1000 and the storey drift h/740.

9.4. PROPORTIONING

In principle, verifications on IMS buildings involve load-bearing capacity, and stresses, rather than structural design for given internal forces.

9.4.1. Column verification

The dimensions of the column are 0.34 x 0.34 m and 4∅22 C 400/500 N/mm^2 reinforcement.

Load-bearing capacity is checked through the ultimate load-bearing capacity line (interaction N-M diagram).

The calculation conforms to the provisions of the Code for Concrete and Reinforced Concrete (PBAB):

- compressive strength of concrete M=45 MPa; strength of prism β_{pr}=0.7x4500= =31.50 MPa;
- concrete working diagram: square parabola;
- ultimate compression of concrete ε_{su}=0.0035
- yield stress in tension of steel σ_{vi}=400 MPa;
- steel working diagram: elasto-plastic
- elongation limit of steel ε_{su}= 0.010.

The diagram shown in Fig. 9.3. has been obtained by applying equilibrium conditions and Bernoulli's hypothesis of planer cross sections.

External columns in frames (rows 3 and 6, Fig.9.2.) at the first storey are loaded by - axial force $N=741 \pm 113.5 = 855$ (627) kN, bending moment $M= \pm 15 \pm 8.5 = 23.5$ (6.5) kNm.

Point A and B in Fig. 9.3. represents the N-M relationship when earthquake acts from left side, and points C and D when it acts from right side.

Internal columns in frames (rows 4 and 5, Fig. 9.2) at the first storey are loaded by - axial force $N= 1482 \pm 4.4 = 1486$ kN, bending moment $M= \pm 0 \pm 9.4 = 9.4$ kNm.
Point E in Fig. 9.3 represents the N-M relationship for internal columns.

It is obvious that the columns have enough safety to the ultimate earthquake forces.

9.4.2. Shear wall verification

Shear wall cross-section is shown in Fig. 9.4. where also interaction diagram is shown. Material properties are same as for columns. Shear wall is loaded at the first storey by - axial force $N= 350 \pm 255 = 605$ (95)kN, bending moment M=8720 kNm, shear force T = 448 kN.

Points A and B in Fig. 9.4. represents N-M relationship for shear wall. The obtaind safety to the ultimate bearing capacity is K=1.33.

Shear force is tranferred in web (wall) by welded fabric 2xØ8/20 cm. In a oblique crack (45^o) there is 20x2Ø8 = 20.1 cm^2 of reinforcement. The ultimate bearing capacity of this reinforcement is T_u=20.1 . 40 = 804 kN. The achieved safety to the ultimate bearing capacity is K=804/448 = 1.80 $>$ 1.33. It is, therefore, the realistic assumption that bending failure will occur first.

9.4.3. Beam column joint

Structural design no longer involves this joint. Considering that this is always the same standard joint for a 4.20 x 4.20 m slab, joined by 4x6 Ø 5 mm cables of 1500/1700 MPa strength, results of a series of tests are used, showing

a) that the joint assumes, under full constant load, an additional (minimum) 50 kNm moment due to horizontal forces until the appearance of the first elastic crack;

b) that the cracks remain elastic, i.e., that they close completely when action is discontinued up to an approximate joint rotation of φ=0.006;

c) that the highest possible moment occurring averagely in the joint is M= 80 kNm;

d) that a hinge is formed in the joint, following the said action, and that the moment in the joint does not increase for rotation in excess of φ= =0.020;

e) that no joint failure or vertical sliding occurs at a joint rotation of φ = 0.05.

Fig. 9.4.A shows beam - column joint M-Ø relationship. Cheque for most loaded joint (see table 2, storey 12, row 1). Bending moment M = 114.6 kNm reffers to the shear wall vertical axis. At the wall-beam joints bending moment reaches M=0.5 . 114.6 = 57.3 kNm. Some cracks will open, but the obtained safety to the ultimate bearing capacity is still K=80/57.3 =1.40.

Cheque for shear force transfer in joint. One joint surface should take from vertical load T_g= 12.5 kN. If prestressing force reaches N= 400 kN/joint, then friction force R= 0.7 . 400 = 280 kN, or 140 kN per one joint surface. Achieved safety is then K=140/12.5 = 11.2. No dowell action of prestressing cables is taken into account. In the case of earthquake, at the most loaded beam at the 12th storey, shear force reaches T_e = M/0.5 L = 57.3/0.5 . 3.82= = 14.97 kN. Total shear force is then $T=T_g+T_e$=12.5 + 14.97 = 27.5 kN and the achieved joint safety K=140/27.5 = 5.09.

9.5. SELECTED DETAILS

The selected details shown in Figs. 9.5 through 9.27 illustrate the structural idea underlying the IMS System, details of joints, and a number of architectural designs. Fig. 9.5 and 9.6 shows the structural concept in axonometric terms, Provisional floor slab supporting and the form of the collar beam are shown in Fig. 9.7. Fig. 9.8 illustrates the pretensioning forces and floor slab weight forces acting on the column. The floor plan of a typical office building is shown in 9.9. Figs. 9.10 through 9.21 illustrate typical structural units, connection details, and several designs of residential buildings. Figs. 9.22 through 9.27 shows the completed projects.

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Notation

Symbols	Explanation	CEB Symbols
K_c	Seismic coefficient	C_d
M	bending moment	M
N	axial force	N
Q_i	Mass at the storey "i"	
S_i	Seismic force at the storey "i"	
T	Transversal force	V
β_i	Spectral coefficient	ω
β_{pr}	strength of prism	$0.7\ f'_c$
K	safety coefficient	
ε_{cu}	concrete ultimate strain	ε_c
ε_{su}	steel limit strain	ε_s
φ	rotation	
σ_{vi}	steel yield stress	f'_y

