Delft University of Technology

MSC, thesis

The influence of the connection characteristics on the seismic performance of precast concrete structures

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Seismic design of precast concrete requires the study of the influence of the seismic forces on the connections. In the context of this thesis, the performance of various types of connections utilised in precast buildings was evaluated numerically. A six storey high building was chosen for investigation.

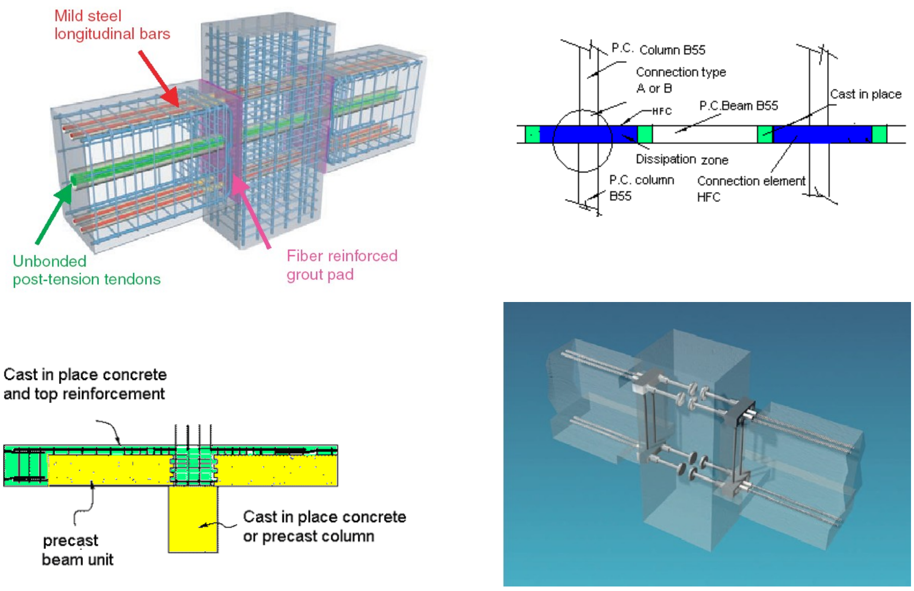


Figure 1: Various connection types.

The seismic actions on the building were computed on the basis of Eurocode 8. The capacity design principle was used for design of the building components. Four different connections of the beams to the columns were designed and evaluated, Fig 1. Non-linear dynamic analysis with the F.E. code Ruaumoko verified the adequacy of the design for the emulated monolithic connection. The results of the investigation have shown that:

with appropriate attention to detail, the connections can be utilised
depending on the location, different connection types may be more appropriate in terms of their earthquake response and/or cost

Contents

Summary

Objective of the thesis is the study of the influence of the connection characteristics on the seismic performance of precast concrete structures. To that end a six storey high building with moment resisting frame was analyzed and designed. The precast concrete building consists of 3×6m span frames at 7.8m centre-to-centre in the transverse direction and 7×7.8m span frames in the longitudinal direction. A 350mm floor spans in the longitudinal direction. The structural system is based on the concept that the precast units should be as long as possible. In the study different types of connections are considered:

(1) A monolithic reinforced concrete connection
(2) A hybrid-fibre concrete connection
(3) A bolted assemblage Dywidag ductile connector DDC.
(4) A post-tension assemblage

The building location is in Greece, zone III. Considering the location and building characteristics, the seismic acceleration A_g is 0.24g. In the calculation and of the seismic force on the building the equivalent shear method is used, the force is distributed on each floor of the building depending on the building mass and height. For the seismic analysis only the frame in the transverse direction is considered most critical.

The design philosophy followed uses the analysis to obtain relations between the consequences of the seismic action and the functioning of the structure globally and locally. That enables the study of the different solutions. The building is considered in the analysis and the design with the different connection types, and for that a suitable strategy is followed, taking into the account the concept of the capacity design where the columns are assumed to be elastic and the yielding occurs in the beam-ends. That enables using an elastic model using elastic response spectrum with a behaviour factor according to EU8. The model is used for the structural analysis in two steps. First an elastic model using redistribution of the bending moments in the beam ends. The calculation results used to determine the modified stiffness of the connected beams in the column beam connections. The modified stiffness of a beam is built as a function of its section dimension, reinforcement, and curvature (Figure 1,and Figure 3).

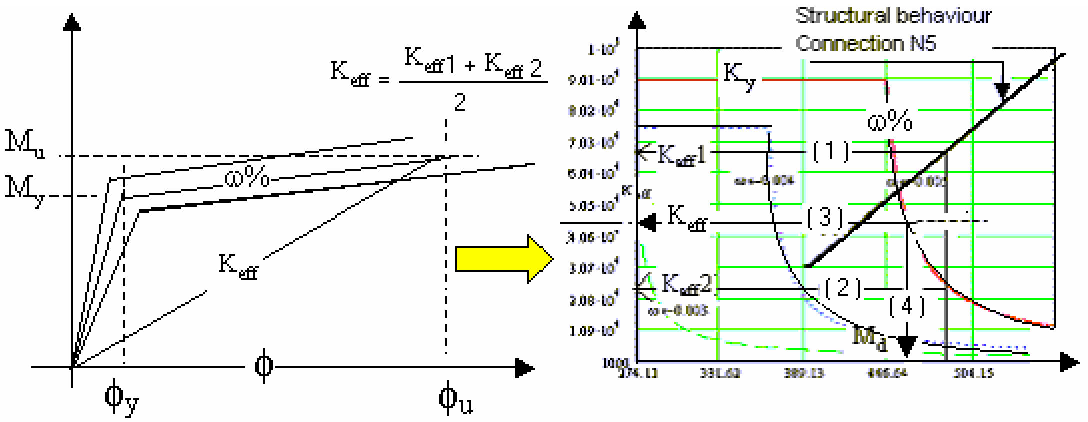


Figure 1: The effective stiffness and the structural behaviour in a column beam connection

Second an elastic model in which the connected beam stiffness is introduced. In the second step the characteristics of the R/C section is described, it accounts for the behaviour of the beam-ends under seismic loading. The latter results in a better response and structural drift estimation. In the elastic model the load combinations are the serviceability limit state and the seismic combinations. All elements are designed for the seismic combination and verified for the serviceability limit state. Important aspects studied are drift of the structure, yield of the beams and the connections, strength and stiffness continuity, and the degradation at the connection. Each connection type provides a special solution to fulfill the seismic performance requirements considering the previous aspects.

The capacity design method is applied in all the solutions; the concrete structure is verified according to EC8. The principle of weak beam strong column connection is applied. The bending moment in the column beam connections is calculated through the analysis model considering the stiffness of the connected members. The distribution of the seismic loading on the columns is influenced by the connected beams strength. Capacity design accounts for the connected member’s strength and its interaction influence during seismic loading. It account for the tolerance in the material and the dynamic loading. The required seismic design strength in the columns is verified against the action of the connected beams due to seismic loading in a direction and its reversal action. It is a function of the connected members bending moment strength with and overstrength factor and the seismic bending moment proportion to its bending moment resistance, which accounts for a seismic magnification factor. According to the analysis, and using the modified stiffness in the beam ends (Figure 1), the calculated top and bottom reinforcement in the beams results in a suitable proportion to resist the seismic loading without high-applied reversal action on the columns. The minimum reinforcement percentage is applied in the columns. The design of the concrete elements is carried out in three steps:

(1) The preliminary design of the columns, for the applied loads and the minimum required reinforcement in the yield locations.
(2) The design of the beams includes the application of the capacity design for shear considering the moment resistance of the beam-ends.
(3) The design of the columns with the capacity design, using the moment resistance of the beams.

The preliminary design of the columns is used as monitoring measure for the design optimization of the different solutions. In case of a significant change is needed in the column reinforcement due to the application of the capacity design, modifications in the column and beam design may be applied. In the design with monolithic and HFC connections the strength reduction and the over strength factors are applied in the design of the shear in the beams and the columns and the flexural design of the columns, while in the design with DDC and the posttension assemblage, the capacity design principles are applied considering those factors within the design criteria’s. In the design of the connections the main aspects as yield locations, drift, ductility and degradation is considered with a specific solution for each type of connection. The emulated R/C connection is composed of cast in place elements on the column top. Hybrid-fibre concrete connection composed of precast elements connected on the column top and connected with interior precast concrete B55 beams The two connection types provides continuity in the stiffness, stresses, and shear. The yielding is relocated from the column faces in the beams where the ductility is provided. The required high shear resistance at the column faces is designed with conventional diagonal shear. In the design with HFC additional reinforcement in the connection core is applied in order to get a stiff and strong connection while the reduction in the reinforcement at the yield locations is used to get use of its high ductility. The Ductile Dywidag connection and the hybrid posttension connection are attached to the columns with ductile bars passing inside the column core and with posttension tendons passing inside PVC through the column. The plastic hinges in the beams normally occur near the beam-ends; the top and bottom beam bars may yield in tension and compression alternatively at the column faces. The important principle for the adaptation of this connection for seismic action is the relocating of the causative actions by relocate the yielding away from the beam toe, within the column where the confinement of the concrete protect it verse the lateral support action. By posttension assemblage mild reinforcing provides energy dissipation during a seismic event is placed at the top and bottom of the beam through the joint and is grouted in place. By limiting post-yield rotations to the joint, damage to the system is minimized. The design of the emulated monolithic concrete structure with the elastic model used 20% reduction of the resultant beams end moment. The drift of the current floors and the top floor are verified to be less than the permitted drift $\Delta/h \leq 3\%$. The calculation with the two steps is used for the other alternatives. The design with monolithic connections is verified using the non-linear analysis program Ruaumoko using two records scaled to the design shear force of $A_g = 0.24g$. Both design results are verified for degradation, using Takeda degrading and plastic hinges at the column bases. The design results in available ductility in the beams greater than the ductility resulting from the non-linear analysis. The design with the modified stiffness resulted in improved moment curvature hysteresis behaviour in the column beam connection (Figure 2), better distribution of the demand ductility among the members, and better demand ductility proportion of the top and the bottom reinforcement in each member end. The resultant forces in the beam-ends indicate that the top and bottom flexural reinforcement resist with identical stress the seismic action. The moment curvature in the connection (Figure 2) designed with modified effective stiffness model is more homogenous, uniform, and curvature range. It indicates that the connection provides higher available ductility and especially by increasing the confinement reinforcement.

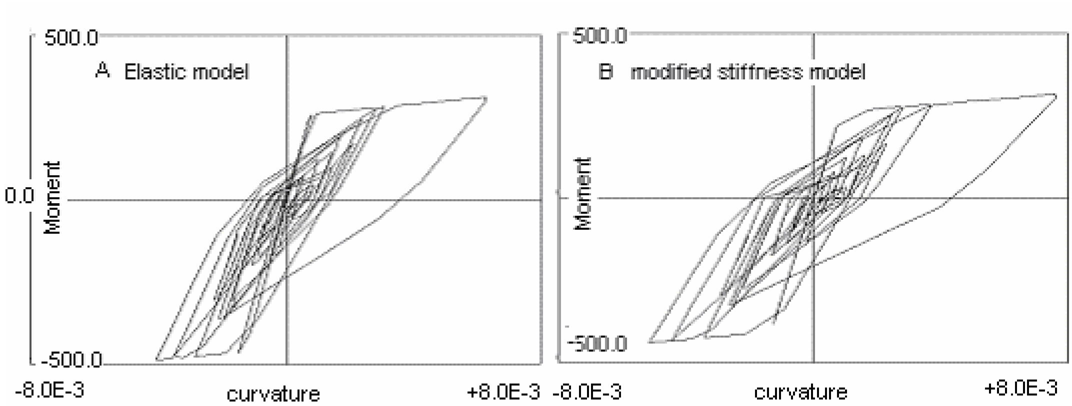


Figure 2: Moment curvature diagram in a connection designed with elastic model and modified effective stiffness model

The design with the modified stiffness results in response increased uniformly in case of increasing the seismic intensity. The top and bottom reinforcement proportionality in the section is an important aspect studied and fined its solution through the application of the stiffness properties in the elastic model, the flexural reinforcement design results in improved connection hysteresis response. The design with the modified stiffness is tested against two seismic records (El Centro 1940, and Bucharest 1977) scaled to the design ground acceleration A_g=0.24g in one model and scaled to 1.6×0.24g =0.38g in other model. The ductility demand in the two models stays within the member design limits. The displacement and drift of the members exceeds considerably the permitted limits, while the flexural response of the section has no considerable increment of the design value when increasing the seismic intensity. The connection possesses stable seismic resistances due to its homogeny response with the seismic actions. We may conclude that building with homogenous seismic resistance connections and sufficient ductility achieved by rational design with accuracy assures the required top and bottom reinforcement proportionality in the sections and the demand ductility (Figure 3). It add a positive influence to increase the structure integrity, the structure keeps its integrity till reaching access in drift. The accuracy determination of the number of bar and its diameter is important practical factor influences top and bottom reinforcement proportionality. The required reinforcement proportion in the sections is decided through the design of the reinforced concrete element using the structural analysis response and applying the capacity design requirements. The introduced connection stiffness properties in the analysis model influence the response. The accuracy needed in the application of the capacity design and structural analysis considering the possible modifications.