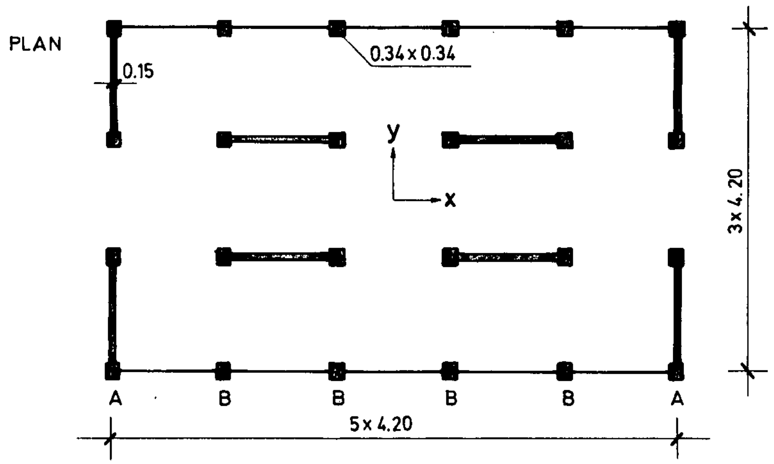

Fig 9.1 IDEALIZED IMS TYPE BUILDING

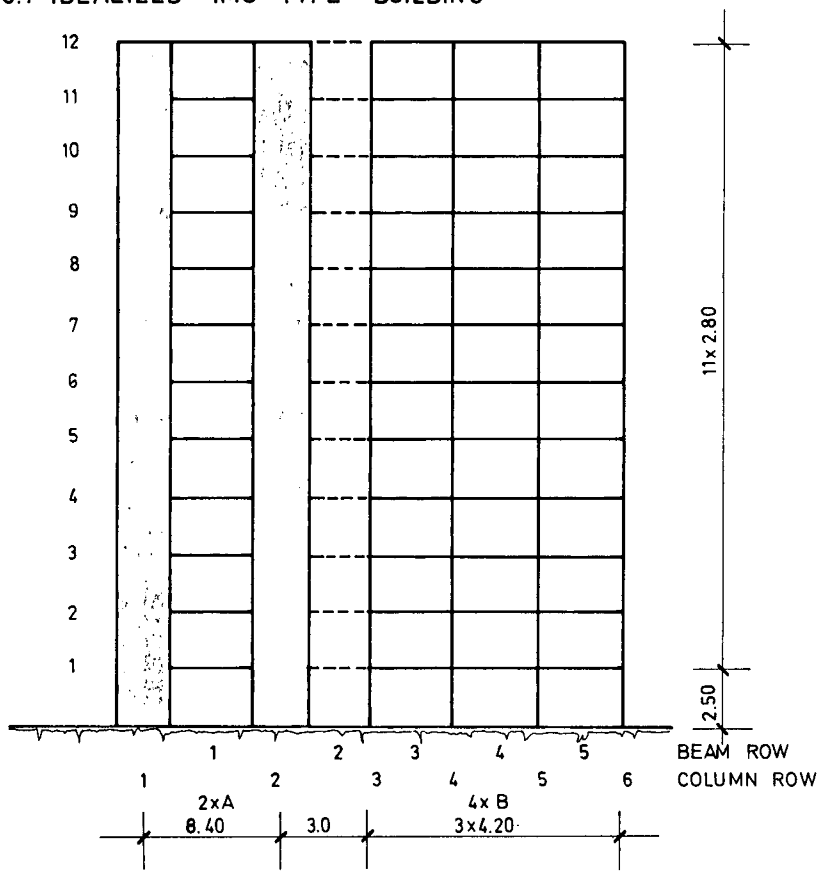

Fig. 9.2 ANALYTICAL MODEL

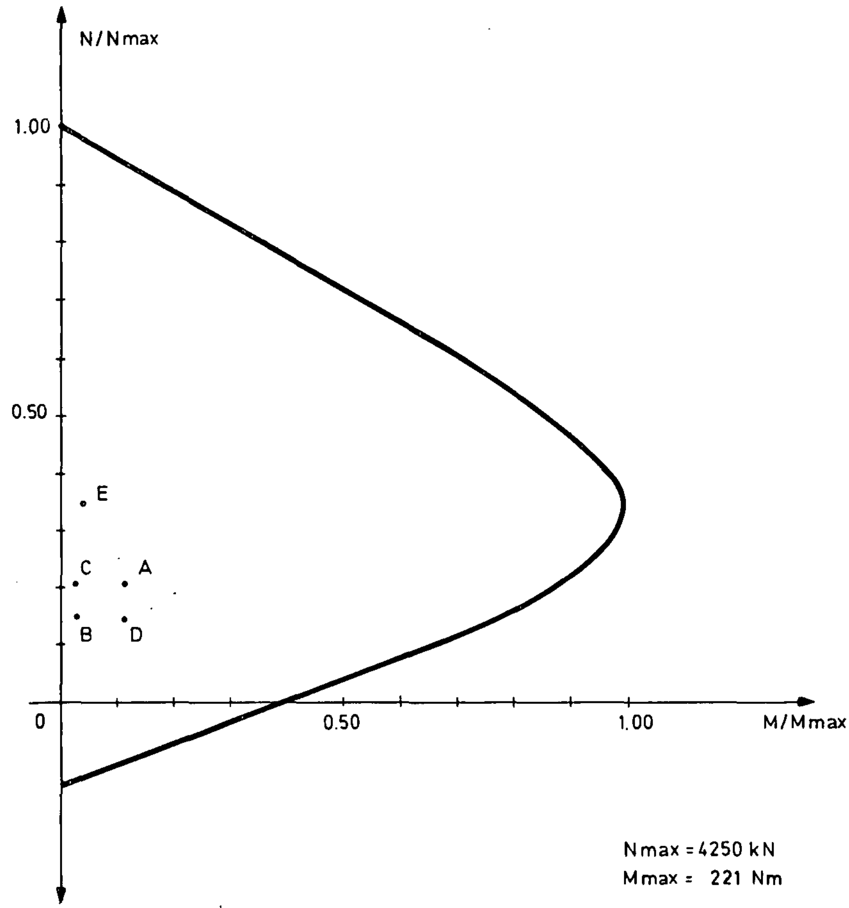

Fig. 9.3 INTERACTION DIAGRAM FOR COLUMN

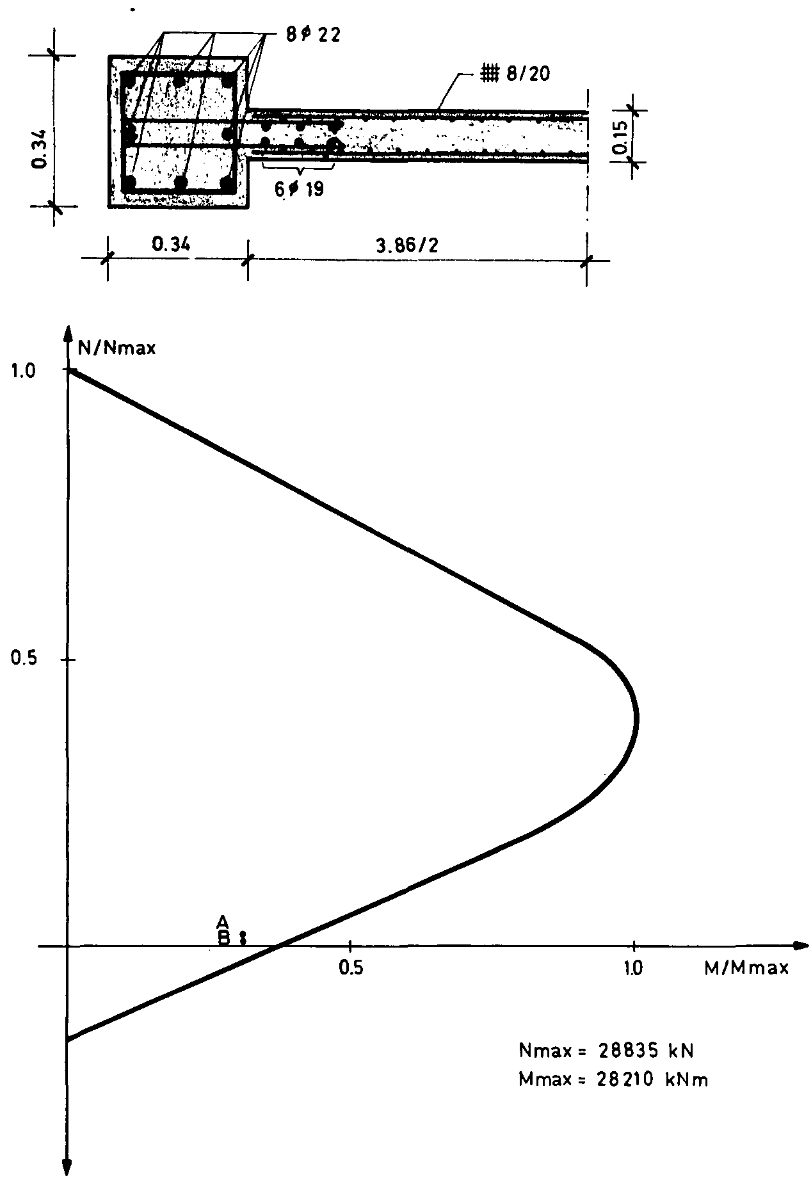

Fig. 9.4 INTERACTION DIAGRAM FOR SHEAR WALL

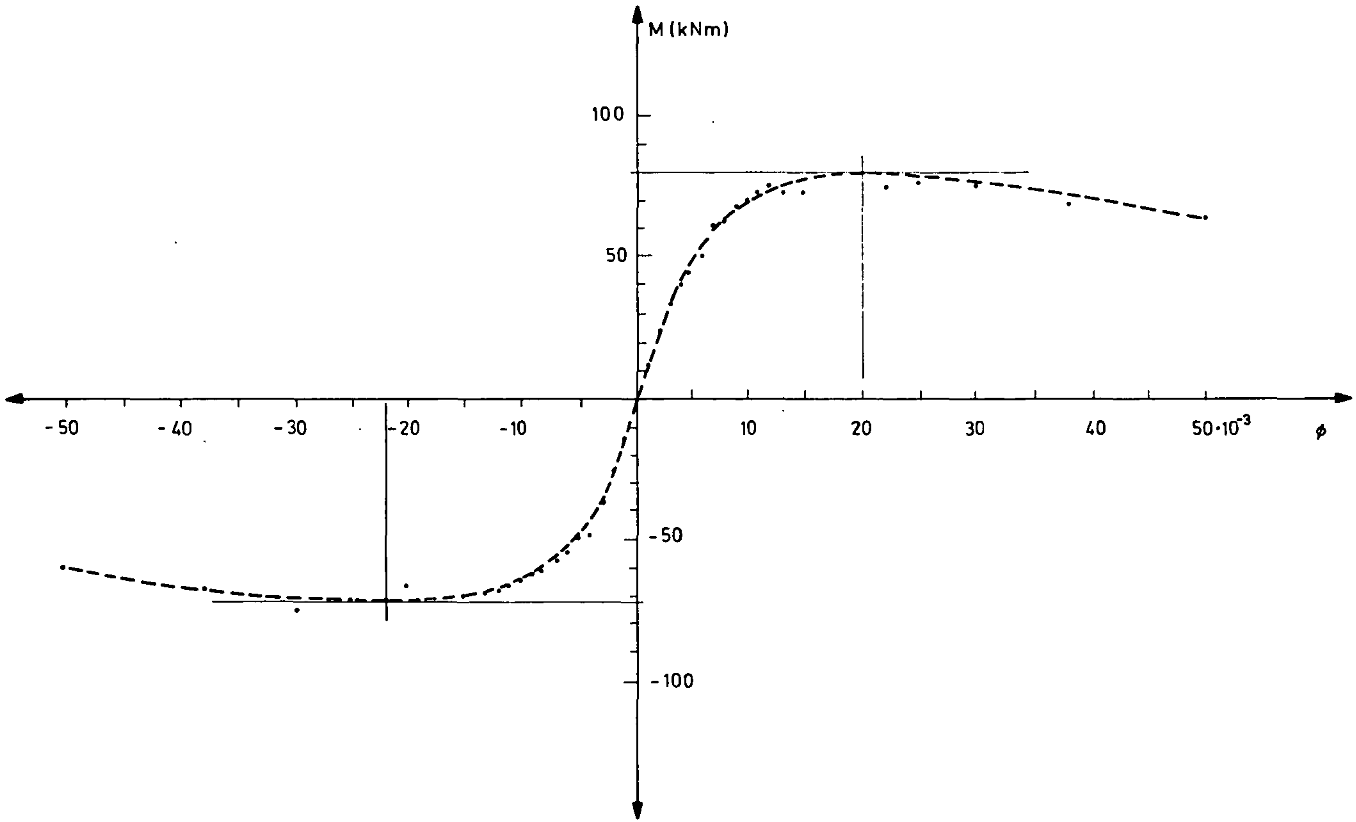

Fig. 9.4. A BEAM - COLUMN JOINT M - ϕ RELATIONSHIP

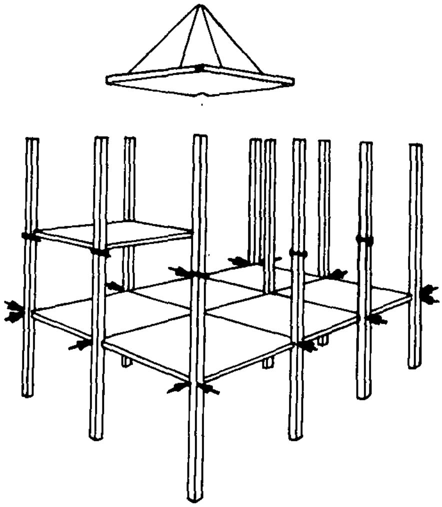

Fig. 9.5 Constructional concept of the IMS System

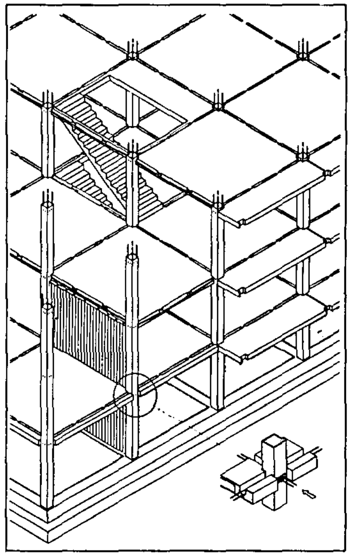

Fig.9.6 Primary structure of the IMS system

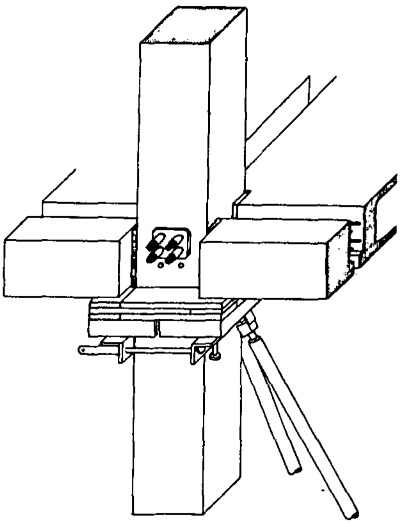

Fig. 9.7 Edge girders

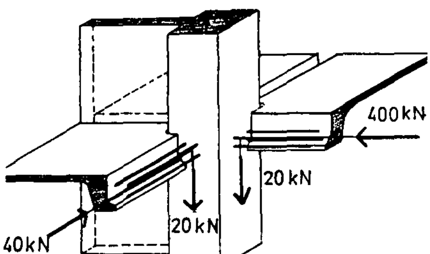

Fig. 9.8 Prestressing forces in the floor slab-column joint

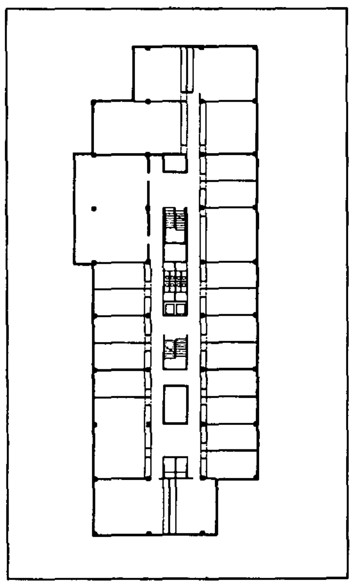

Fig. 9.9 An office building

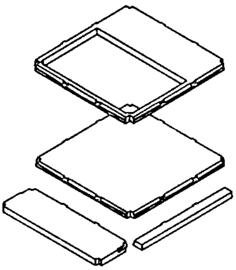

Fig.9.10 Floor slabs, cantilever slabs and edge girders

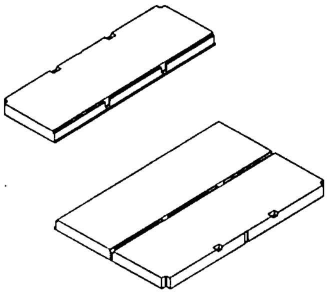

Fig.9.11 Three-segment floor slab for large spans

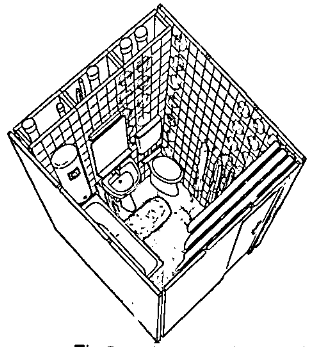

Fig.9.12 "Sigma" bathroom unit

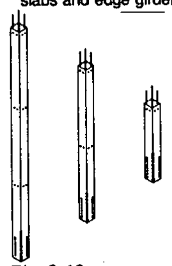

Fig.9.13 Columns

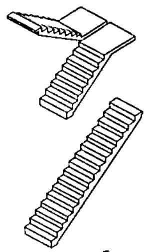

Fig.9.14 Staircases

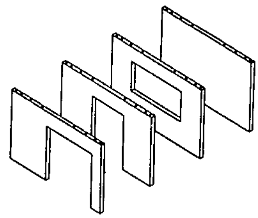

Fig.9.15 Shear walls

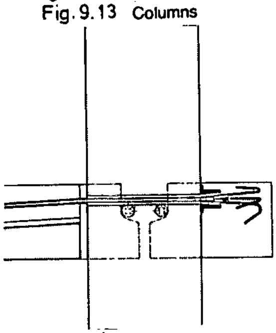

Fig.9.16 Joint between column and floor slab

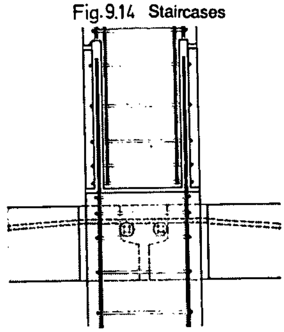

Fig.9.17 Detail of column continuation

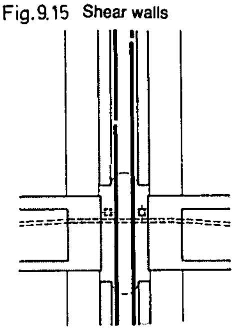

Fig.9.18 Joint between floor slab and shear wall

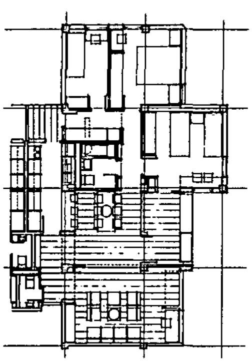

Fig.9.19 Standard apartment, Sarajevo

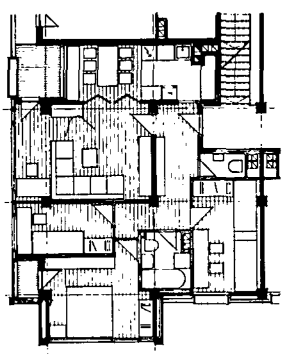

Fig.9.20 Standard apartment, Niš

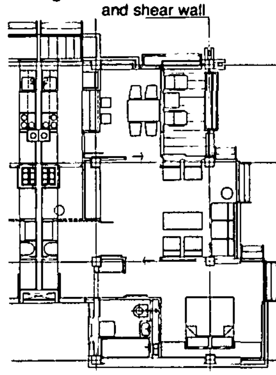

Fig.9.21 Standard apartment, Galenika estate, Belgrade

Fig. 9. 22 The Liman housing estate in Novi Sad (1969)

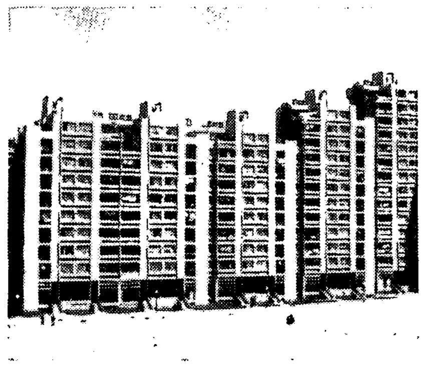

Fig. 9. 23 The Alipašino Polje housing estate in Sarajevo (1978)

Fig. 9. 24 Galenika housing estate in Zemun (Belgrade) (1977)

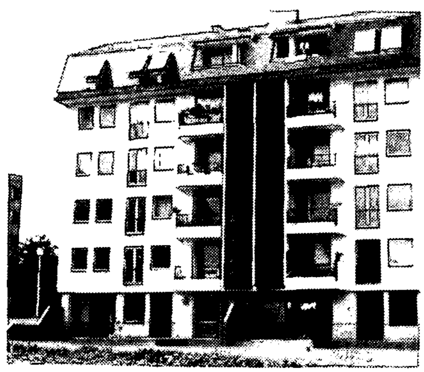

Fig. 9. 25 Hiseta housing estate in Banja Luka (1980)

Fig. 9. 26 Hiseta housing estate in Banja Luka (1980)

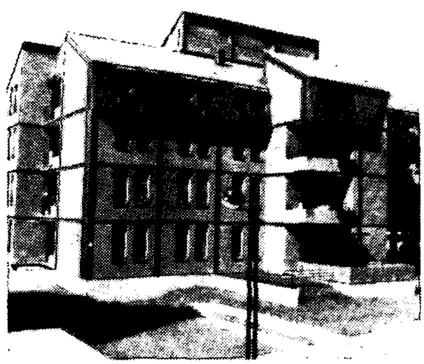

Fig. 9. 27 Residential building on the Cerak estate (1981)

Annex 9.: Experimental Results on the IMS System

Tests up to 1972.

All the elements and basic joints of the IMS System have been tested experimentally, either on full size or on scale models. While part of normal system development, these tests were also necessary to prove quality, safety and all other properties which a system must possess for large scale application.

A 4.20x4.20 m coffered floor slab was test loaded to failure in the laboratory; failure occurred at 25.4 kN/m^2, i.e., at a value 5 times higher than the design load.

The same model was previously tested dynamically and statically by applying a rotating angle to the wall-to-slab joint. The model was loaded by powerful harmonic oscillations in the duration of 40 minutes with an angle of rotation of up to $2.3 \cdot 10^{-3}$, i.e., 2.3 times the design angle for a IX^{o} earthquake. Following this, the model withstood test loading to failure.

Tests carried out in 1972-1982.

The purpose of model tests was to determine the loadbearing capacity of the prestressed joint under cyclic loading and the relationship between the angle of rotation, ϕ, and bending moment, M, at the joint. It was also required to formulate an analytical model describing the joint. Several scaled models were tested, of which two will be described here.

The first model represents 2/3 (in terms of width) of a standard 4.20x4.20 m span IMS slab, joined at both ends to shear walls of much greater stiffness, as shown in Fig. 9.28. The slab was tied to the shear walls by pretensioning four 6ϕ5 mm steel cables in the same way as on actual buildings. Lateral prestressing was applied in the same way. Figure 9.29 shows the model in plan with the layout of the pretensioning cables. The total prestressing force used in the tests was 400 kN per batch of four cables. After prestressing the cables were protected by injecting the ducts in the walls and concreting in the gap between the two halves of the slabs where they were laid.

The shear walls were butted against steel supports via roller bearings, thereby defining their turning points. To ensure equal rotation at the two ends of the floor slab, the upper edges of the shear walls were joined by a strong reinforced concrete beam, also via roll-bearings.

The model was loaded by a double action hydraulic jack, as shown in Fig. 9.28. The force (or moment) applied was measured by load cell. The rotation of the wall at the point of application of the jack was measured by an inductive displacement transducer. The process was precisely controlled by an electric servo system with the LVDT as the pilot in the feedback loop. The angle of rotation, ϕ , and the applied moment were automatically recorded on a plotter.

Throughout the tests the model was subjected to a constant load of 2.33 kN/m^2, corresponding to the gravitational loading which would be expected in practice.

The tests yielded a number of M-ϕ hysteresis loops for various load states. For small rotations, up to $\phi=2.10^{-3}$, with no cracking, the loops had the usual elongated elliptical shape which corresponds to elastic behaviour of the structure. With the apperance of cracks the M-ϕ curves took on the s-shape typical of prestressed concrete. Figure 9.32 shows one of the M-ϕ plots. It may be seen that the ascending and descending branches are practically parallel. It may also be deduced that the model behaved as a bilinear elastic system. This is confirmed by many other M-ϕ curves. It was also found that the distance between the rising and failling branches was constant, except at very high rotations when the joint was seriously damaged.

The hysteresis loop only began to change its shape at high rotations, when the cracks became much wider and the influence of the ordinary reinforcement began to make itself felt. The ascending and descending branches of the loop were no longer parallel, and the distance between them increased with increasing rotation amplitude. Figure 9.31 shows the shape of the loops for angles $\phi = 9.10^{-3}$ to $\phi = 14.10^{-3}$, clearly illustrating the above statements. Above $\phi \doteq 12.10^{-3}$ some initial instability of the loop becomes detectable.

The model was loaded up to a rotation of $\phi = 23.10^{-3}$, which corresponds to a relative displacement between storeys of 2.3 % of storey height. Larger displacements could not be implemented with the given measuring and ancillary equipment. The previously calculated load-bearing capacity was anyway greatly exceeded. A subsequent, more precise analysis of the joint using the results of strain measurements in its vicinity, revealed that the influence of the lateral, transversally prestressed, surfaces was much more significant than had been assumed in prior analyses. It was found that they contributed about 50 % of the joint's load-bearing capacity.

On termination of the tests the slab was still supporting the vertical load without any signs of vertical displacement of the joint.

Upon off-loading the model was still in a quite good condition. It was repaired by injecting all cracks with an epoxy resin solution and, after three days, retested in the same way. The results were practically the same as in the first test, even with somewhat less scatter, so that they need not be presented here.

The second model differed from the first only in two details: the upper beam was omitted and the left shear wall was replaced by a column with two hinges (a pendulum column). Unlike the first, this is a statically determinate model the analysis of which is much simpler (Fig. 9.30).

The different bending moment distribution in this model, with the zero point at the left support, gave rise to a different cracking pattern. Apart from a large transverse crack at the joint with the shear wall, a certain number of cracks extending all the way to the middle of the span appeared. This crack configuration led to greater involvement of the ordinary reinforcement in the slab, which was manifested in the shape of the hysteresis loop. The basic shape of the loop remained the same as in the first model. Up to $\phi = 7.10^{-3}$ all remarks made concerning the first model are applicable here. The model was loaded up to a wall rotation of $\phi = 50.0.10^{-3}$, which corresponds to a relative displacement between storeys of 5% of storey height. Then the joint began to behave like a kind of hinge which permitted further rotation of the shear wall without increment of the bending moment at the joint. It was therefore impossible to break the model completely in this way.