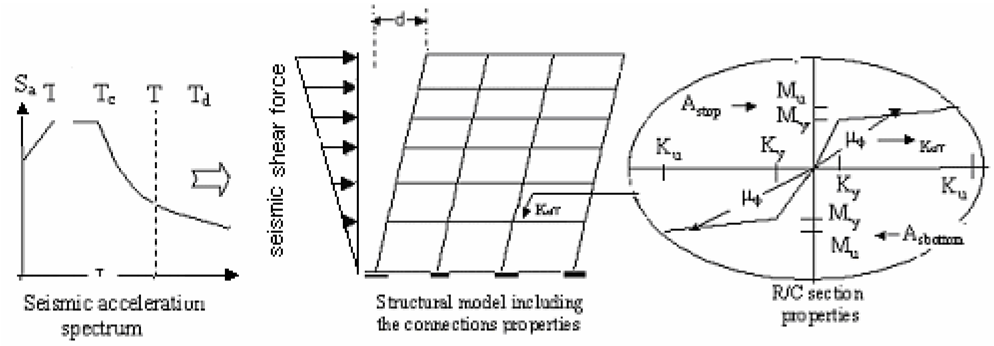


Figure 3: The structure analysis introducing the connections properties

The top and bottom reinforcement ration, its proportionality, and the sufficient available ductility in the beam end determine the effective stiffness for a certain loading level (Figure 3). It influences the response of the structure and the applied factors in the capacity design. Its accurate application leads to seismic resistance stability in the connection (Figure 2). The demand ductility and the displacement in the structure may increase considerably while the connections still possesses sufficient strength resistance and hysteresis behaviour against the seismic action.

The design and analysis procedure is applied in the solution of the four connection types, and the analysis of the structure proved its efficiency to obtain the response, the links with the specific connection requirements. The analysis with the modified elastic model used the R/C monolithic properties of the beam-ends beyond yield that governs the determination of the seismic structural behaviour. It provides a good assessment for the structural displacement and drift with factor incused its application in the connections region. The solution provides an answer to the required connection stiffness for the required seismic structural displacement. That enables using the structural analysis results as a general solution for the different connection types. The ductility demand is used in the analysis as a measure for yielding. The demand ductility is provided by the structural analysis is verified against the available ductility provided in the connected members for all the connection types. It varies within the available ductility in the connected members.

The design and the choice of a connection type requires information and knowledge covering analysis, function, material properties and its usage, the possible additional ductile members or posttension rods. The technique and construction process of the building and its application in the connection is an important factor. The use of more than one connection type in a building provides the application of specific needed character in a specific location. It requires the determination of the needed stiffness, strength, ductility in the connections and the application of the suitable connection type that provides the better solution. The usage of the ductile rods in the hybrid connection may be reduced in case of no need for high shear and bending moment resistance. In a building, for the first second and third floor construction with Dywidag or posttension assemblage is preferable where high ductility is required, while the 4th and the 5th floors may be constructed with emulated monolithic R/c or HFC. The bending moment, and the resistance moment in the beam-ends is important factor that influences the behaviour of the connection. It can be performs through the design with reasonable redistribution of the response considering the postyield behaviour of the reinforced concrete, the variation of the stiffness and the strength as a function of seismic, and the intervention of the designer to apply stiffness and flexibility in location that lead to improve the response.

Table 1: Description and comparison of the connection characteristics

Connection type	Function	Resistance		
		Shear	Bending moment	Ductility
R/C	Integrity and continuity of the building due to the use of the same material and regulations. Possible mixing with (DDC) and (PT), by adding extra posttension rods or ductile members to increase the shear resistance and its ductility. Yield locations are relocated from the column face.	+++	+++	++
Hybrid HFC	Provides more ductility, high shear resistance. Yield locations are relocated from the column face. Possible mixing with (R/C), by using the R/C in the locations where no high shear resistance or ductility is required.	++++	++++	+++
DDC	Ductile members provide high ductility in the column faces. Extra moment, transfer the acting force inside the column, enables reversal action. Shear resistance due friction caused by the pretension of the ductile members	++	++++	++++
Post tension (PT)	Add pretension force on the column increasing the shear resistance due to the friction. Dissipation of the energy is provided by the addition of the ductile rods, which provides flexural moment also.	++	+++	++++
+ Is a relative comparison degree of the resistance provided in each connection type In the design with R/C and HFC yielding is relocated from the column face in the beams. In the design with DDC and Post tension assemblage yielding occurs in the beams close to the column face in location protected against degradation.				

Comparison and description of the connection characteristics are given in Table 1. The design with R/C connection possesses continuity in the mechanical properties of the materials, and the distribution of the response on the connected members. The reinforcement is continuing in the core and the beam. The concrete stresses are extending to the stresses in the core and the beam. The local force actions can be minimised by suitable calculation, distribution and selection of the reinforcement. It provides high shear resistance and moment strength. The design for ductility requires the attention in relocation for the yield locations. Construction with HFC requires the accurate design of the bending moment reinforcement to reduce the exerted seismic reversal beams action on the columns. The use of high reinforcement ratio in the HFC cross-section increases considerably its moment resistance. According to the capacity design the increase of the strength in the connected beam end leads to apply high seismic moments on the columns The core of the connection requires extra reinforcement to assure its resistance against cracking, and yielding of the beams away from the column face.

1 Introduction

The development of the seismic connections is essential in the precast construction. Most of the precast concrete constructions adopt connections details emulated cast-in-place concrete structures such that they should have equivalent seismic performance as monolithic concrete members. While other connection types with using ductile members is generated. The precast concrete structures are composed of elements jointed together and form the construction for specific functioning and requirements. In order to realize such requirements different problems should be solved. The question how to construct with these elements, is a subject to be study on two major levels: the building connection system, and the connection of the elements, which are related together, and it has been studied experimentally and theoretically. The system of connection may influence the connections type. In the design of the connection the investigations of the influence of the seismic effect on the joint and the neighboring regions in the connecting elements is required. The progress in the technical solutions at the connections and the performance of the connections leads to generate new building systems, and create real problematic which can't be separated from the connection problems and its progress. The connection performance in a structure should cover the connection joint, the yielding locations, and its relation with the connecting members and the overall system. In seismic resistance structures, the behaviour of the end members and the connections due to the seismic action is important.

1.1 Prefabricated concrete constructions under seismic effect

The non-linear behaviour of the reinforced concrete structures, under seismic action depends essentially on the inelastic deformations and distribution in the structure and on the behaviour of the member under the influence of the different combined forces including alternative seismic action. The structural system and the loading are major factor influences the final response of the structure, and the function of the member. In the precast structures, the concrete elements are connected at their ends in different manner. The degree of fixation of these elements varies from free to rigid connection, which enables to construct considering suitable degree of freedom and rotations in the elements. The redistribution of the inelastic deformation should be considered in the analyses and the design of the structure. In seismic analyses it is important to define the principle aspects such as: drift, ductility, and stiffness of the construction and assess its range of change especially in the elements ends. Seismic effect is more effective in the connection regions of the prefabricated elements, and it is important that this effect will be limited within the reserve resistance capacity. There are needs to define criteria's and to formulate and establish analyses and design methods governing the resistance capacity and the nonlinear deformations at the elements, and the structure level. Simple and explicit relations and methods are helpful to take decisions concerns the building system. In the design and the choice of the prefabricated elements, the compatibility, the functioning of the elements, the connection type and dimensions of the different construction elements governs.

In the choice of an element, the resistance capacity varies according to the reinforcement distribution in the middle spans and in the connection zone, which influence the elements properties. The erection and execution method plays a role in structural modeling and solutions and then on the resultant response of the different force combinations acting at the connection. An optimum solution is to design elements possesses properties satisfying the serviceability and ultimate limit state requirements, and to limit the seismic effect in the elements resistance capacity. The design of the connections locally is a question to solve with engineering judgment covering the local details and the general relations in the building system. It requires the determination of the requirements of the system functioning to resist seismic in combination with the serviceability of the building, and the distribution of the seismic influence on the components, the elements and then on the connections. In the connection different solutions are available, which may influence the whole system.

1.2 Objective

The objective of the thesis is the influence of the connection characteristics on the seismic performance of the precast concrete structures. The behaviour of the members in the reinforced concrete studies leads to the conclusion of the major influence of the members end connections on its total behaviour. In seismic resistance structures, the non-linear behaviour of the end members due to the seismic action is significant. There is need to study the performance of the structural elements and especially its connection. In this study the characteristic of the connections, the behaviour of the structure and its response is investigated. The relations between the characteristics of the connection, the post-yield locations and the response of the structure under seismic loading are one of the major points in the study towards solving the performance questions of the precast concrete structures.

1.3 Scope of the study

The study objective is the influence of connection characteristic on the seismic performance of the precast concrete structures is discussed in chapter 1. The discussion explained that the usage of the precast concrete elements arise performance problems, the concrete, the seismic action and the connections are essential parts in the seismic design where the end members and the connection is an important subject for study. A study plan and scope of the study is given in the same chapter.

The precast construction is discussed in chapter 2. It includes seismic resistance building system, different types of connections and solutions. It shows the use of ductile elements to obtain ductile seismic resistance frame, wall, and diaphragm. The lateral force resisting system is distinguished in frame, wall, dual wall, core and inverted pendulum systems. Seismic resistance systems are illustrated with different solution covers the ductility, detailing for ductility, steel arrangement and dimensioning in the connections. Solutions involves ductile connections are discussed for the connections with ductile members and post tension tendons. Dissipation of the energy is illustrated using of ductile member, mild steel rods in connections, and dissipation by concrete cracking and steel deformation in hybrid fibre concrete.

The earthquake phenomena and the seismic effects on the structures are discussed in chapter 3. The discussion covers the dynamic analysis principles, and the non-linear analyses methods to quantify the seismic forces on the structures. The structure behaviour under the hazardous seismic action depends on two basic parameters; the intensity of the earthquake, and the quality of the structure. The terms magnitude, intensity and damage are used as a measure for the earthquake and its consequences.

Aspects in the design and test of precast concrete seismic resistance structures are reported in chapter 4. The chapter provides experimental and theoretical information covering shear resistance, bar anchorages, and stiffness deterioration in the connections. Different issues are explained as the importance of beam-column joint detailing in moment resisting frames, the significant increase in flexibility of the frame due to diagonal shear cracking, the usage of the transverse reinforcement to limit the maximum tensile stress of the concrete. The plastic hinges formation is explained as a consequence of yielding and its occurrence near the beam-ends. In order to create possible plastic hinge location yielding is allowed to take place in some locations by reducing the flexural reinforcement. The inelastic deformation in the structure is required without significant loss of strength; the ductility in the R/C sections covers this requirement, and the ductility of the beams and the columns can be increased considerably using confinement reinforcement.

The capacity design concept is discussed in chapter 5. The discussion covers the capacity design method, its principles and its formulation according to EC8. The capacity design method states that ductile behaviour in a structure requires that the yield capacity is reached first in ductile response modes rather than in brittle modes. The chapter illustrates methods to fulfil this requirement using the EC8 code, which allows designing using three structural ductility levels. A higher the ductility leads to a lower of the design force and thus a lower the required strength. EC8 approach aims defining the global mechanism, through the so called beams sway mechanism, in which all beams at all stories form plastic hinges, while all columns remain elastic for their entire height.

The non-linear analysis method is discussed in chapter 6. The discussion includes degradation of the concrete structures under seismic loading and methods and rules used in the calculations. It is shown that the different between dynamic problems are different from static ones in different factors to be considered as: The inertial force, damping, strain rate effect, and oscillation. Dynamic tests of real buildings are rather aimed toward obtaining data to confirm the validity of mathematical modelling techniques for a linearly elastic structure, and to obtain damping characteristics of different types of structures. The energy-dissipating mechanism in the structures is shown to take the following forms: Inelastic hysteretic energy dissipation, radiation of kinetic energy through foundation, kinetic friction, viscosity in materials and aerodynamic effect. And the speed of loading is known to influence the stiffness and strength of various materials, the strain rate during an oscillation is highest at low stress levels, and the rate gradually decreases toward a peak strain. In the chapter it is shown that the hysteretic model is able to provide the stiffness and resistance under any displacement history, which needs the basic characteristics definitions of the member geometry and material properties. Different hysteresis tests and rules are illustrated to define the flexural, shear and bond cyclic behaviour of reinforce concrete connections.

A new method for the non-linear behaviour of unconfined reinforced concrete section is formulated in chapter 7, and it is applied on the reinforced concrete B55 and the Hybrid-fibre concrete. It includes the definition and mathematically formulation of the compression and the tension parameters for a rectangular reinforced concrete sections. Figure and tests are included showing that hybrid fibre concrete (HFC) possesses high ductility and performs well under cyclic loading. A non-linear behaviour model for rectangular R/C cross section is built for concrete B55, and for hybrid fibre concrete. The compression and tension behaviour of the concrete are defined by parameters enabling the substitution of the stress strain relations into the equilibrium equations. For the representation the post yield response of the member's properties the complex behaviour of the steel and concrete are considered, such as the non-linearity of the concrete and steel.

The stiffness factors are formulated for a rectangular reinforced concrete section in chapter 8. A precast concrete beam model is built. The model shows the significant influence of the applied moment in the post yield situation on the stiffness in the member ends, which leads to introduce the structural model with semi rigid connections applying a new structural analysis model using modified effective stiffness. The application of the relation load-stiffness-reinforcement ratio-ductility-enables building model for the precast concrete beam. Introducing that in a concrete structure subjected to seismic action shows that the use of the effective stiffness obtained from the local section properties can provides description for the structure under seismic loading, which may be used in the design as a redistribution method for the applied bending moment in the beam ends. Description of the stiffness, its relations, and application are included.

Project of 6 storey precast concrete building is introduced in chapter 9. The dimensions, the construction requirements, the load calculation, and the premier design considerations are discussed. Seismic loading is calculated using the equivalent shear method. The equivalent shear method is applicable to our building with regular shape and a uniform distribution of mass and stiffness. The chapter includes the distribution of the lateral forces on the structure elements.

The design with emulated monolith connections is introduced in chapter 10 with three alternatives, the hybrid connection, the DDC connection and the hybrid fibre concrete connection. It is shown that materials, ductile members, and special technique are used in the connection to improve its shear, moment resistance and ductility during the seismic loading. And the connection it self as an intersection between at least two elements has its problematic to be solve, as the shear, and moment resistance, the integrity with the supports and the ductility. The connections are divided in system connection considering factors as moment, shear, dimension to be considered and member connection where the performance for the previous mentioned factors may be simplified. The design procedure is carried out by splitting the problem to find economical design in which local properties and solution can be applied as the use of the ductile members to increase the connection ductility, and the use of the prestressed elements to produce shear by friction.

Premier design of the building is introduced in chapter 11.The loading on the building and the calculation of the lateral seismic force on each level is included. The analysis of the structure is applied with two analysis models, the elastic model considering the serviceability limit state and seismic combinations and introducing the connection characteristics considering the seismic combinations. According to the design requirements the drift of the current floors and the drift of the top floor is considered to be less than the permitted drift $\Delta/h \le 3\%$. The shear force on the building is distributed on the interior column and the exterior columns in a manner lead to decrease the moments at the exterior columns. The decrease the bending moments at the first floor beams is carried out applying yield locations at the first floor and the use of different degree of fixation in the columns supports to get more rigid connection in the interior columns and flexible connection at the exterior columns. The design optimisation is illustrated using the modified stiffness model and the non-linear analysis with Ruaumoko program.

The design of the emulated monolithic concrete structure according to EC8 is given in chapter 12. Appling the capacity design requirements for the interior and exterior columns beams connections. In the design of the ductile connections (DDC, Posttension assemblage) the capacity design is included in the design procedure of the connected elements- the beams and the columns. In the designing of ductile moment resisting frame subjected to lateral loads, the design for the flexural moment in the beams includes the design of the yield location locations, the shear, and the control of the possible plastic hinge formation at the beam-ends. In the first assessment for the ductility the local curvature ductility in the beams ends are obtained using the q factor obtained from the design spectrum and apply relations given by the CEB for the design period T. The design of the concrete structure is optimised using the modified elastic stiffness, which is built based on the elastic frame model reinforcement. The resulting bending moment, at the column beam connections with modified stiffness model is about 80% of that which is obtained with the elastic model results, and the design reinforcement in the beams and the columns are identical

The design of the connection with Ductile Dywidag Connection DDC, Hybrid Post Tension Connections and Hybrid Fibre Concrete HFC is introduced in chapter 13. The chapter covers concepts, calculation methods and the design processes. In the different alternatives design the response used is the structural analysis is carried out using the elastic stiffness model with 20% reduction of the moments in the beams ends. Hybrid fibre concrete (HFC) is used in the column beam connections, as a precast unit supported on the column. In the connection core extra flexural reinforcement is added to increase its resistance. Accuracy is important in the flexural reinforcement design to limit the influence of the reversal seismic forces exerted by the beams on the columns. In the design with DDC connection the relocating of the causative actions is by relocate the yielding element within the column where the confinement of the concrete protect it verse the lateral support action. The calculations indicate that nominal moment capacity of ductile member group decided the transfer load on the column, and the nominal shear capacity in the connectors. It is shown that the applied force on the ductile connector is unaffected by the preloading level, and the connecting members should be logically verified for the uncertainties associated with each of the considered transfer mechanisms. The Hybrid beam system consists of concentric post-tensioned cables anchored at both ends of the frame, and the clamping force creates a friction force between the beams and columns, which transfers the shear demands. Mild reinforcing provides energy dissipation during a seismic event is placed at the top and bottom of the beam through the joint and is grouted in place. The calculations shows that the restoring force provided by the elastic post tensioned and the flexure strength is provided by combination of unbounded post tensioning strand and bonded mild steel. The chapter shows that factors should be considered and verified are the strand stress, its nominal moment, and the flexural moment provided by the mild steel, and the friction due to the compression force of the tensioned tendons provides the connections shear resistance.

Summary and evaluation of the study is introduced in chapter 14. The evaluation of the study provides description for the different activities, the solutions, and the main applied concepts. The design summary with the different solution, and comparisons between the different types of connection are reported. Conclusion is included it shows that in the design information, knowledge concerns the analysis and the function of the connection parts is required. The use of different connection types in a building enables to control the specific needed characters in a connection, and the characteristics of one connection type may be used in the a specific design requirement.

2 Precast Concrete

Keywords: Precast and prestressed concrete structures, beam column joints, lateral force resisting system, ductile equivalent monolithic resisting frames, jointed reinforced concrete resisting frames, ductile hybrid jointed moment resisting frames, equivalent monolithic reinforced concrete structural walls, jointed reinforced concrete structural walls of limited ductility, ductile hybrid structural walls, diaphragms.

Lateral force resisting systems can be distinguished in Frame systems, Wall systems, dual wall systems, core and inverted pendulum systems. Precast concrete built in equivalent monolithic and jointed system with ductile, and limited ductility connection system. The development of a specific connection type requires specific details in the connection as: covers, dimension, connected members arrangement, the flexural and the shear reinforcement, and the used ductile elements. Design studies lead to specific ductile equivalent monolithic moment resisting frames for column-to-column, beam-to-beam, and wall-to-wall connection, and for the column-to-beam, wall-to-floor connection. The reinforcement at the column ends are used with lap splits bars that can be protected by capacity design approach to prevent plastic hinging occurring in the column ends, relatively weak connection achieved mainly by welding or bolting. Hybrid connection system combined unbounded-tensioned tendons with longitudinal non-prestressed reinforcing bars. The post-tension tendons may run directly into the connection core or runs inside a PVC through the center of the beam and the core. The tendons prestressed so that it will not reach the proportionality limit during the earthquake. These bars provide energy dissipation by yielding in tension and in compression over unbounded regions next to the beam-to-column connection. The system has the advantage of reduced damage during an earthquake. Bolted assemblage Dywidag ductile connectors were motivated to improve the post-yield behaviour of the concrete ductile frame. The design with steel fibre reinforced concrete has not been yet applied in a wide range. The studies indicate that it perform well in seismic design due to the confinement provided by the steel fibre, and hence improve its toughness and ductility. The energy is dissipated by concrete cracking, steel deformation, steel bending etc. and via progressive fibre pullout from concrete. Fibre concrete can sustain a portion of its resistance following cracking to resist more cycles, the fibre concrete joints had better energy dissipation, ductility, and stiffness, as well as spalling than plain concrete joints. In many part of the world the precast and prestressed concrete possessed successful applications in earthquake resisting building and structures.

The construction and the design of earthquake resistance structures in precast and prestressed concrete required considering important aspects in the connection design. Due to poorly designed precast and prestressed concrete structures have badly performance in seismic regions. In major earthquake failure occurs due to brittle behaviours of poor detailed connections between elements, poor detailed of elements and poor design concept. Examples of major damage to poorly designed and constructed precast concrete structures as a result of severe earthquakes are shown in Figure 2-1. This has resulted in the use of precast concrete in earthquake resisting structures being regarded with suspicion in some countries [11].

Figure 2-1 Collapse of poorly designed and constructed precast concrete building

Experience has shown that structures incorporating precast concrete elements, which are well designed and constructed for seismic resistance will perform well in earthquakes [12]. The designer and erector must possess knowledge of the capabilities and limitations of precast element processes, and an understanding of how the structure will be safely assembled.

For the design of precast concrete structures there are several important rules to be applied as follows: Detail as simple as possible. Design for maximum repetition. The shape of the elements should be as simple as possible. Design the element as big as possible. Realize that the elements must be transported from the factory to the site. Avoid pouring concrete on site; the structure must be stable during erection. Provide for tolerance. Use standard dimensions, details, cross sections, connections and base type products [5]. Many codes contain provisions for the design of precast concrete in seismic regions to various degrees. For example, seismic provisions for precast concrete are scattered throughout the New Zealand concrete design standard [13]. The ACI building code in its 2002 edition [14] has seismic provisions for precast concrete but does not permit the option of non-emulative precast wall structures. The Architectural Institute of Japan has published draft design guidelines for precast construction of equivalent monolithic reinforced concrete buildings [15].

2.1 Lateral force-resisting systems

Lateral force resisting system in general can be divided in five systems:

- Frame system: Where the space frame mainly resists both the vertical and lateral loads.
- Wall system: Where the loads are resisted by wall system coupled or uncoupled with high shear resistance.
- Dual wall frame system.
- Core system: Composted of wall or frame system without satisfactory torsion rigidity.
- Inverted pendulum system: Where 50% of its mass is located in the upper height of the structure.

Precast concrete element in general built in equivalent monolithic or jointed system. They are distinguished according to the type of connections between there elements.

- Equivalent monolithic systems (ductile): The connections in equivalent monolithic system are designed to emulate the performance of the cast in place concrete constructions. They are ductile connection that have the required strength and can yield.
- Equivalent monolithic systems (Limited ductility): They are strong enough but they are of limited ductility designed to remain in the elastic rang during the seismic actions, while the building is satisfying the ductility demand imposed by severe earthquake.
- Jointed system: The connection in jointed system does not emulate the performance of cast in place concrete constructions.

2.2 Ductile equivalent monolithic R/C moment resisting frames

(a) Arrangement of members:

Equivalent monolithic ductile reinforced concrete construction systems used in Japan, Architectural Institute of Japan [15] and New Zealand, Centre for advanced Engineering [16] designed for weak beam-strong column behaviour are shown in Figure 2-2. In all systems a precast concrete floor system is placed seated on the top of the precast concrete beam elements and spanning between them.

System 1: The precast concrete beam elements are placed between either precast or cast-in place concrete columns, seated on the cover concrete below and/ or propped adjacent to the columns. Reinforcement is then placed in the top of the beams and in the beam-column joint cores. One approach with this system, typically used in New Zealand, is to splice the beam bottom bars using hooked anchorages in the joint core (see (a) of System 1). An alternative approach, typically used in Japan, is for the bottom bars to be continuous through the joint (see (b) of System 1). The faces of the precast beam elements are either keyed or roughened

In System 2: The vertical column bars of the column below the joint protrude up through vertical corrugated steel ducts in the beam unit where they are grouted and pass into the column above. The beams are connected using a cast-in-place concrete joint at mid span.

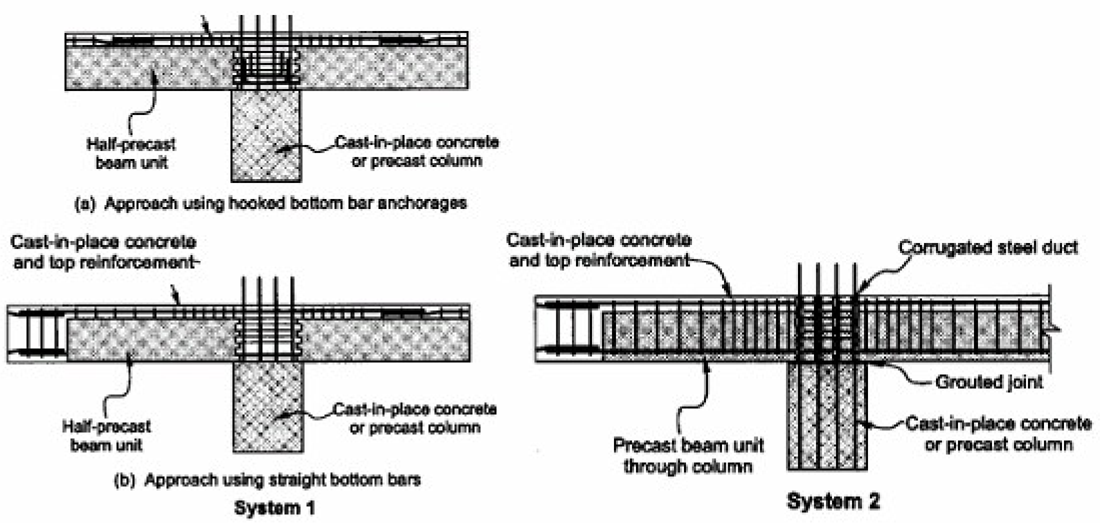


Figure 2-2 Ductile equivalent monolithic R/C moment resisting frame system, system 1and system 2

In System 3: T-shaped or cruciform shaped precast concrete elements are connected vertically by column bars and horizontally by a cast-in-place concrete joint at mid span.

System 4: is mainly used in New Zealand, pretension prestressed concrete U-beam and cast in- place reinforced concrete are used refer to Figure 2-3.

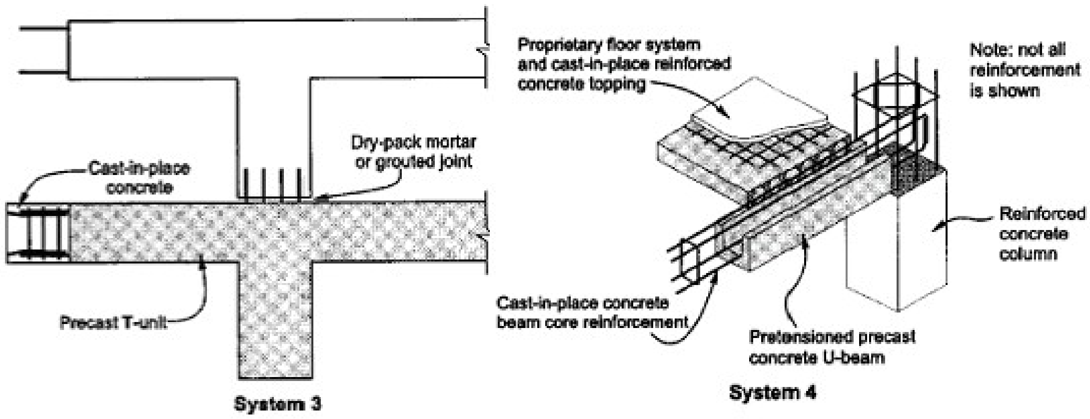


Figure 2-3: Ductile equivalent monolithic R/C moment resisting frame system, system 3,and system 4

(b) Column-to-column and beam-to-beam connections:

The precast reinforced concrete elements in Systems 2 and 3 can be spliced either at the end above the beam or at mid-height either grouted steel sleeves, or non-contact lap splices involving refer to Figure 2-4, the longitudinal bars protruding from the precast column bars “G” are grouted into corrugated metal ducts in the mating column. Adjacent to the ducts there are two smaller diameter bars, bars “L”, of about the same cross sectional area as the grouted bar. Those bars are lap spliced a distance not less than the same time as the ducts. Note that splices at the end of columns can be protected by a capacity design approach to prevent plastic hinging occurring in the column there.