At the extreme rotation achieved ($\phi = 50.10^{-3}$) the crack width at the joint was about 5 mm, with crushing of the concrete in some parts of the joint.

Upon off-loading the cracks still closed up very well. In this state the model still supported the entire vertical load withoud trouble and with no vertical displacement of the joint.

For rotations up to $\phi=10^{-2}$ the relative energy consumption per cycle correponds on the average to that of an elastic system with viscous damping 2% of critical.

In conclusion the following remarks may be made:

1. Well-designed prestressing provides excellent bonding of structural components and guarantees the integrity of the structure throughout the duration of an earthquake.
2. The IMS prestressed joint has a very high load-bearing capacity, exceeding the usual load-bearing capacity of reinforced concrete shear walls. Therefore the joint between the slab and the vertical elements remains capable of carrying all the loads throughout the duration of an earthquake, and is certainly not the weakest point in the structure.
3. Energy absorption of "pure" prestressed concrete is not particularly great and corresponds to that exhibited by elastic structures. It does not increase with the appearance and widening of cracks, until perhaps just failure, a point which could not be tested here.
4. Energy absorption can easily be increased introducing small amounts of ordinary, ductile steel reinforcement at selected points in the prestressed concrete structure, or by adding specially designed reinforced concrete elements to act as the principal energy absorbers.

The third model tested was a scaled three-storey shear wall. It represents the three lowest storeys of a shear wall belonging to a common eight-storey building. The scale adopted was 12.17. The general view of the testing arrangement is shown in Fig. 9.33.

The model was monolithically cast. The columns were not tied to the middle panels except at the floor slabs imitated by cross ribs. To prevent the interaction of columns and middle panels, save via the slab joint, a smooth plastic foil was inserted between the columns and the panel during casting. The cross ribs simulating beams of the frame were presstressed with a steel cable 6Ø5 mm and 110 kN force corresponding to an average prestressing of joints in IMS buildings.

An axial force corresponding to an eight-storey building was simulated on the model by means of cables. Figure 9.33 shows the cable arrangement. A force of 58.1 kN was applied to the upper and a force of 60.0 kN to the bottom column of the model. These values were 9% lower than the calculated force which is an average to be expected in such a building. Reinforcement by means of ordinary reinforcement was provided similarly in all to the original structure.

The load was increased in increments, and hysteresis loops were plotted simultaneously.

Testing of the shear wall model helped to obtain firstly the P-δ diagrams which were then used for the analytical realisation of a curve as a function of the bending moment C(M). On the basis of the achieved function C(M) an analytical model was set up for the elasto-plastic numerical analysis of IMS buildings.

It has also been proved that prestressed joints across which the shear stresses of the shear wall are transmitted have a very high load-bearing capacity up to the point of failure. The behaviour of the wall in the elastic stage came close to the one foreseen by a somewhat simplified process for the calculation of the shear wall as an integral homogeneous structure which also includes the columns.

The shear wall also had a very high ductility. Displacement of the wall top towards the end of the test reached 83 mm which is 27 times the displacement a linear elastic wall would have had with an identical bending moment.

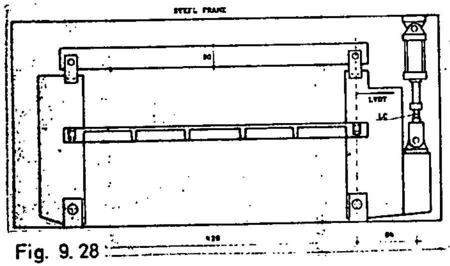

Fig. 9.28

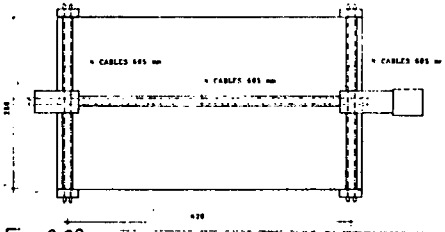

Fig. 9.29

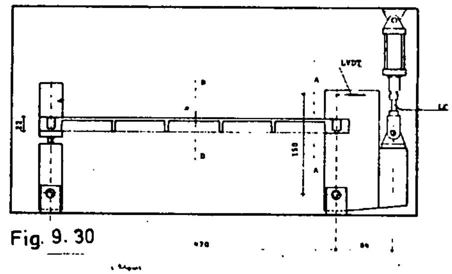

Fig. 9.30

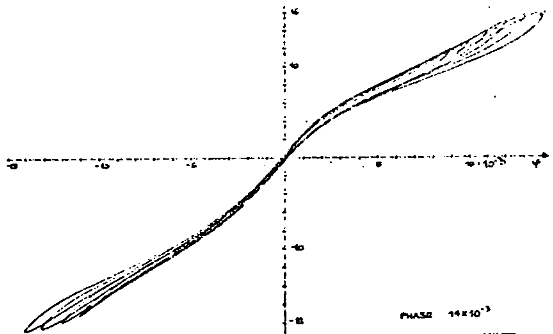

Fig. 9.31

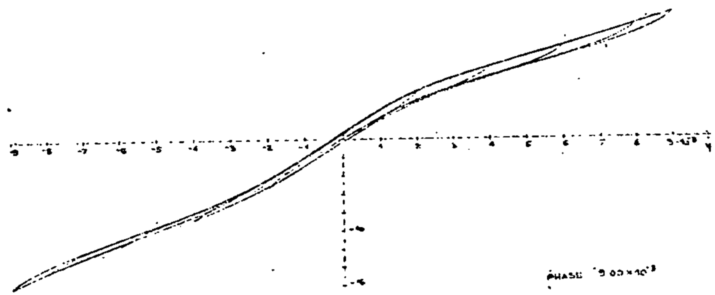

Fig. 9.32

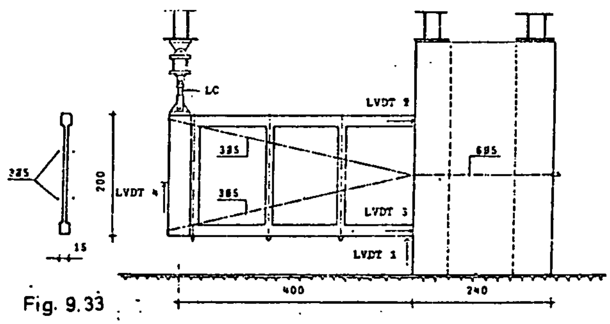

Fig. 9.33

10. REPRESENTATIVE EXAMPLE BUILDING SYSTEM YUGOSLAVIA

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10.1 SCOPE

The (Tempo) System presented here is based on full prefabrication of all nonstructural units, partial prefabrication of floor slabs (Omnia), while the structural walls are of in situ cast concrete. The structural system, method of unit fabrication in the plant, construction sequence and transport are discussed. Structural analysis for a typical 8-storey building involved first the determination of earthquake forces in accordance with the respective codes, followed by the calculation of internal forces. Proportioning is shown for the selected structural unit (one load-bearing wall). The review is supplemented by details of selected nonstructural prefabricated units. The appendix discusses the methodology of unit testing for loads in the stage of fabrication, transport, erection and use.

10.2 DESCRIPTION OF STRUCTURAL SYSTEM

10.2.1 Review of Structural System

The industrialized building system developed by TEMPO is a successful combination of the traditional, monolithic procedure of reinforced concrete residential building and of a fully industrialized large panel system. The structural system lends itself to application in earthquake areas.

The load bearing walls are cast in place.

Foundations are reinforced concrete strips laid on bearing soil. The strips, connected in both orthogonal directions and making up a rigid capping beam, are poured in situ. The foundation strips support 0.16 m load-bearing reinforced concrete walls. The cross walls are spaced uniformly at 5.56 m, while the longitudinal walls are provided in the central part of the building. The walls are concrete, poured in situ. Typical floor plans are shown in Fig. 10.1.

Floor slabs are of the Omnia type. These are 0.04 m reinforced concrete slabs with welded mats and truss girders giving the slab required stiffness and safety during transport and erection. The slabs are supported every 1.30 - 1.50 m, and concrete is then poured on top to full slab thickness, 0.16 m. Being continuous girders, the slabs are only supported by cross walls. Slab continuity is achieved by reinforcement mats laid across wall lines. The slabs are made monolithic along joints by additional mats laid over the joints, and a top concrete layer. The wall-to-slab joint is also considered to be monolithic.

The facades are completely prefabricated. These are three-layer panels, with the load-bearing reinforced concrete structure as the first layer, an expanded polystyrene heat and vapor barrier as the second layer, and the third, external concrete layer protecting the second one. The third layer is also used to shape the external face of the building. The first and third layer are connected by stainless bars. Complete facade elements are delivered on site by special vehicles, and placed. Connection with the walls is provided by soft reinforcement which enters cross walls from the facade panel. The facade are erected and fixed prior to concrete wall pouring, and their link with the load-bearing system is accordingly entirely monolithic.

Staircase flights are prefabricated and placed in the respective position, and connected with floor slabs.

Bathroom units are completely plant prefabricated, and fitted into the building as complete "boxes" with all installation connections.

All secondary elements - concrete or aerated concrete partition walls, parapets, cornices, decorative concrete containers for flowers and shrubs etc. - are precast.

Structural design for the action of vertical and horizontal forces follows the procedure applied to a monolithic reinforced concrete structure according to standard specifications. Calculations are also available for all stages of production, transport and erection, and for the joints, for all prefabricated components.

10.2.2 Component Manufacture

A plant for the completely industrialized manufacture of components has a capacity of 1200 apartments a year.

Facade elements are produced on rails fitted onto movable 6 x 3.5 m tables, on a circular production line. Faster concrete hardening is carried out in a continuous tunnel chamber at 65-75°C at a relative humidity of 85 %. The facility is on automatic control. Partition walls are produced in a tandem battery (mold) consisting of 2x6 compartments. The battery is fitted on a rail track;the concrete is cured in a tunnel chamber. Omnia floor slabs are produced on two parallel steel tracks. Circulating steam heaters are fitted under the working plane. The finisher moves along the track (65 m long), pouring the concrete, vibrating and smoothing it. All the elements are stored in the position which they will have on the building. Floor slabs are placed on wooden pads spaced 1.5-2.0 m. Facade elements and partition walls are fitted onto pallets in a vertical position.

10.2.3 Construction

The components are erected according to the erection plans. Floor slabs are fitted after the supports. Partition walls "hanging" from the floor slabs are fitted on four metal supports.Floor slabs are connected by reinforcement along the longitudinal joint, and by reinforcement in the top zone at the line of intersection with the walls. After the next stage, pouring of the top concrete layer of the slab, a monolithic joint is achieved.

Facade elements are provisionally supported by oblique props. A large format steel formwork, connected by bolts, is placed in the position of the future load-bearing walls;facade element reinforcement enters the formwork and is linked with wall reinforcement. Pouring of the concrete walls connects the whole into a monolithic assemblage. Facade elements are suspended on gable walls and connected with the floor slabs before the pouring of the top concrete layer of the latter.

10.3 STRUCTURAL ANALYSIS

The floor plan of a typical building is shown in Fig. 10.1, and the elevations of the structural walls in Figs. 10.2 and 10.3. Because of space limitations, only the earthquake calculation for the longitudinal direction (x) has been shown. A segment of the building having a floor plan area of 12.80 x 11.20 = 142 sq.m. is analysed. According to codes, the calculation must be repeated for the orther direction as well (y).

10.3.1 Design Loads

Table 10.1 lists mass distribution along the elevation according to engineering design data.

TABLE 10.1 Vertical mass distribution

Level (m)	19.0	16.80	14.00	11.20	8.40	5.60	2.80	0.00
Mass (in tons)	131.4	122.9	130.3	143.9	145.6	159.0	196.5	219.3

Total mass G = 1248.9 tons

Total earthquake force (see Yugoslav seismic code in Vol.7)

$S = K \cdot G$
$K = K_o \cdot K_s \cdot K_d \cdot K_p$
$K_o = 1.0$ $K_s = 0.050$ for zone VIII MCS $K_d = 1.0$ $K_p = 1.0$

$S = 0.05\ G = 0.05 \cdot 1248.9 = 62.4$ t (624 kN)

The coefficient $K_d = 1.0$ because the fundamental period $T_1 \leqslant 0.7$ sec. See Code, Art. 25.

10.3.2 Computation technique

Vertically the force is distributed along the building as follows.

- 0.15 S = 93.6 kN at the building top
- 0.85 S = 530.4kN is distributed according to the approximate formula:

$$S_i = 0.85\ S \cdot G_i H_i \,/ \sum_{4}^{8} G_i H_i$$

TABLE 10.2. Vertical distribution of earthquake forces, total shear forces and bending moments

Level	(m)	19.6	16.8	14.0	11.2	8.4	5.6	2.8	0.0	-4.5
H_i	(m)	24.1	21.3	18.5	15.7	12.9	10.1	7.3	4.5	0.0
$G_i H_i$	(tm)	3167	2618	2411	2259	1878	1606	1434	987	0
S_i	(kN)	102.7	84.9	78.2	73.2	60.9	52.1	46.5	32.0	0
$0.15\ S_i$		93.6								
T_i	(kN)	196.3	281.3	359.4	432.6	493.5	545.6	592.1	624.0	624.0
M_t	(kN.m)	0	549.6	1337	2343	3554	4936	6464	8122	10930

$$\sum G_i H_i = 16360$$

$0.85\ S / \sum G_i H_i = 53.04 / 16360 = 0.0032421$

10.3.3 Internal Forces

The walls Z1, Z2 and Z3 are equal and each of them will take 1/3 of the seismic force.

TABLE 10.3 Internal forces in wall Z1

Level	(m)	19.6	16.8	14.0	11.2	8.4	5.6	2.8	0.0	-4.5
T_i	(kN)	65.4	93.6	119.7	144.0	164.3	181.7	197.2	207.8	207.8
M_i	(kNm)	0	183	445	780	1183	1644	2152	2705	3640

Influence of torsion: according to the Art. 34 of the Seismic Code only actual torsion is taken into account.

Accidental torsion is neglected. The building is constructed in pure symetry and therefore no calculation for torsion is requested.

10.4 PROPORTIONING

Wall Z1, level -4.50 m

$F_b = 0.16 \cdot 3.70 = 0.592\ m^2$

$W_b = 0.16 \cdot 3.70^2/6 = 0.365\ m^3$

$N = 1900$ kN

$M = 3640$ kNm

$T = 207.8$ kN

$\sigma_o = 1900/0.592 = 3.209$ MPa

$\tau_o = 207.8/0.592 = 0.351$ MPa

Calculation of longitudinal reinforcement

$e = M/N = 3640/1900 = 1.91$ m

$$\sigma = \frac{1900}{0.592} \pm \frac{3640}{0.365} = 3.209 \pm 9.973\ \text{(MPa)}$$

Ritter's tables for symetrical reinforcement

$e = 1.75 + 1.91 = 3.66$ m $\qquad$ $h = 3.60$ m

$e/h = 3.66/3.60 = 1.017$

$M_a = M + N \cdot 1.75 = 3640 + 1900 \cdot 1.75 = 6965$ kNm

$b \cdot h^2 = 2.073\ m^3$

$M_a/bh^2 = 3358.9\ kN/m^2 = 3.359$ MPa

$\sigma_{br} = 1.5 \cdot 12.0 = 18.0$ MPa

$\sigma_a = 1.5 \cdot 240.0 = 360.0$ MPa

$C_1 = \sigma_{br} \cdot bh^2/M_a = 18.0/3.359 = 5.36$

$C_2 = \sigma_a \cdot bh^2/M_a = 360/3.359 = 107.17$

$\mu = \mu' = 0.2$ %

$f_a = 0.0020 \cdot 5920 = 11.8\ cm^2$, take 4$\phi$ 20 ribbed bars.

Calculation of shear reinforcement

$T' = \tau_o . b = 351 . 0.16 = 56.2$ kN/m^1

$T_{max} = 2T' = 112.4$ kN/m^1 (shear force doubled to avoid brittle failure)

$\sigma_a = \sigma^y = 500$ MPa (welded wire fabric)

$f_{as} = T_{max}/\sigma_a = 112400/50000 = 2.25$ cm^2/m^1

This reinforcement is uniformuly distributed in the wall: vertically and horizontally.

10.5 SELECTED DETAILS

The Tempo (Zagreb) System is distinguished in particular by its nonstructural units. Figure 10.4 illustrates a typical facade panel consisting of three layers (with the thermal insulation layer in the middle). Figure 10.5 shows a three-layer parapet which is fixed to the load-bearing structure like a facade panel.

External structural walls are lined with panels the form of which is shown in Fig. 10.6; the attachment to the load-bearing structure is shown in Fig. 10.7.

10.6 CONCLUDING REMARKS

The numerous buildings so far erected by the Tempo (Zagreb) industrialized system have shown such a construction method to provide for the required modern conveniences at an economical cost. From the standpoint of earthquake safety typical buildings can be strengthened by more structural walls if necessarry in the longitudinal (x) direction.

Most of the cross walls in the y direction have to be connected in a better way by moving the chimney flues and ventilation ducts off the wall plane (Fig. 10.2, row 1, 3 and 5). All partition walls erected as prefabricated panels must be structurally separated from the load-bearing structure by the magnitude of storey drift.

In consideration of the new earthquake codes, the nearest future will require nonlinear dynamic analyses of such buildings.

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Notation

Symbol used	Explanation	CEB Symbol
G	Total weight of the building	
H_i	Floor distance above foundation top level	
I	Moment of inertia	I
K	Total seismic (base shear) coefficient	
K_d	Dynamic coefficient	
K_o	Importance factor	
K_p	Coefficient of plasticity and damping	
K_s	Seismic coefficient	C_d
M	Bending moment	M
M_a	Bending moment at the tensile reinforcement	M
M_i	Bending moment at specific level	M
M_t	Torsional moment	M_t
N	Axial force	N
S	Total seismic force (base shear)	
S_i	Seismic force at specific level	
T	Shear force	V
T'	Unit shear force	
T_i	Shear force at specific level	
b	Shear wall thickness	
e	Eccentricity	e
e_y	Eccentricity between mass and stiffness center	
f_a	Tensile steel area	A_s
f_{as}	Shear reinforcement area (hor.and vert.)	A_s
h	Shear wall section height	
x,y	Distance in the respective direction	x,y
μ	Percent of tensile reinforcement	
μ'	Percent of compressive reinforcement	
σ	Normal stress	
σ_a	Reinforcement stress	
σ_{bv}	Concrete edge stress	
σ_o	Average axial stress	
τ_o	Average shear stress	