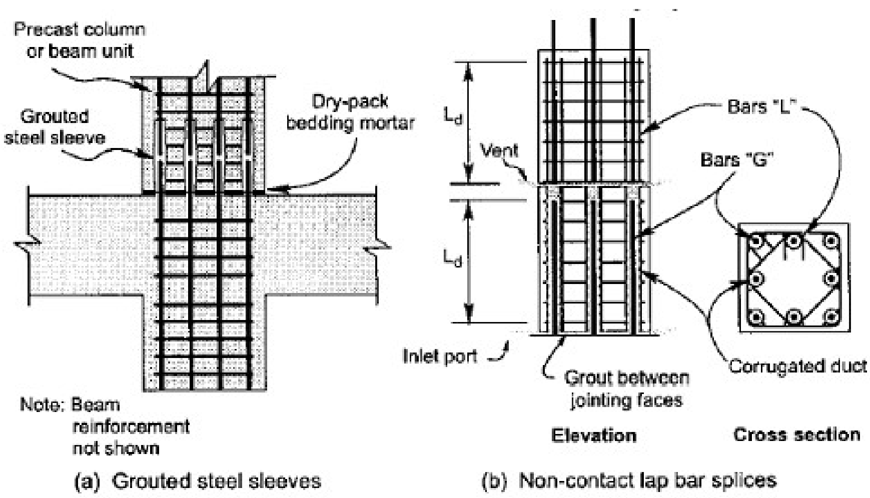


Figure 2-4: Column to column reinforced concrete connection

2.3 Jointed reinforced concrete moment resisting frames of limited ductility

Some jointed reinforced concrete moment resisting frame systems that have relatively weak connections between precast elements, achieved mainly by welding or bolting and dry packing, have been attempted. Such systems should be used with caution since they can have very limited ductility.

2.4 Ductile hybrid and Dywidag jointed moment resisting frames

Hybrid systems are jointed systems, which combine unbounded post-tensioned tendons with longitudinal non- prestressed reinforcing bars. Hybrid systems were developed in the United States and adopted in the PRESSS (Precast Seismic Structural Systems) programme (Priestley et al)

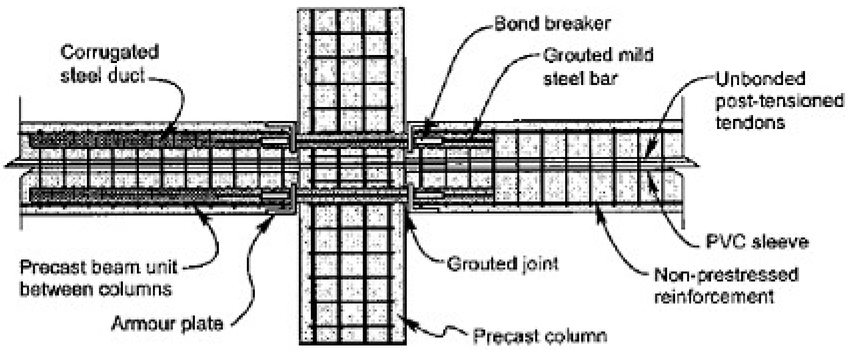


Figure 2-5: General reinforcing details of hybrid frame system [17]

Figure 2-5 shows an example. The precast concrete beams are connected to multi-storey columns by unbounded post-tensioned tendons that run through a PVC duct at the centre of the beam. The gap between the beam and the precast column is filled with fibre-reinforced grout. The tendon prestressed, such as to ensure it will not reach the limit of proportionality during an earthquake. Non- prestressed steel reinforcing bars are placed in corrugated ducts on the top and bottom of the beam and through the columns and grouted. These bars provide energy dissipation by yielding in tension and compression over rebounded regions next to the beam-to-column connection. The non-linear deformations come from the opening and closing of the crack at the interface between the precast concrete elements. The system has the advantage of reduced damage during an earthquake and of been self centring i.e., practically no residual deflection after an earthquake.

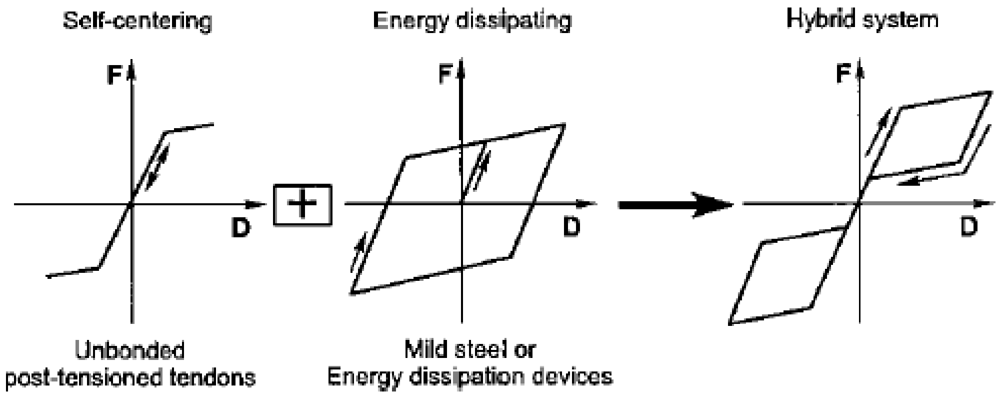


Figure 2-6: Idealized flag-shape hysteric rule for hybrid system [12]

Figure 2-6 shows the idealized flag-shape hysteretic rule for a hybrid system composed of the sum of the self-centring contribution of the unbounded post-tensioned tendons and the energy dissipation of the mild steel reinforcing bars [12]. Bolted assemblage Dywidag ductile connectors DDC were motivated to improve the post-yield behaviour of the concrete ductile frame. The DDC system has the ability to deform for a storey-drift of over 4% without a significant loss of strength (Figure 13-4). They introduce a ductile road or fuse into the path away from the toe of the beam. These connectors allow post-yield deformation where the members are jointed. The desired behaviour of the structure achieved through merging the steel technology with the basic objective of seismic loading limitations.

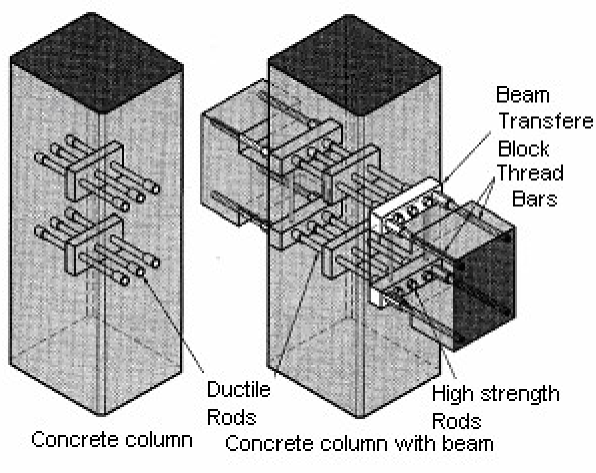


Figure 2-7: Bolted assemblage, Dywidag ductile connectors (DDC).

The important principles for the adaptation of this connection type for the seismic action is the relocating the causative actions by; Relocate the yielding element to within the column where the confinement of the concrete protect it verse the lateral support action, allow the strain in the toe region of the beam to be verified, and transfer shear force by friction from steel to steel. In the design of the ductile connectors the capacity design principles are applied trough considering the strength reduction factor and the over strength factors.

2.5 Seismic joints of Steel fibre reinforced concrete

The design with steel fibre reinforced concrete has not been yet applied in a wide range. There are literature studies available over the properties of this material and its behaviour under cyclic loading. These studies indicate that this material perform well in seismic design. The main reason is the confinement for the concrete provided by the steel fibre, and hence improve its toughness and ductility, which is one of the important concepts in the seismic design. In earthquake loading, in conventional joint the energy is dissipated by concrete cracking, steel deformation, steel bending etc. In steel fibrous joints, the goal is to dissipate such energy via progressive fibre pullout from concrete. Earthquake causes seismic joints to be subjected to multiple cracking in reverse cycles of loading. Conventional concrete looses its resistance completely after cracking. However, fibre concrete can sustain a portion of its resistance following cracking to resist more cycles. It was found that the fibre concrete joints had better energy dissipation, ductility, and stiffness, as well as spalling than plain concrete joints.

2.6 Equivalent monolithic reinforced concrete structural walls

New Zealand practice for horizontal and vertical joints in monolithic structural wall incorporated precast concrete construction is discussed below. At the horizontal joints between precast reinforced concrete wall panels or foundation beams the ends the panels are usually roughened to avoid sliding shear failure and the joint made using mortar or grout. The vertical reinforcement protruding from one end of the panel and crossing the joint is connected to the adjacent panel or foundation beams by means of either grouted steel splice sleeves or grouted corrugated metal ducts much as for the column-to-column connections in Figure 2-5.Vertical joints between precast concrete wall panels are typically strips of cast-in-place concrete into which horizontal reinforcement from the ends of the adjacent panels protrude and are lapped. [10]

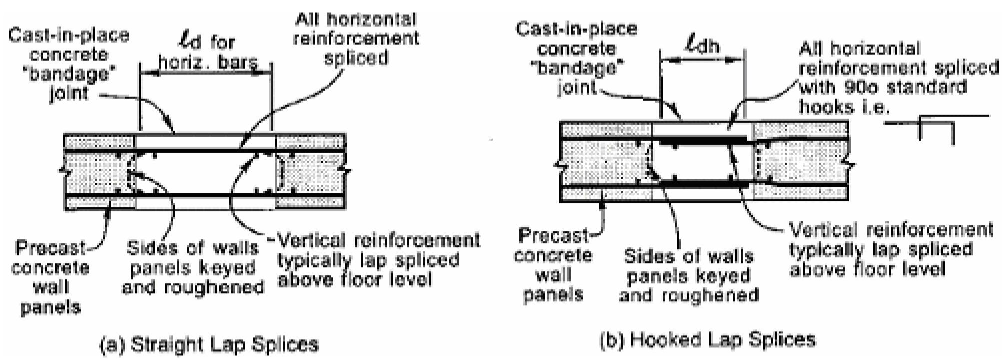


Figure 2-8: Vertical joints for equivalent monolithic precast reinforced concrete structural wall construction type a and b [12]

Figure 2-8 shows in (a), joint with sufficient width to accommodate the lap splice length of the straight horizontal bars that protrude from the precast wall panels. In (b) shows hooked lap splices that enable the width of joint to be reduced.

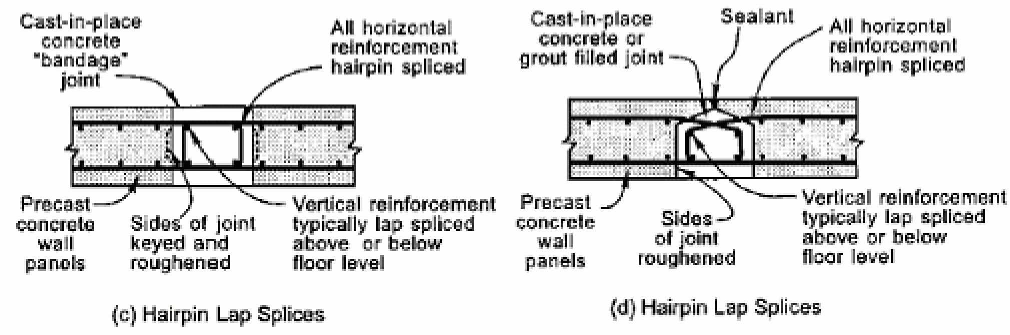


Figure 2-9: Vertical joints for equivalent monolithic precast reinforced concrete structural wall construction type b and c. [12]

Figure 2-9 shows in (c) and (d) hairpin splice bars which may not be convenient to construct since once the lapping bars have been overlapped the ability to lower the precast panels over starter bars is very restricted. Support for precast floor units at walls can be achieved in a number of ways.

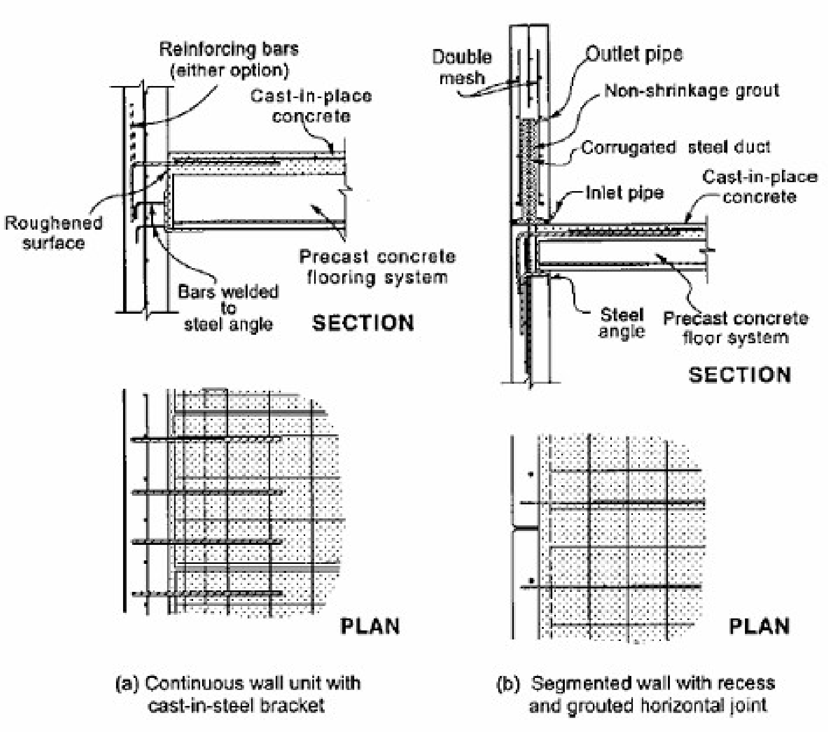


Figure 2-10: Examples of precast reinforced concrete exterior wall-to-precast floor connections

Two examples for the exterior wall is shown in Figure 2-10 for the exterior wall. The figure shows in (a), the precast wall panel is continuous through the connection and the floor is seated on a steel angle anchored to the precast wall panel. Alternatively a concrete corbel could be used. In (b), the precast wall panel is segmented and the floor is seated on a recess in the wall. Hybrid structural walls are jointed walls, which contain unbounded post-tensioned tendons with longitudinal steel reinforcement and/ or other energy dissipating devices. They were developed in the United States and adopted in the PRESSS Programme (Priestley et al [17]). The idealized flag-shaped hysteretic rule also of the hybrid wall is shown in Figure 2-12.

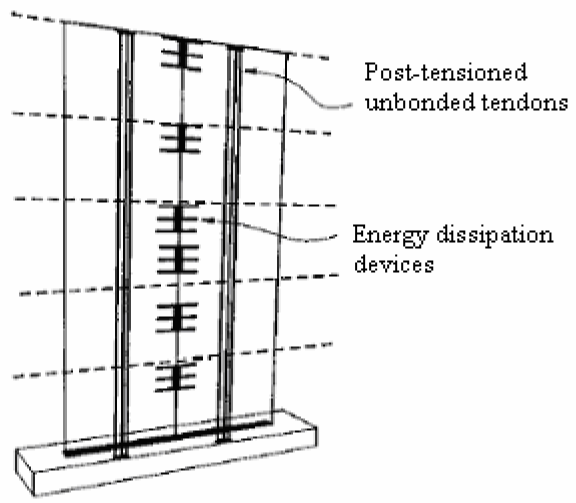


Figure 2-11shows a wall system adopted. The post elastic demand is concentrated at the joint

Figure 2-11: Hybrid structural wall system developed in the PRESS program (Priestley et al [17]).

2.7 Diaphragms

As well as carrying gravity loading, floors and roofs need to transfer the in- plane imposed wind and seismic forces to the supporting structures through diaphragm action. A convenient way to achieve diaphragm action when precast concrete floor and roof elements are used is to provide a cast-in-place reinforced concrete topping slab over the precast units. Where precast concrete floor and roof units are used without an effective cast-in-place concrete topping slab, in-plane force transfer due to diaphragm action must rely on appropriately reinforced joints between the precast units. In Italy topping slabs may not be preferred [12].

Adequate support of precast concrete floor and roof units is one of the most basic requirements for a safe structure. The units should not lose their support. One source of movements during severe earthquakes, which could cause precast concrete floor and roof units to become dislodged, is that the beams of ductile moment resisting frames tend to elongate when forming plastic hinges, which could cause the distances spanned by precast concrete floor and roof members to increase. This elongation may be in the order of (2 to 4)% of the beam depth per plastic hinge (Centre for Advanced Engineering, [16]) as has been observed in tests where expansion was free to occur. This elongation occurs because of the extension of longitudinal reinforcement in the beams due to plastic strains. It may cause tearing away of diaphragms in extreme events [12].

Two possible support types for precast concrete hollow core or solid slab flooring units seated on precast concrete beams are identified by the New Zealand guidelines (Figure 2-12). Notice that the connections can be divided into member-to-member connection; column-to-column, beam-to-beam, or the wall to wall, and to system connection, where more members are connected of different functions as column beams connection or floor walls connection which is required more complicated detail.

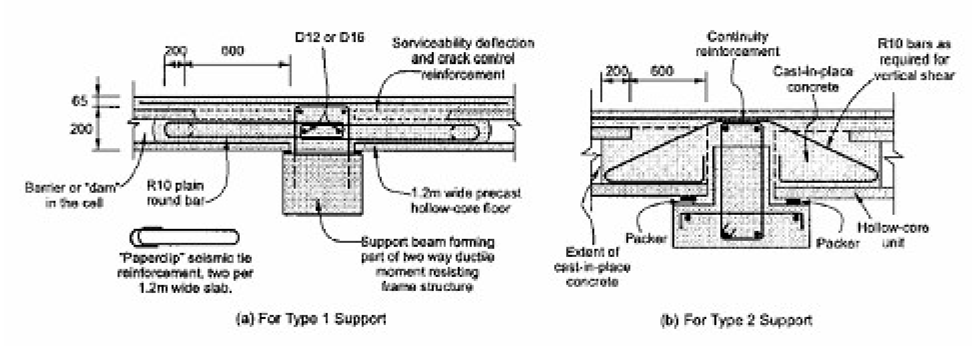


Figure 2-12: Examples of support and special support reinforcement at ends of precast concrete hollow core floors (Center for advanced engineering [16]).

3 Earthquake resistance structures

The behaviours of the structure under seismic action mainly depend on two basic parameters: the intensity of the earthquake, and the quality of the structure. The earthquake is hazardous, stochastic phenomenon and consequently long-term records are needed to express it. The magnitude of the earthquake is a measure of the ground motion in terms of the energy. The term earthquake intensity is a measure of the consequences that this earthquake has on the people and the structures of a certain area. The damage is usually qualitatively estimated using empirical intensity scales. An earthquake has only one magnitude but different intensities from place to another. In Europe an empirical attenuation law for the ground acceleration has been proposed. Response spectra are used in the seismic analysis and design of the structures due to the earthquake influence, it present the maximum response of the structure to a given ground motion. The response of a linear structural system could be evaluated using the Duhamel integral, which require the use of numerical methods. A very useful numerical integration technique for problems of structural dynamics is the so-called step-by-step integration procedure. In this procedure the time history under consideration is divided into a number of small time increments. During a small time step, the behaviour of the structure is assumed to be linear. As non-linear behaviour occurs, the incremental stiffness is modified. In this manner, the response of the non-linear system is approximated by a series of linear systems having a changing stiffness. A structure that can vibrate only in one particular mode can be represented as a single-degree-of freedom system, where the mass, the viscosity, the stiffness and the applied force are required. The numerical equations required to be evaluated and the non-linear response can be developed, considering the equation of dynamic equilibrium. Nonlinear static analyses can use the same solution procedure without the time related inertia forces and damping forces. The equations of equilibrium are written in matrix form for a small load increment during which the behaviour is assumed to be linear elastic.

3.1 Elements of engineering seismology

3.1.1 Origin of earthquake and its recording instruments

Earthquakes are ground vibrations caused mainly by the fracture of the earth crust or by the sudden movement along an already existing fault (tectonic earthquake). According to the elastic rebound theory (Reid 1906), earthquakes are caused by sudden release of elastic strain energy in the form of kinetic energy along the length of geological fault. The accumulation of strain energy along the length of geological faults can be explained by the theory of motion of lithosphere plates into which the crust of the earth is divided. These are developed in oceanic rifts and they sink in the continental trench system (Papazachos, 1986, Strobach and Henk, 1980). There are two basic categories of the instruments that facilitate the quantitative evaluation of the earthquake phenomenon.

The seismographs record the displacement of the ground as a function of time and they operate on the continuous real time basis. Their recordings are of interest mainly to the seismology. The accelerographs record the acceleration of the ground as a function of time. They are adjusted to start operating whenever certain ground acceleration is exceeded, and used for recording of strong ground motions that are of interest to the structural engineers, where it is used for the design of structures.

3.1.2 Earthquake magnitude, intensity, seismicity, and seismic hazard

The magnitude of the earthquake is a measure of the ground motion in terms of the energy, which is released, in the form of seismic waves, at its point of origin. Its measure on Richter scale and it is defined zero magnitude earthquake, one which is recorded wit 1 μm amplitude at a distance of 100 km.

The local magnitude $M_l = \log(A) - \log(A')$, (3.1)

Where A` is the amplitude of the zero magnitude earthquakes M_L=0. The magnitude M_L of the earthquake is related to the energy released from the epicentre through the relation:

$$\log(E) = 12.24 + 1.44 \cdot M_L \text{ [Erg.]} \tag{3.2}$$

The above relation shows that one unit increase in the magnitude of the earthquake; result in the increase of the energy release of about 28 times. It is generally accepted that earthquake with magnitude below 5 on the Richter scale are not destructive to engineering structures [1]. The biggest earthquake since 1900 was M = 8.7. Many record of known earthquake was registered from that time refer to Table 3-1.

Table 3-1: Known earthquake records

Place	Year	Magnitude on Richter scale
El Centro	1940	M = 7.1
Alaska	1964	M = 8.4
Friaul	1976	M = 6.7
Mexico	1985	M = 8.1
Armenian	1989	M = 6.9
North California	1988	M = 7.1

The term earthquake intensity is a measure of the consequences that this earthquake has on the people and the structures of a certain area. The damage is usually qualitatively estimated using empirical intensity scales. An earthquake has only one magnitude but different intensities from place to another. The soil conditions have a significant effect on the distribution of the structural damage. This effect estimated through the so-called micro-zonation studies. The most common macro seismic scales used are: Modified Mercalli MM scale Medvedev – Sponheuer - Karnik (MSK) scale [3] Points of equal intensity are connecting on a map resulting in isoseismic counters, which divide the effected area into sections of equal intensity. The intensity does not provide quantitative information about the parameters that are related to the ground motion. From the structural point of view the ideal way is to estimate the seismic hazard of an area using long-term records and the statistical processing of their basic elements. The earthquake is a stochastic phenomenon with random distribution of magnitude and intensity in time and in space. The seismicity increases with the magnitude and the frequency of occurrence of earthquake in an area. Gunterberg law defined it as:

$$\mathrm{Log}(N) = a - b \cdot M \qquad (3.3)$$

Where N is the frequency of the earthquake, M is the magnitude of the earthquake and a, b constants that are defined from the statically processing of seismic records. [1]
Seismic hazard in an area is expressed either through the probability of occurrence of an earthquake with acceleration a_g or intensity I larger than a certain value in a certain period of time, or through the value of the acceleration a_g or intensity I for which the probability of exceeding that value in a certain period of time is less than a predefined limit. In Europe the following empirical attenuation law for a_g, has been proposed by Ambraseys and Bommer (1991).

$$\log(a_g) = -0.87 + 0.217 \cdot M_s - \log(r) - 0.00117 + 0.26 \cdot P \qquad (3.4)$$

Where: $r = \sqrt{\Delta^2 + h^2}$, and $\Delta$is the source distance and h is the focal depth.

The value of P is 0 for 50 percentile and 1 for 84 percentile.

The earthquake acceleration record in Mammoth Lakes, May 25 is shown in Figure 3-1. The earthquake is of special interest for the structural engineer working in seismic areas. It is hazardous phenomenon primarily when it is considered in relation with structures. Richter scale is a measure for the magnitude of earthquake. The destructiveness of an earthquake is a function of its magnitude, the focal depth, the distance from the epicentre, the soil conditions and the mechanical properties of the structures. The intensity of the earthquake is a measure of the consequences that the earthquake has on the people and the structures of a certain area. The earthquake is a stochastic phenomenon and consequently long-term records are needed. The limited reliability of the seismic hazard leads to base the safety of the structures in seismic area on specially designed extra reserves of strength and energy and dissipation mechanisms at low additional cost, which constitute the basic concept for the design of earthquake – resistant structures [1].

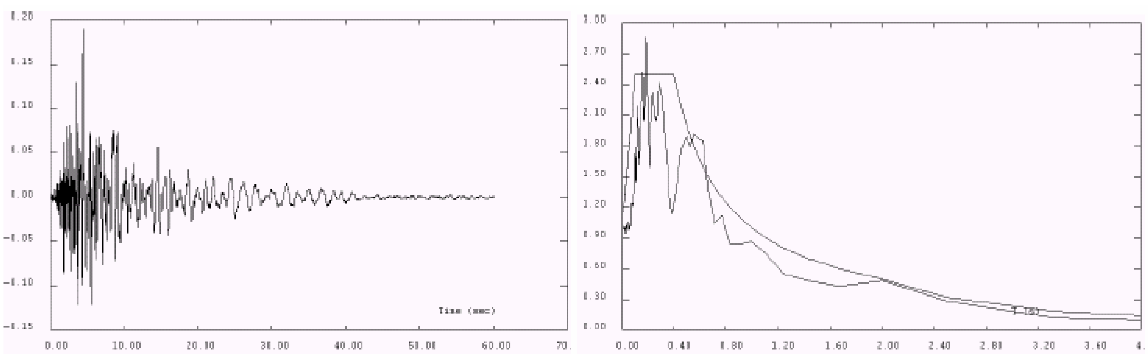


Figure 3-1: Accelerograms a (t) /g and elastic response spectrum S(T) /a max : Mammoth Lakes, May 25 1980, Gilroy.