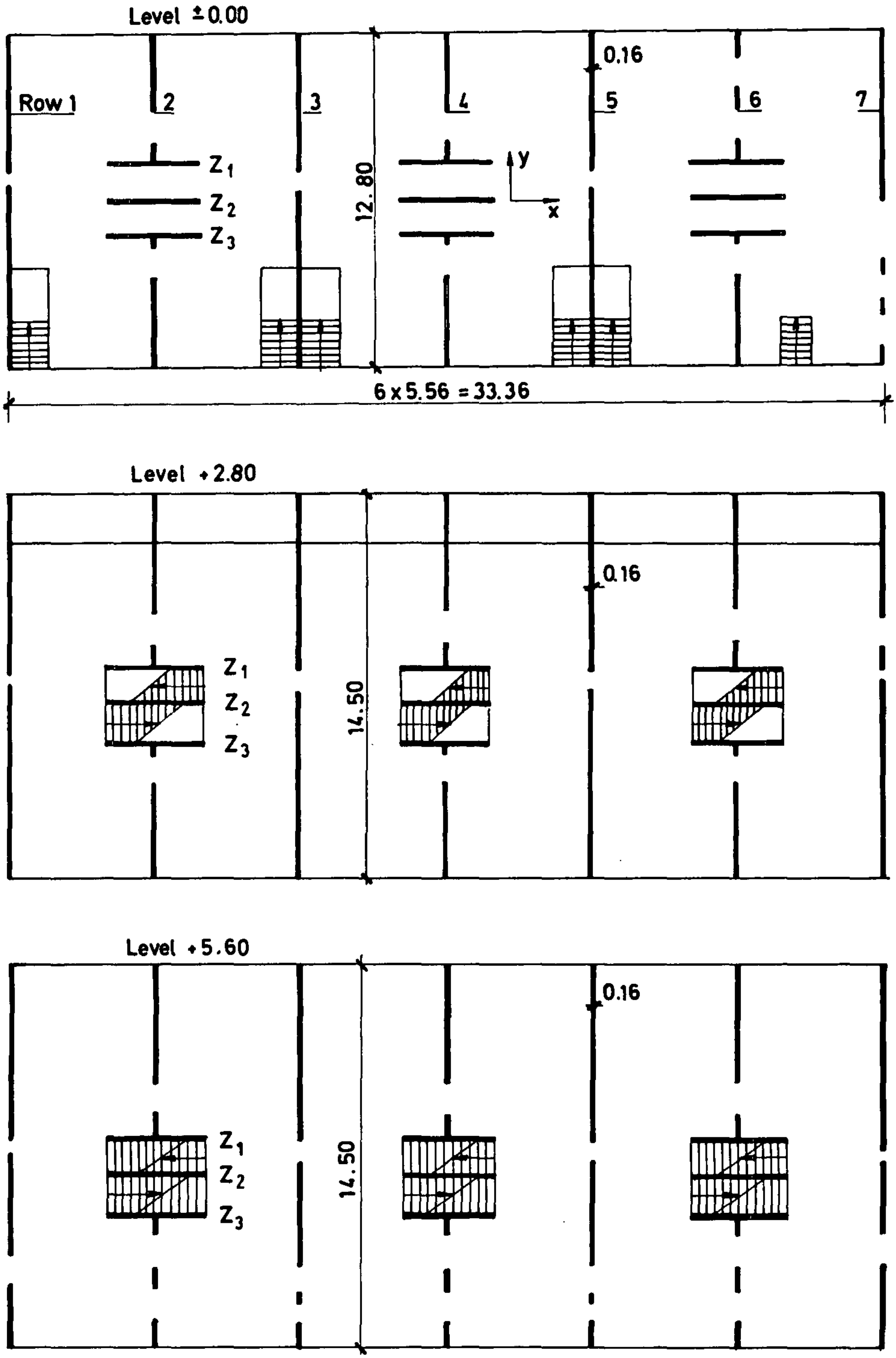


Fig 10.1 TYPICAL FLOOR PLANS

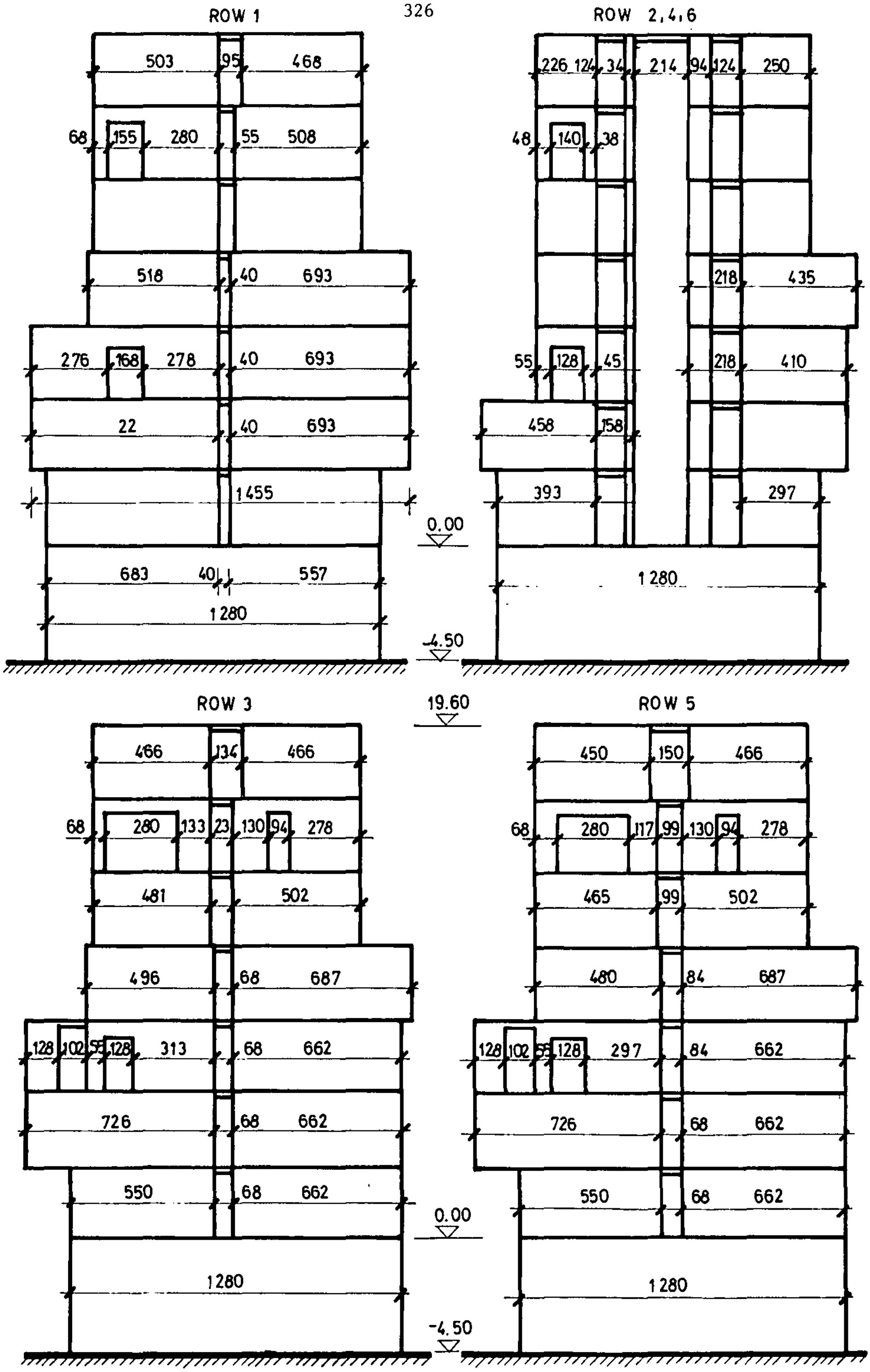


Fig 10.2 VERTICAL SECTION OF SHEAR WALLS

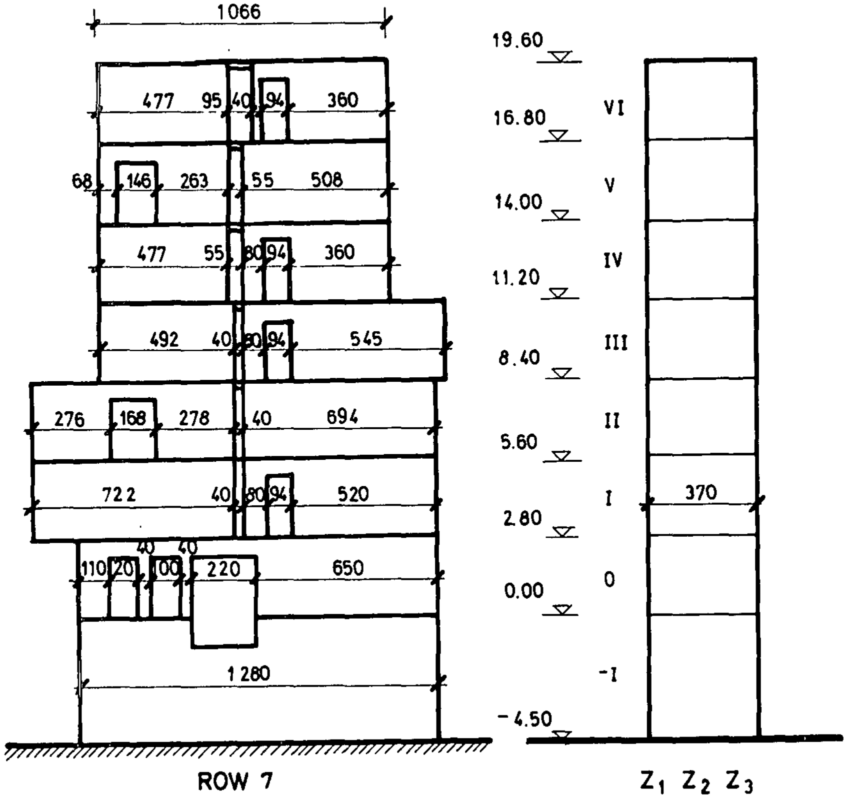


Fig 10.3 VERTICAL SECTION OF SHEAR WALLS

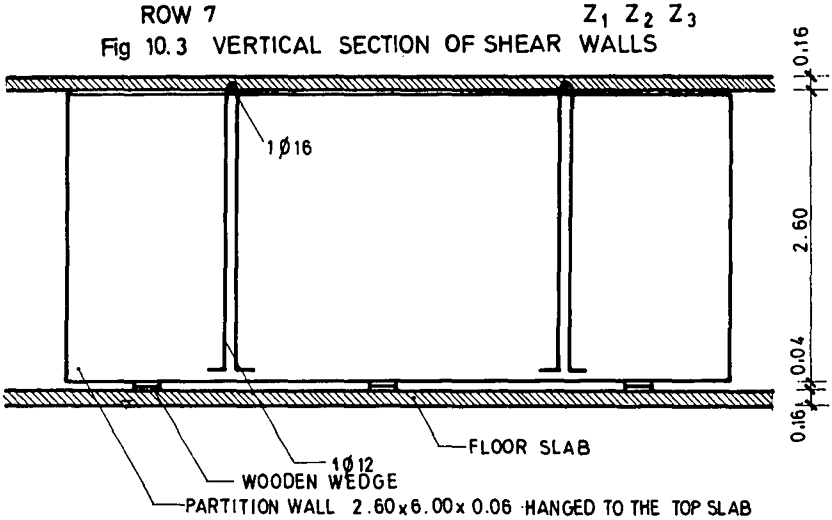


Fig 10.3 A PARTITON WALL CONNECTION TO THE FLOOR SLAB

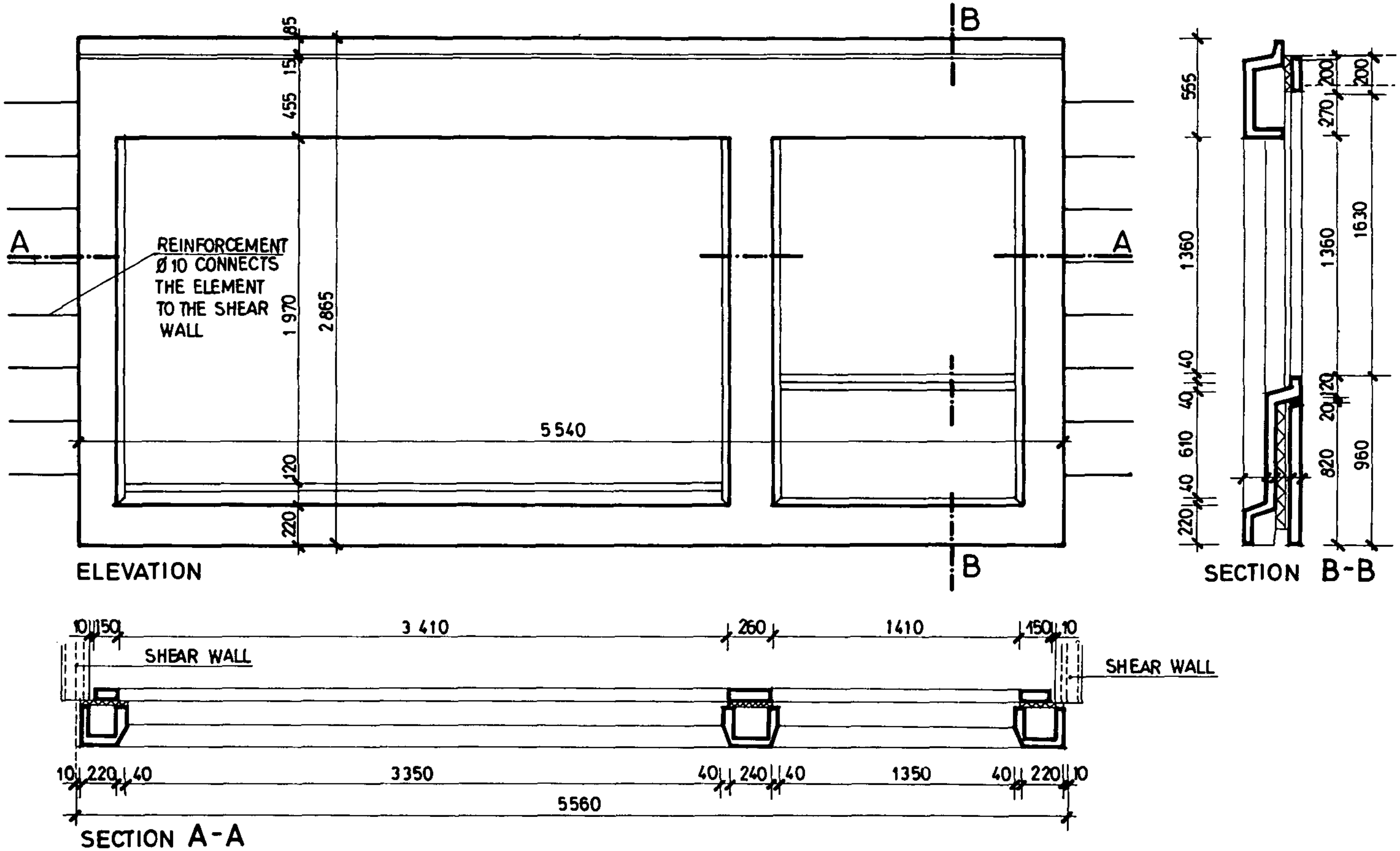


Fig 10.4 TYPICAL PREFABRICATED FASADE PANEL

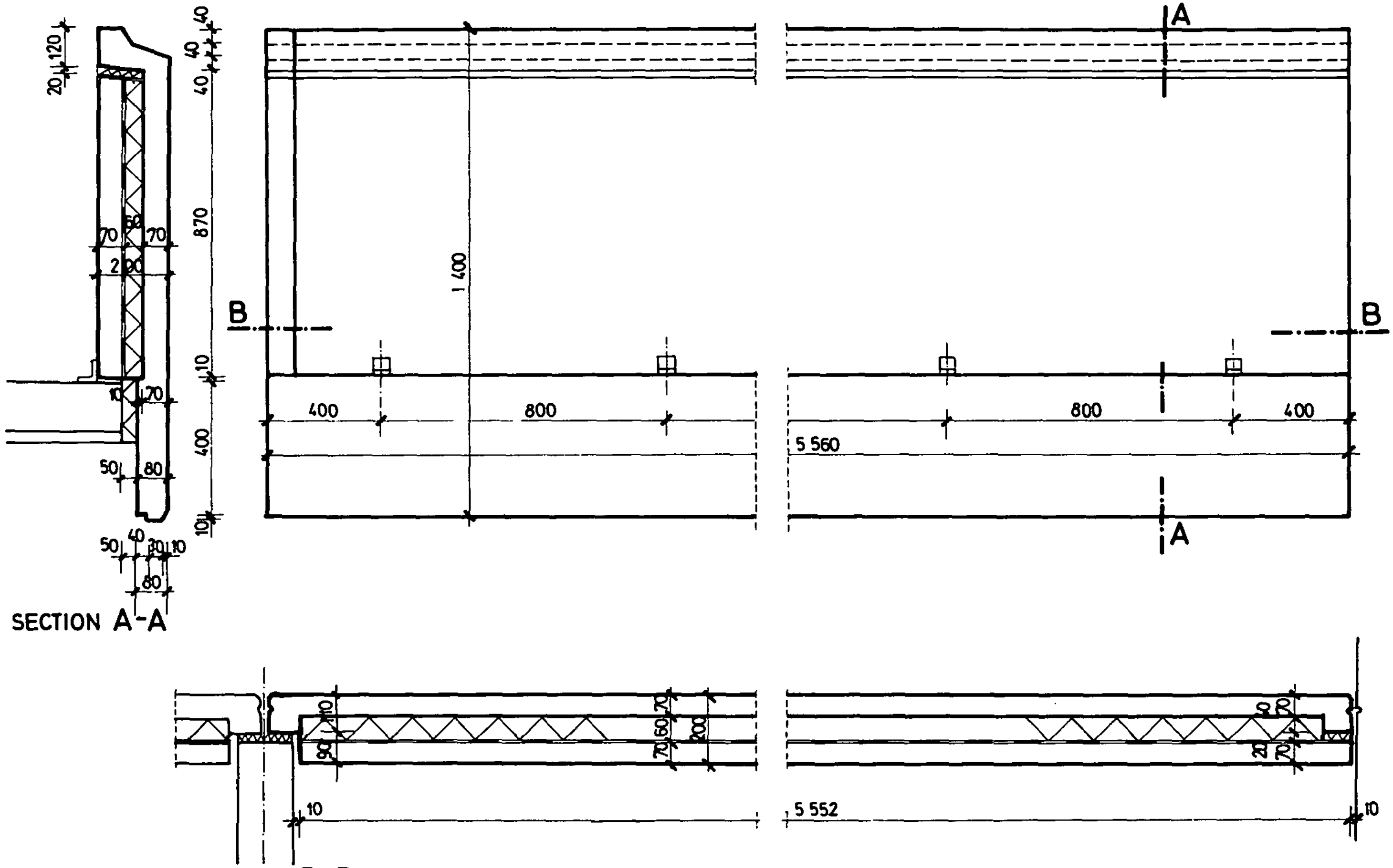
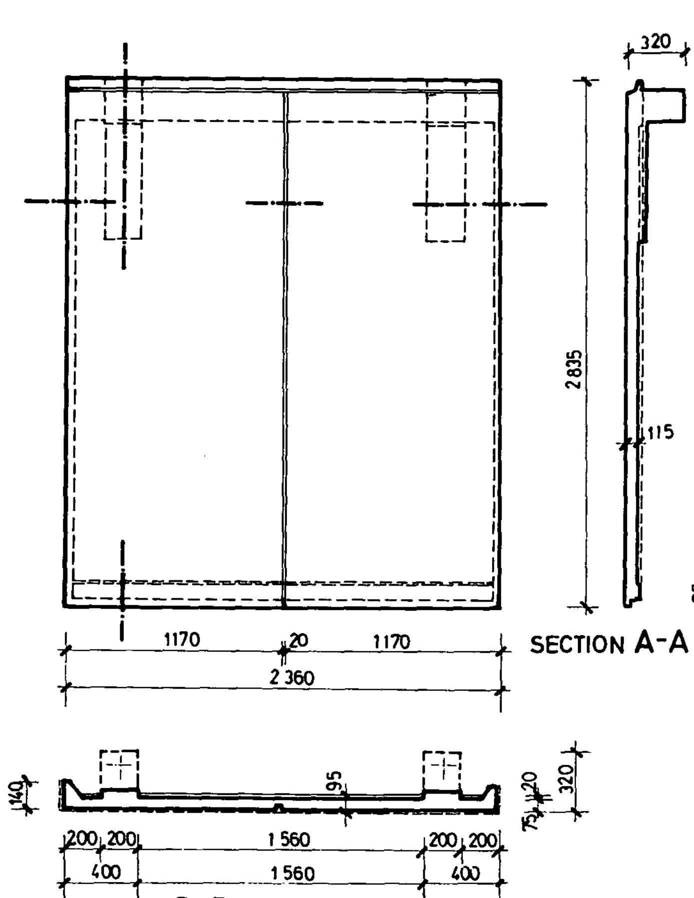


Fig 10.5 TYPICAL PREFABRICATED PARAPET



SECTION B-B

Fig 10.6 PREFABRICATED GABLE WALL LINING ELEMENT

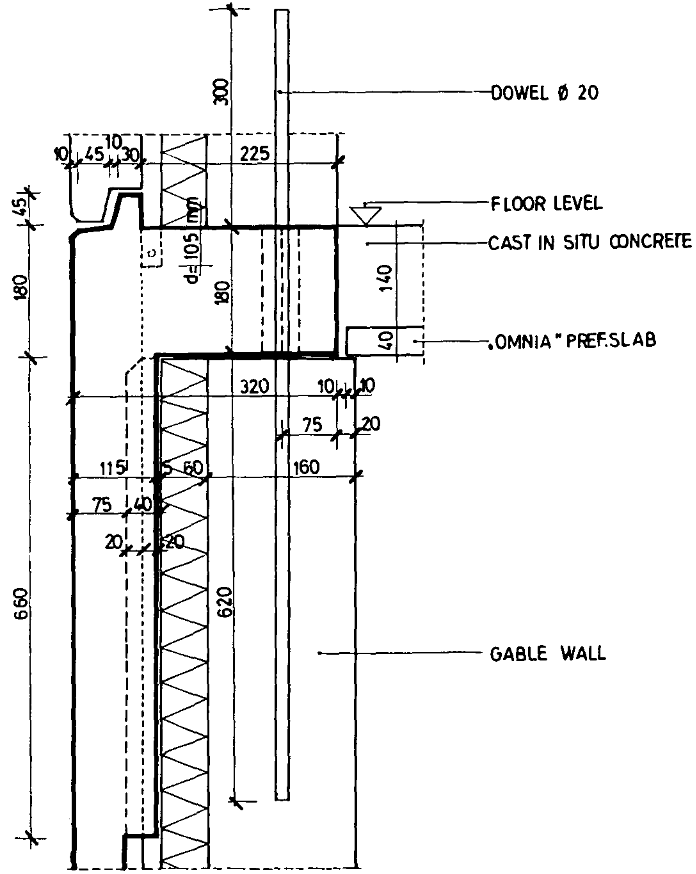


Fig 10.7 DETAIL OF CONNECTION OF LINING ELEMENT TO THE GABLE WALL

Annex 10.: Experimental Results on "Tempo" System

Tests involving nonstructural elements of the Tempo System are presented here. Tests are focused on proving element load-bearing capacity during transport and erection, and for loads acting directly on the element (wind, earthquake). The load-bearing capacity of partition walls rigidly joined with the bearing structure loaded by horizontal forces has also been tested. Numerical test data are not listed, and only testing methodology is reviewed.

Testing of load-bearing capacity of prefabricated lining facade elements (Fig. 10.8.A)

Structural analysis has shown the necessity to test the load-bearing capacity of the short cantilever on which the element is hung. The element was tested horizontally on a rigid test floor. The element was stabilized against lifting and turning. A gradually increasing horizontal force was applied next to the short cantilever at the level of the tubular opening on which the element is hung. See Figures 10.8.A and 10.7.

The force was increased up to the development of the first cracks in the cantilever and then to failure. Failure occurs through a crack localized at the corner of the element, after which displacement increases without an increase of force level.

Testing of the load-bearing capacity of prefabricated facade panels (Fig. 10.8.B and C)