3.2 Elements of Structural dynamics

3.2.1 Dynamics analyses methods

A structure that can vibrate only in one particular mode can be represented as a single-degree-of freedom (SDOF) system. When the geometric shape of the deformed shape system is known or prescribed then the time dependant amplitude is the only unknown, which is needed to define the position of each point in the structure in space and time. That can be idealized by the schematic SDOF system (Figure 3-2). The system consist of a mass m, supported by a spring of a constant k and subject to damping characterized by a dashpot with damping coefficient c. Isolating the mass as free body. The applied forces must be in equilibrium with the inertial force, where:

$$\ddot{u}_t = \ddot{u} + \ddot{u}_g \tag{3.5}$$

The velocity – proportional damping force is $f_c = c\acute{u}$. The spring restoring force and the inertial forces are given as follows:

$$f_i = mu'' \text{ , } f_c = ku \tag{3.6}$$

The possibility of the some with the external forcing function p(t) is formulated as follows:

$$m\ddot{u}_t + c\acute{u} + k\upsilon = p(t) \tag{3.7}$$

Ignoring the external forcing function; p(t) = 0 , the equation of motion is obtained as follows:

$$\ddot{u}_t + 2 \cdot \xi\omega\acute{u} + \omega^2\upsilon = -\ddot{u}_g \tag{3.8}$$

The natural frequency of the system is obtained as follows:

$$\omega = \sqrt{\frac{k}{m}} \tag{3.9}$$

And the damping ratio expressed as fraction of the critical damping is formulated as follows:

$$\xi = \frac{c}{2\omega m} = \frac{c}{c_{crit}} \tag{3.10}$$

[28] .The equation of motion shows that the most significant factor related to the seismic excitation for the description of the oscillation is the time-dependent base acceleration, which is the accelograph record. If during oscillation the exciting force become zero ($\ddot{u}_t(0) = 0$) the system continues to vibrate freely. In this case and for zero damping ($c = 0$), the following relations are applicable:

$$u = u_0 \sin(\frac{2\pi}{T_0} t) \tag{3.11}$$

$$\text{Where } T_0 = 2\pi\sqrt{\frac{M}{\kappa}} = \frac{2\pi}{\omega} \tag{3.12}$$

The natural period T_0 is the dynamic constant of the system, in which the characteristics of the system, that is the mass M and the spring constant k, has been incorporated, and ω is the natural circular frequency [rad /sec]. When damping is different from zero (c≠0), the following relations are applicable [1]:

$$u = u_0 \cdot \exp[-(c/2M)t] \cdot (A \cdot \sin(\gamma t) + B \cdot \cos(\gamma t)) \tag{3.13}$$

$$\text{Where } \gamma = \sqrt{\frac{k}{M} - (\frac{c}{2M})^2} \tag{3.14}$$

For $\gamma=0$ (with $c = c_{cr} = 2 \cdot \sqrt{Mk}$) the above equation becomes as follows:

$$u = u_0 \cdot \exp^{-\xi\omega t} \cdot (A \cdot \sin(\omega_D t) + B \cdot \cos(\omega_D t)) \tag{3.15}$$

$$\text{Where: } \omega_D = \omega \cdot \sqrt{1-\xi^2} = \gamma \quad \text{and} \quad T_D = \frac{T_0}{(1-\xi^2)^{\frac{1}{2}}} \tag{3.16}$$

In the excitation case the particular integral of equation (3.8) should be added in equation (3.15). In the transient seismic excitation case the above process of summation of one general and one particular integral of the differential equation is not possible. The numerical methods applied by the computer are required. The integration process describes the time history of the oscillation phenomenon for the variables:

(u = u(t), ú = (ú(t)), ü = ü(t))

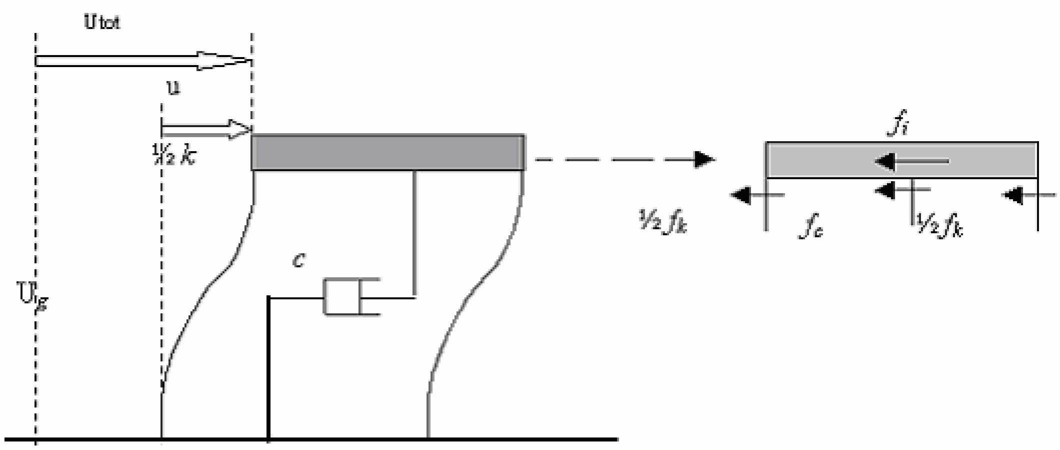


Figure 3-2: Schematic representation of elastic single degree of freedom construction

3.2.2 Earthquake response spectra:

Response spectra are important design tools in the seismic analysis and design of the structures due to the influence of the earthquake. The response spectra present the maximum response of the structure to a given ground motion .The response spectrum introduced by Biot and Housner describes the maximum response of damped SDOF system oscillated at different frequencies or periods. Newmark and hall have proposed a method for constructing an elastic response [29] in which the primary input datum is the anticipated maximum ground acceleration. The corresponding values for the maximum ground velocity and the maximum ground displacement are relatively proportioned to the maximum ground acceleration normalized to 1g (Figure 3-3).

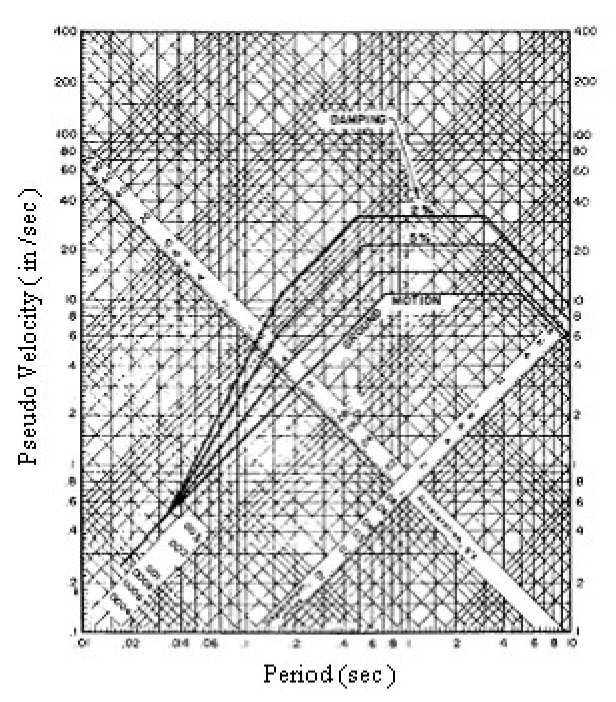


Figure 3-3: A New mark Hall design spectrum

3.3 The response of nonlinear SDOF systems

The response of a linear structural system can be evaluated using the Duhamel integral. The approach is limited to linear systems because the Duhamel -integral approach makes use of the superposition principle in developing the method. The evaluation of Duhamel integral for earthquake input motions requires the use of numerical methods. For these reasons it may be more expedient to use numerical integration procedures directly for the response evaluation of linear systems to subjected general dynamic loading. These methods have the additional advantage that with only a slight modification they can be used to evaluate the dynamic response of non-linear systems. Many structural systems will experience non-linear response sometime during their life. Any moderate to strong earthquake will drive a structure designed by conventional methods into the inelastic range, particularly in certain critical regions. A very useful numerical integration technique for problems of structural dynamics is the so-called step-by-step integration procedure. In this procedure the time history under consideration is divided into a number of small time increments Δ t. During a small time step, the behaviour of the structure is assumed to be linear. As non-linear behaviour occurs, the incremental stiffness is modified. In this manner, the response of the non-linear system is approximated by a series of linear systems having a changing stiffness. The velocity and displacement computed at the end of one time interval become the initial conditions for the next time interval, and hence the process may be continued step by step.

3.3.1 The equation of motion in numerical form

For the consideration of the single degree of freedom system (SDOF), the construction properties as the mass (m), the viscosity (c), the stiffness (k (t)) and the applied force (p(t)), where the applied force and the stiffness are functions of time are required. The stiffness is actually a function of the yield condition of the restoring force, and this in turn is a function of time. The damping coefficient may also be considered to be a function of time; however, it is convenient to determine the damping characteristics for the elastic system and to keep these constant throughout the complete time history. In the inelastic range the principle mechanism for energy dissipation is through inelastic deformation, and this is considerate through the hysteretic behaviour of the restoring force. The numerical equation required to evaluate the non-linear response can be developed, considering the equation of dynamic equilibrium given previously by Equation (3.7), in terms of the forces, can be written as:

$$f_i(t+\Delta t) + f_d(t+\Delta t) + f_s(t+\Delta t) = p(t+\Delta t) \tag{3.17}$$

Where the forces are defined as follows:

$$\begin{aligned} f_i &= m\ddot{u}(t+\Delta t) \\ f_d &= cu'(t+\Delta t) \\ f_s &= \sum_{i=1}^{n} K_i(t)\cdot\Delta u_i'(t) = \Gamma_i + K(t)\cdot\Delta u'(t) \end{aligned} \tag{3.18}$$

And the deformation is defined in as follows:

$$\begin{aligned} \Delta u'(t) &= u'(t+\Delta t) - u'(t) \\ \Gamma_i &= \sum_{i=1}^{n} K_i(t)\Delta u_i'(t) \end{aligned} \tag{3.19}$$

The equations of equilibrium in numeric terms defined as follows:

$$\begin{aligned} P(t+\Delta t) &= P_e(t+\Delta t) = -mg(t+\Delta t) \\ m\ddot{u}(t+\Delta t) + cu'(t+\Delta t) + \sum K_i\Delta u_i' &= -mg(t+\Delta t) \end{aligned} \tag{3.20}$$

For the ground accelerations representation the equilibrium equation is defined as follow:

$$P(t+\Delta t) = P_e(t+\Delta t) = -mg(t+\Delta t) \quad (3.21)$$

That leads to the system equilibrium equation as follows:

$$m\ddot{u}(t+\Delta t) + cu'(t+\Delta t) + \sum K_i \Delta u_i' = -mg(t+\Delta t) \quad (3.22)$$

It should be noted that the incremental stiffness is generally defined by the tangential stiffness at the beginning of the time interval. And it can be expressed as follows:

$$K_i = \frac{df_i}{du'} \quad (3.23)$$

3.4 Nonlinear response of MDOF systems

The non-linear analysis of buildings modelled as multiple degree of freedom systems (MDOF) closely parallels the development for single degree of freedom systems presented earlier. The non-linear dynamic time history analysis of MDOF systems is currently considered to be too complex for general use. Therefore, recent developments in the seismic evaluation of buildings have suggested a performance-based procedure, which requires the determination of the demand and capacity.
Demand is represented by the earthquake ground motion and its effect on a particular structural system. Capacity is the structure's ability to resist the seismic demand.
In order to estimate the structure's capacity beyond the elastic limit, a static non-linear (pushover) analysis is recommended. For more demanding investigations of building response, non-linear dynamic analyses can be conducted. For dynamic analysis the loading time history is divided into a number of small time increments, whereas, in the static analysis, the lateral force is divided into a number of small force increments. During a small time or force increment, the behaviour of the structure is assumed to be linear elastic. As non-linear behaviour occurs, the incremental stiffness is modified for the next time-load increment. Hence, the response of the non-linear system is approximated by the response of a sequential series of linear systems having varying stiffness.

3.4.1 Static nonlinear analysis

Nonlinear static analyses are a subset of nonlinear dynamic analyses and can use the same solution procedure without the time related inertia forces and damping forces. The equations of equilibrium are written in matrix form for a small load increment during which the behaviour is assumed to be linear elastic.

$$[K]\{\Delta u'\} = \{\Delta P\} \quad (3.24)$$

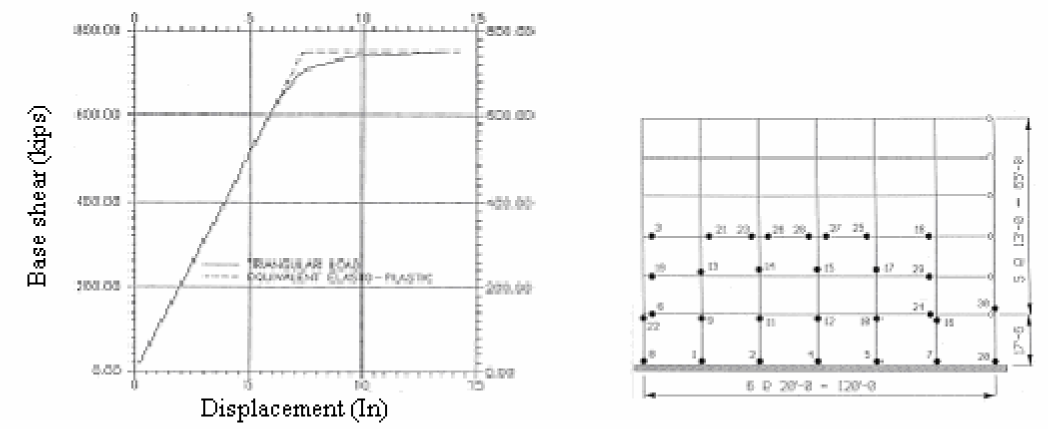
For computational purposes it is convenient to rewrite this equation as follows:

$$[K_t]\{\Delta u'\} + \{R_t\} = \{P\} \quad (3.25)$$

Where K_t is the tangent stiffness matrix for the current load increment and R_t is the restoring force at the beginning of the load increment, it is defined as follows:

$$\{R_t\} = \sum_{i=1}^{n-1} [K_{ii}]\{\Delta u'\} \qquad (3.26)$$

The lateral force distribution is generally based on the static equivalent lateral forces specified in building codes, which tend to approximate the first mode of vibration. These forces are increased in a proportional manner by a specified load factor. The lateral loading is increased until either the structure becomes unstable or a specified limit condition is attained. The results from this type of analysis are usually presented in the form of a graph plotting base shear versus roof displacement. The pushover curve for a six-story steel building, and the Sequence of plastic hinging formed in the building is shown in Figure 3-4.



Pushover curve

Sequence of plastique hinge formation

Figure 3-4: Pushover curve and sequence plastic hinge formation, in six story steel building

The equations of equilibrium for a multiple degree of freedom system subjected to base excitation can be written in matrix form as:

$$[M]\{\ddot{u}\} + [C]\{u'\} + [K]\{u\} = -[M]\{\Gamma\}\,g(t) \qquad (3.27)$$

This equation is of the same form as that for the single degree of freedom system. Vectors containing the additional degrees of freedom have replaced the acceleration, velocity and displacement. The mass matrix has replaced the mass, which for a lumped mass system is a diagonal matrix with the translation mass and rotational mass terms on the main diagonal. The incremental stiffness matrix has replaced the incremental stiffness and the damping matrix has replaced the damping.

3.4.2 Dynamic response of structures

A useful way to define the damping matrix for a non-linear system is to assume that it can be represented as a linear combination of the mass and stiffness matrices of the initial elastic system

$$\left[C\right] = \alpha \cdot \left[M\right] + \beta \cdot \left[K\right] \tag{3.28}$$

Where α and β are scalar multipliers that may be selected so as to provide a given percentage of critical damping in any two modes of vibration of the initial elastic system.

These two multipliers can be evaluated as follows:

$$\begin{Bmatrix} \alpha \\ \beta \end{Bmatrix} = 2 \cdot \begin{bmatrix} \omega_j & -\omega_i \\ -\dfrac{1}{\omega_j} & \dfrac{1}{\omega_i} \end{bmatrix} \frac{\omega_i \omega_j}{\omega_j^2 - \omega_i^2} \begin{Bmatrix} \lambda_i \\ \lambda_j \end{Bmatrix} \tag{3.29}$$

Where ω_i and ω_j are the percent of critical damping in the two specified modes. Once the coefficients α and β are determined, the damping in the other elastic modes is obtained from the expression:

$$\lambda_k = \frac{\alpha}{2\omega_i} + \frac{\beta \cdot \omega_k}{2} \tag{3.30}$$

4 Aspects in the design of reinforce concrete seismic frames.

4.1 Design of reinforced concrete beam column joints.

The detailing of beam-column joints of moment resisting frames was given attention since 1960. In spite of the improvement of the adjacent beams and column, the lack of the attention to the shear strength of beam column joints shows severe cracking in these joints, which observed in many laboratory tests and in many structures during earthquakes [10]. The consequence of the diagonal tension cracking is a significant increase in flexibility of the frame, and joint shear failure that has often led to collapse of the structure due to P-delta effects or loss of gravity load carrying capacity through the joint when columns have carried heavy axial compressive loads. Seismic design codes defers in the design approaches to beam-column joints, and this remains the most controversial aspect of the seismic design of reinforced concrete moment resisting frames at present [18]. USA/New Zealand/ Japan/ China before about 20 years established a project involving a comparison of code approaches for beam-column joints and comparison of the results of simulated seismic load tests on reinforced concrete beam-column joint specimens designed according to those codes [10]. The detailing of beam-column joints of moment resisting frames is quite important in the seismic resistance frames design. Severe cracking in the joints is observed in many laboratories. The diagonal tension cracking increase the frame flexibility, and joint shear failure often led to collapse of the structure due to P-delta effects or loss of gravity load carrying capacity. The maximum design horizontal and vertical shear forces acting on the joint core can be calculated from the forces when the flexural over strength in the adjacent plastic hinges are developed. The nominal horizontal joint shear stress is calculated from the horizontal joint shear force divided by the effective horizontal area of the joint, which is normally the area of the column. The horizontal shear reinforcement is designed often in the form of rectangular hoops and cross ties or spirals, and the vertical shear reinforcement is already present in the form of intermediate longitudinal column bars in the side faces between the corner bars. In Europe the transverse reinforcement is provided in the joint to limit the maximum diagonal tensile stress of the concrete to the tensile strength of the concrete. Vertical reinforcement is also required to be present in the joint. The plastic hinges in the beams normally occur near the beam-ends; the top and the bottom beam bars may yield in tension and compression alternatively at the column faces. The high bond stresses on the longitudinal bars in the joint core may lead to significant bond deterioration and some bar slip through the joint core. Design standards generally attempt to reduce bar slip by limiting the bond stresses by specifying maximum values for the ‘‘beam depth /column depth’’. Note that if the bond deterioration is significant the bar tension will penetrate through the joint and the bar tensile force will be anchored in the beam on the other side of the joint, which means that the “compression” steel there will actually be in tension. Solutions for such problem has been applied in this study by relocate the yield location to a sufficient distance from the column face. For most buildings, economy is achieved by allowing yielding to take place in some critically stressed elements under moderate-to strong earthquakes, for force levels significantly lower would be required to ensure a linearly elastic response. Analysis and experience have shown that structures having adequate structural redundancy can be designed safely to withstand strong ground motions allowing inelastic deformations to take place under strong earthquakes with an additional requirement to insure that yielding elements be capable of sustaining adequate inelastic deformations without significant loss of strength, i.e., they must possess sufficient ductility. The ductility of a section subjected to flexure or combined flexure and axial load is the ratio of the ultimate curvature attainable without significant loss of strength. The ductility of the beams and columns can be increased considerably using confinement reinforcement. In the past years full-scale test on R/C members and building was undertaken. Full-scale test on a precast building is included in this chapter; the objective is the developing seismic design recommendations for precast concrete system, the high performance of the connections uses ductile members for the seismic requirements is illustrated.

4.1.1 Nominal horizontal joint shear forces

The maximum design horizontal and vertical shear forces acting on the joint core can be calculated from the forces when the flexural over strength of the adjacent plastic hinges are developed. The nominal horizontal joint shear stress is calculated from the horizontal joint shear force divided by the effective horizontal area of the joint, which is normally the area of the column.

The forces from beams and columns during earthquake acting on an interior beam-column joint and the resulting crack pattern and bond forces after diagonal tension cracking initiates in the joint core is shown in Figure 4-1

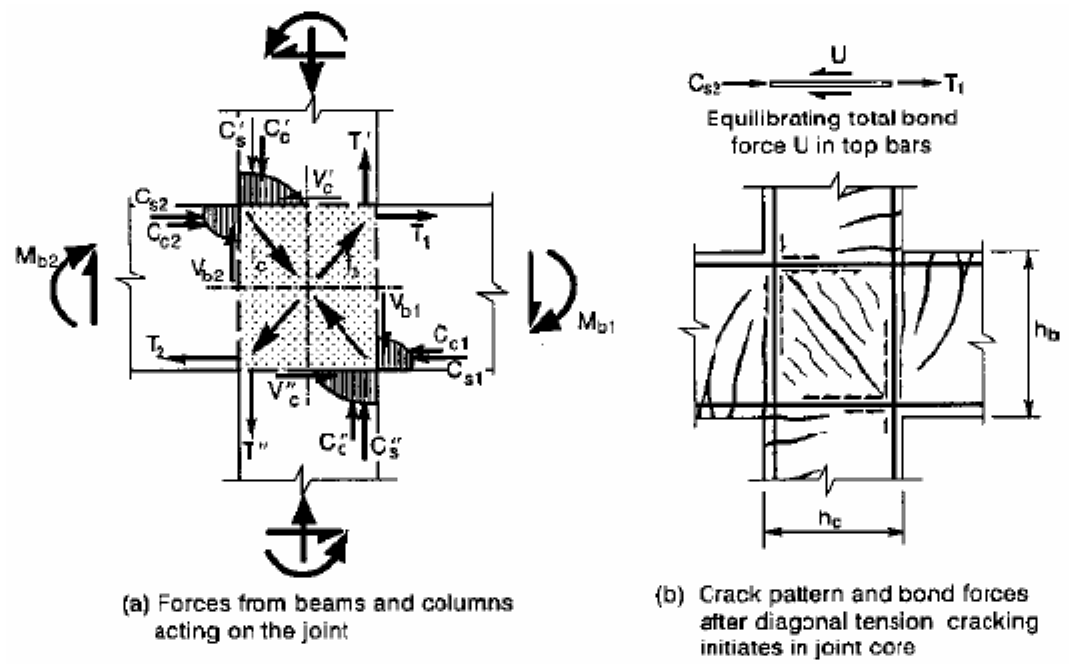


Figure 4-1: Force acting on a reinforced concrete interior beam-column joint during an earthquake.

4.1.2 Design approaches for shear

(a) The New Zealand approach

In the New Zealand concrete design standard [4] the joint shear forces are considered to be resisted by the contributions of the two mechanisms (Figure 4-2): The first mechanism in (a) consists of a single concrete diagonal compression strut which transfers both horizontal and vertical shear forces applied to the ends of the strut mainly by the compression forces in the concrete from the beams and columns. Also some of the bond forces from the longitudinal bars passing through the joint are transferred to the ends of the strut. The second mechanism in (b) is a truss mechanism consisting of a concrete diagonal compression field and the horizontal and vertical reinforcement in tension needed for equilibrium after diagonal tension cracking. The truss mechanism transfers mainly the shear induced in the joint core by the bond forces in the interior of the joint from the longitudinal bars passing through the joint.

The New Zealand concrete design standard [13] gives equations for determining the amount of horizontal and vertical shear reinforcement required. Normally the horizontal shear reinforcement is in the form of rectangular hoops and cross ties or spirals, and the vertical shear reinforcement is already present in the form of intermediate longitudinal column bars in the side faces between the corner bars. The New Zealand standard [13] also requires that the quantity of horizontal joint reinforcement placed for shear should not be less than that required in the end regions of the column above and below the joint for column ductility. The nominal horizontal joint shear stress is not permitted to exceed $0.2 \cdot f_c'$ in order to prevent a diagonal compression failure [10].

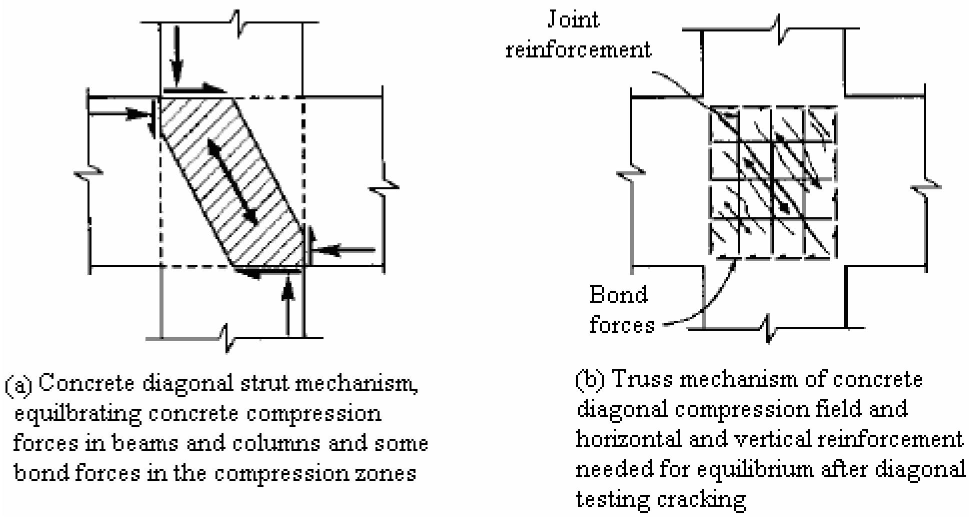


Figure 4-2: The resulting cracking and mechanism of force transfer assumed in New Zealand

(b) USA

The design approach of the building code of the American Concrete Institute [14] requirements is that the nominal horizontal joint shear stress does not exceed limiting values, which depend on the degree of confinement of the joint core by beams entering the sides of the joint. The limiting nominal horizontal joint shear stresses are $1.7\sqrt{f`_c}$ MPa for joints confined on all four faces, $1.25\sqrt{f`_c}$ MPa or joints confined on three faces or on two opposite faces, and $1.0\sqrt{f`_c}$ MPa for other cases. The amount of horizontal reinforcement in the joint core (area of transverse steel divided by the spacing) is required not to be less than that given by equations for confinement in the adjacent ends of the columns specified by the ACI building code [14]. These equations are based on the consideration that the axial load compressive strength of the core of the column (i.e., after spalling of the cover concrete) should at least equal the axial load compressive strength of the gross column section [10].

(c) Europe and Japan

In the design approach for joint shear in Europe [1] transverse reinforcement is provided in the joint to limit the maximum diagonal tensile stress of the concrete to the tensile strength of the concrete. Vertical reinforcement is also required to be present in the joint. On the other hand the Architectural Institute of Japan considers the joint shear strength not to be affected by the amount of transverse reinforcement in the joint [10].

4.1.3 Anchorage of longitudinal bars in interior beam-column joints

The plastic hinges in the beams normally occur near the beam-ends and hence during seismic loading the top and bottom beam bars may yield in tension and compression alternatively at the column faces. The high bond stresses on the longitudinal bars in the joint core may lead to significant bond deterioration and some bar slip through the joint core. As a result of this slip a beam bar can be close to yield in compression at one column face and at yield in tension at the other column face. Design standards generally attempt to reduce bar slip by limiting the bond stresses by specifying maximum values for the 'beam depth /column depth (d_b / h_c ratio) for the joint The ACI code [14] recommends a single constant maximum permitted value for 'db/hc' o 1/20 and hence places less emphasis is on limiting the bar slip. In New Zealand it is recognized that some bar slip is inevitable, but significant slip during a severe earthquake is considered to be undesirable for three main reasons: (1) It leads to a considerable reduction in stiffness of the frame which is residual, (2) bond deterioration is difficult to repair by epoxy resin injection, and (3) It leads to a reduction in the available curvature ductility factor of the adjacent plastic hinges in the beams. Note that if the bond deterioration is significant the bar tension will penetrate through the joint and the bar tensile force will be anchored in the beam on the other side of the joint. This means that the "compression" steel there will actually be in tension. The result is that, although the flexural strength of the beam may not be greatly reduced (except for large steel ratios), the available ultimate curvature at a specified ultimate concrete compressive strain is very greatly reduced [10].