Engineering design and structural analysis showed four different tests to be required for proving the structural load-bearing capacity of the element under consideration:

a) testing of the load-bearing capacity of the middle column to the action of horizontal wind or earthquake load;
b) testing of the load-bearing capacity of the bond between the outer and inner layer of the element;
c) testing of the load-bearing capacity of the element transport anchor;
d) testing of the load-bearing capacity of the plastic anchor for provisional element support, before connecting it to the main bearing structure.

The individual test procedures are described below.

a) Testing of the load-bearing capacity of the middle column (Fig. 10.8.B)
The element was tested in a vertical position. The supports were placed at 2.50 m, i.e., points of intersection of the column with the top and bottom beams. The horizontal force was applied at column mid-height. The force was progressively increased to the point where a satisfactory safety factor was achieved.

b) Testing of the load-bearing capacity of the bond between the outer and inner element layer (Fig. 10.8.C)
According to structural design, the bond is achieved by 6 stainless steel keys. The keys carry the weight of the outer part of the element. The element was tested in horizontal position. The inner side was turned upwards. The top edge of the outer layer was supported along 3.0 m on a rigid line bearing on a testing floor. Force was applied progressively to the bottom edge of the inner element, whereby the bond between

the inner and outer elements was loaded by shear.

c) Testing of the load-bearing capacity of element transport anchors
The anchor was tested by screwing the bolt with the lifting lug into the element; the pull-out force was progressively increased next.

d) Testing of the load-bearing capacity of the plastic anchor for provisional element support
The element was set horizontally, the inner side upwards; plastic anchors were pulled out next by progressively increasing a force acting perpendicularly to the element plane. Structural analysis makes no provision for these anchors. The pull-out test was performed with a hollow hydraulic press.

Testing of the load-bearing capacity of the facade parapet (Fig. 10.8.D)

Engineering design and structural analysis have shown proving structural load-bearing capacity to be required as follows:

a) Wind load has to be simulated, and loads established at crack occurrence and possible failure.
For testing reasons the element was supported, in vertical position, on line vertical supports spaced 4.10 m. A concentrated line force was then applied at mid-span vertically along the panel. The force was progressively increased until the achievement of a satisfactory safety ratio. Deflection at mid-span was measured during testing, and crack development observed. (Fig. 10.8.D).

b) The load-bearing capacity of the bond between the outer and inner component layers has to be tested.
Six stainless steel keys are envisaged by structural analysis.
The keys carry the weight of the external section of the element. The element was tested in horizontal position, the inner side upwards.
The top edge of the outer layer was supported over 3.0 m by a rigid line bearing on a testing floor. A progressive force was applied to the bottom edge of the inner element, testing - by - shear the bond between the outer and inner element layers.

Testing of Omnia prefabricated floor slabs

The slab was placed on 4 bearings spaced 1.50 m - the span matching the maximum permissible spacing of supports in the erection stage. This has produced a structural continuous girder scheme through identical spans spanning 1.50 m. The load was applied between the lattice reinforcement girders and, at mid-span, by a force distributed over 0.20 x 0.20 m area. The force, applied by an hydraulic press, was distributed uniformly along the width of the slab. Measurements were made of the force, and of the vertical displacements at mid-span on both sides and at points along the external lattice girders. 1/100 mm dial gages were placed at the measuring points. Force and deflection data were recorded for load stages. The applied load increment was P=10 kN.

Testing of the load-bearing capacity of the reinforced concrete staircase flight (Fig. 10.8.E)

Load-bearing capacity was tested by applying at mid-span a concentrated force. The elements are tested up to failure.

Testing of the reinforced concrete partition wall (Fig. 10.9)

The partition wall is one of the basic nonstructural elements. In structural nalysis partition walls are not usually designed for the transfer of vertical and horizontal forces, i.e., inadequate account is taken of load-bearing system deformation, partition wall deformability, and partition wall interaction. The partition wall should be separated from the load-bearing structure or it should take a vertical or horizontal load. If the partition wall is joind with the columns and beams by direct contact, it will take part of the load. The joint with the load-bearing structure may be by friction, welding or reinforcement. Because of their joint action the behaviour of load-bearing structures differs substantially from that assumed in structural analysis. Accordingly, knowledge is required about the deformability of the partition wall under the action of horizontal forces in its plane.

A 4.0 m long by 2.60 m high by 0.065 m thick precast reinforced partition wall was selected for the test. The wall is reinforced with a welded wire fabric. The test required the provision of a special device for simulating wall behaviour within the structure. The steel structure illustrated in Fig. 10.9.A, has been found most adequate because of rapid assembly and disassembly, and repeated test capability.

Displacement, measured by an induction displacement gage (LVDT) was controlled by servo-controlled electro-hydraulic equipment. Load history is presented in Fig.10.9.B, hysteresis loops in Fig. 10.9.C, and cracks in specimen in Fig. 10.9.D. The ultimate bearing capacity of the partition wall was 500 kN horizontal force.

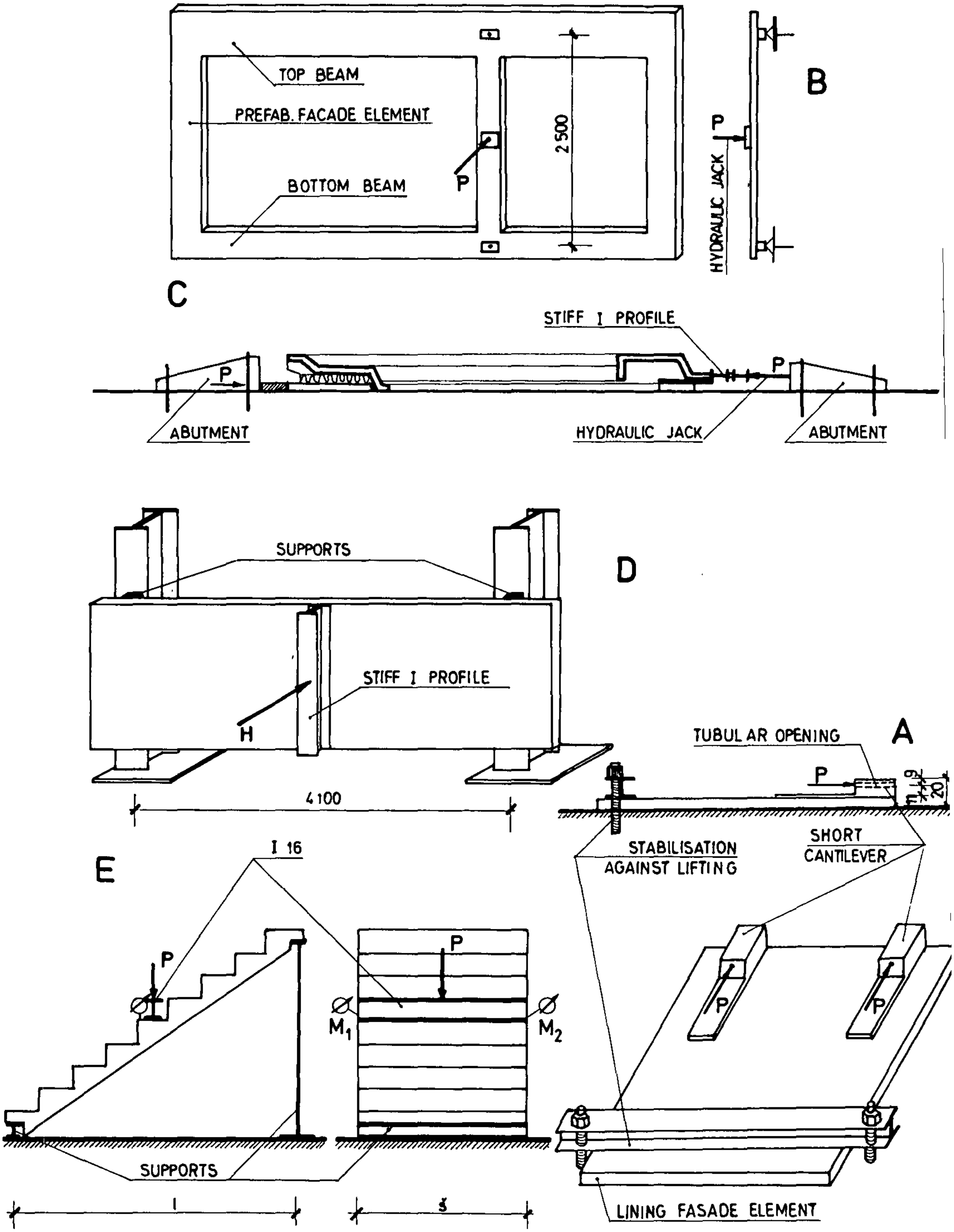


Fig 10.8 TESTS ON STRUCTURAL ELEMENTS

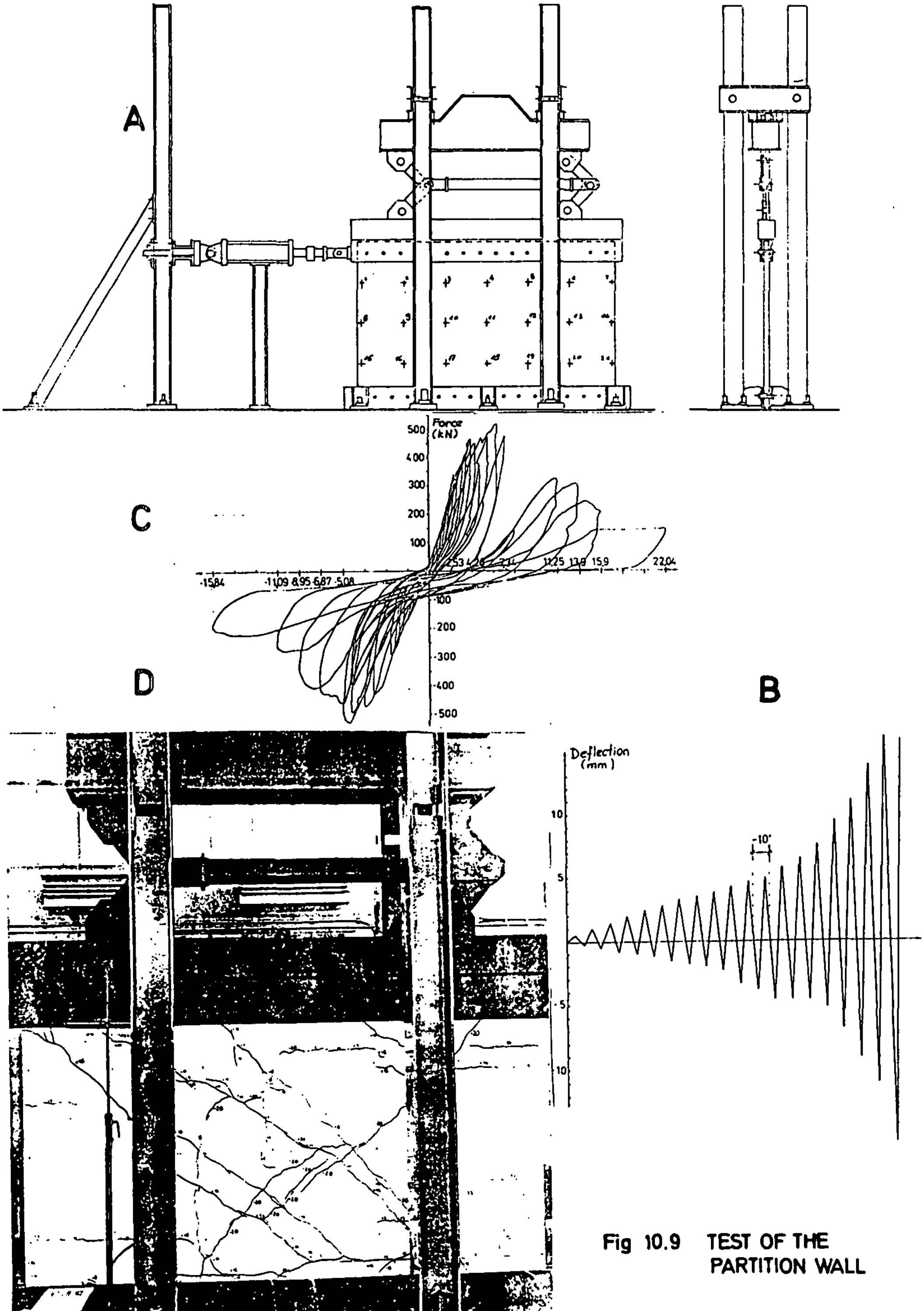


Fig 10.9 TEST OF THE PARTITION WALL