4.2 Design for ductility

For most buildings, and particularly those consisting rigidly connected frame members and other multiply redundant structures, economy is achieved by allowing yielding to take place in some critically stressed elements under moderate-to strong earthquakes. This means designing a building for force levels significantly lower than would be required to ensure a linearly elastic response. Analysis and experience have shown that structures having adequate structural redundancy can be designed safely to withstand strong ground motions even if yielding is allowed to take place in some elements. As a consequence of allowing inelastic deformations to take place under strong earthquakes in structures designed to such reduced force levels, an additional requirement has resulted and this is the need to insure that yielding elements be capable of sustaining adequate inelastic deformations without significant loss of strength, i.e., they must possess sufficient ductility. The ductility of a section subjected to flexure or combined flexure and axial load is the ratio of the ultimate curvature attainable without significant loss of strength, ϕ_υ to the curvature corresponding to first yield of the tension reinforcement, ϕ_y. Thus sectional ductility is formulated as follows:

$$\mu = \frac{\phi_u}{\phi_y} \qquad (4.1)$$

4.2.1 The ductility in building

- The demand ductility in the building varies, according to the height of the building and the number of storey. (Figure 4-3) illustrates relations between the horizontal displacement, the ductility and the building level.

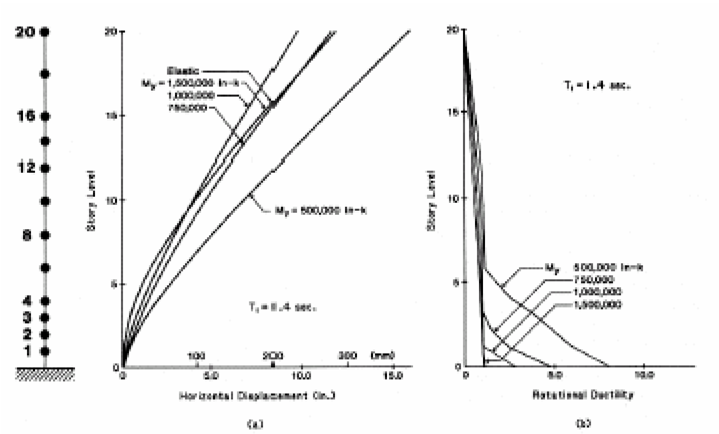


Figure 4-3: Effect on yield level on ductility demand. Note approximately equal maximum displacements for structures with reasonable yield level

The typical relation ductility-height of the building may be obtained as follows:

- The variation of the ductility demand is roughly uniform over the height of two and five storey building, for taller building, the ductility demand are larger in the upper and lower stories and decreases in the middle stories.

- The deviation of story ductility demand from the allowable ductility increases for the taller building.
- The relative story yields strength, which were chosen in conformance with the height wise distribution of the earthquake forces specified in the codes do not lead to equal ductility demand in all stories, but this is design objective.
- In most cases the ductility demand in the first storey is larger among all stories. The ductility variation according to the yield resistance of the building at each level.

4.2.2 Detailing for ductility

The strength and the ductility of the column is directly dependent on the confinement and bar slip details in the plastic hinges locations. Based on the experimental work done by Priestley, Seible, and Chai (1992), four models, as shown in Figure 4, were proposed to represent the strength and ductility of different plastic hinge conditions for column members. The parameter, μ_{ϕ}, is the displacement ductility, defined as the ratio of maximum displacement at the top of a column divided by the yield displacement.Figure 4-4.shows the flexural strength and ductility of sections (Priestley and Seible 1994) Line (1) represents columns that have a comparatively well-confined section, in which the nominal moment capacity, M_n, will be reached at μ_{Δ}=1 and then strain-hardening of flexural reinforcement and confinement effects will occur so that the moment reaches an overstrength moment capacity, M_0, that can be attained by moment-curvature analysis. Line (2) represents a poorly confined column without lap splices in the plastic hinge region, for which the maximum strength is typically equal to the nominal strength.

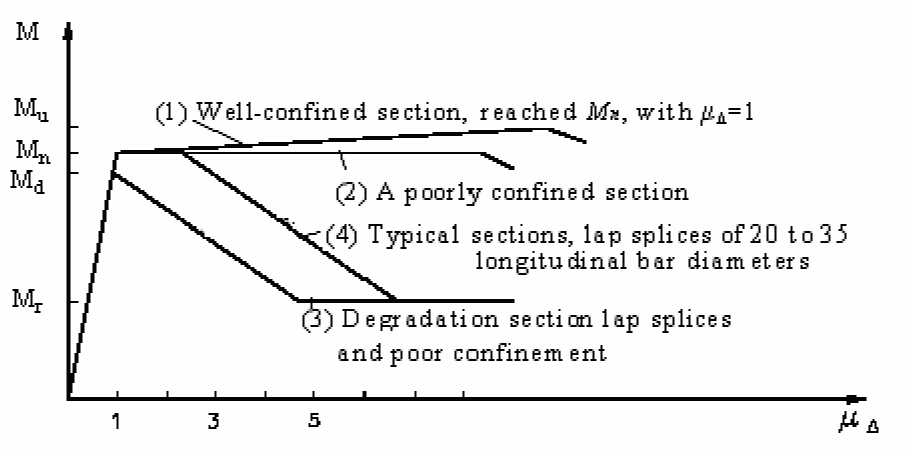


Figure 4-4: Flexural strength and ductility of sections (Priestley and Seible) 1994)

This model is suitable for the columns designed by pre-1971 design codes. Strength degradation will occur if the limit for line (1) or line (2) is reached due to crushing of core concrete and buckling of longitudinal reinforcement. Line (3) represents degradation of a column with lap splices and poor confinement in the plastic hinge region, in which the nominal moment capacity will not be achieved. The strength starts degrading before μ_{Δ}=1 strength, M_r, which is a function of the magnitude of the axial load. The cause of the degradation is that the longitudinal bar in the lap splice region cannot develop its yield force before slip and/or buckling occurs. Line (4) represents degradation of a column with partially confined lap splices, in which the nominal moment capacity M_n can be reached and then a relatively small plastic plateau can be achieved before degradation begins. The degradation occurs when the extreme fiber compression strain ε_c is 0.002. This model indicates that the longitudinal bar in the lap splice region can develop its yield force before slip and/or buckling. The line degrades to M_r, parallel to line (3).

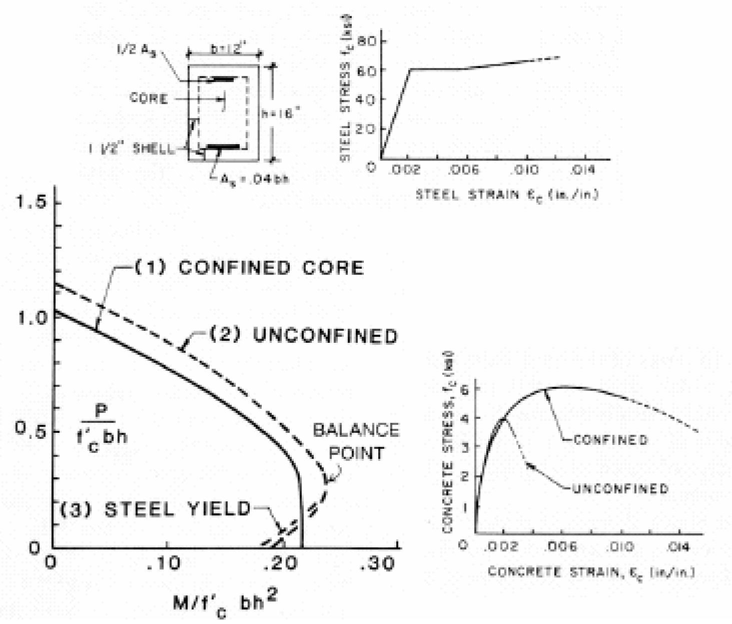


Figure 4-5: axial load moment interaction and load curvature for reinforced concrete section with unconfined and confined core.

An important design consideration assures that the plastic hinge regions of reinforced concrete members in moment resisting ductile frames is the provision of adequate longitudinal compression reinforcement as well as tension reinforcement, and the provision of transverse reinforcement in the form or rectangular hoops with or without cross ties, or circular hoops or spirals (
Figure 4-5 and Figure 4-6). The transverse reinforcement is needed to act as shear reinforcement, to confine and hence to enhance the ductility of the compressed concrete, and to prevent premature buckling of the compressed longitudinal reinforcement. According to the New Zealand concrete design standard [13] in order to confine the compressed concrete of columns in potential plastic hinge regions the center to - center spacing of transverse reinforcement along the member should not exceed one-quarter of the least lateral dimension of the cross section or 200 mm, whichever is greater. Also, the center-to center spacing of transverse reinforcement along the member in potential plastic hinge regions of beams and columns should not exceed six longitudinal bar diameters in order to control bar buckling [13]. The amount of transverse reinforcement necessary in columns to ensure adequate available ductility to match the imposed ductility can be calculated by moment-curvature analysis incorporating the stress-strain relations for confined concrete [20,18,21-22].

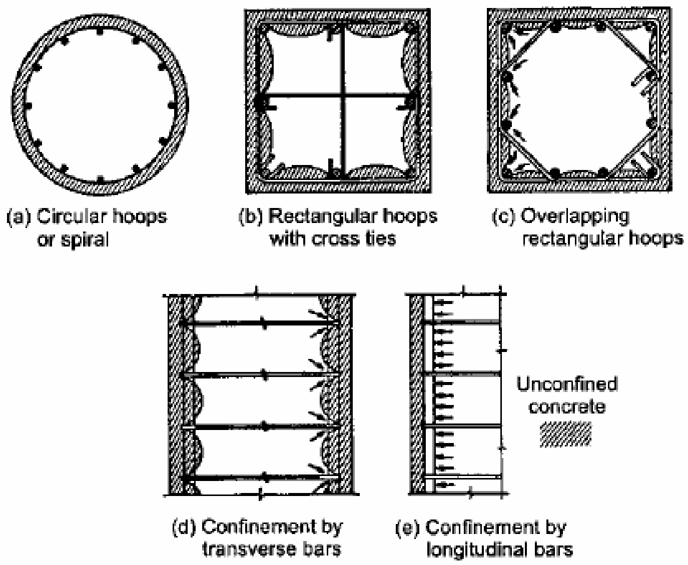


Figure 4-6: Reinforcement in column for shear resistance, concrete confinement and prevention of longitudinal bars buckling

The equation for the amount of transverse reinforcement required confining the compressed concrete of reinforced concrete columns in potential plastic hinge regions recommended by the New Zealand concrete design standard [13], was derived by Watson et al [23]. The amount is a function of the axial load level $N^*/f`A_g$, where N* is the axial compressive load on the column, $f`_c$ is the concrete compressive cylinder strength and Ag is the area of the column. According to the New Zealand equation the required amount of confining reinforcement increases with axial load level. Heavily loaded columns need more confining reinforcement because greater neutral axis depth c means that a greater extreme fibre concrete compressive strain $\varepsilon^{'}_c$ and hence a greater amount of confinement is needed to achieve a given ultimate curvature ($\phi_u = \varepsilon_c /C$) An example of the quantities of transverse reinforcement required by the New Zealand concrete design standard [13] for confinement of concrete in the potential plastic hinge regions of columns the curvature ductility factor $\phi u/\phi y$ of 20 is required (Figure 4-7), where ϕ_u is the ultimate curvature and ϕ_y is the curvature at first yield.

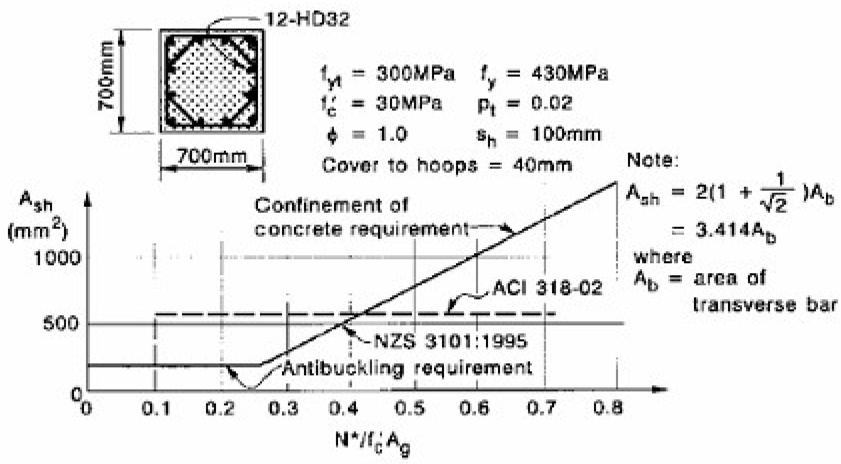


Figure 4-7: Example of transverse reinforcement required in ductile column.

Note that the requirement for concrete confinement in the New Zealand standard governs at higher axial loads and the requirement for preventing buckling of the longitudinal reinforcement governs at lower axial loads. Also in some cases transverse reinforcement required for shear resistance may govern. By comparison the ACI building code [14] amount, required for confinement, as it has been shown in Figure 4-7, is a single constant value regardless of the axial load level. The EC8 [24] requirements are also dependent on the axial load level and generally lead to greater quantities of transverse reinforcement than the New Zealand requirements [22]. The design with CEB for beams ductility requires taking into the account the span- depth relation, and the hoop spacing in the member, where the clear span should be greater than four times the member depth (for flexure response to prevail). Hoops should be provided for a distance of twice the member depth at each end, maximum hoop spacing not exceeding d/4 or 8 times the smallest bar diameter of longitudinal reinforcement.

4.3 Full -scale test on building

In the past years full-scale test on R/C members and building was undertaken .The objective is the developing seismic design recommendations for precast concrete system. Here is an overview of one of these tests on five storey-precast building, which simulated under seismic loading, according the press (Precast Seismic Structural Systems) program [25].

4.3.1 Building test

The five storey prototype building of (100×200 ft^2) per floor, the story height of 12 ft and 6 in, the bay length of 25 ft, the test building was modelled at 60% scale of the resized prototype building, to fit with the lab height.
In one direction of the building, seismic resistance system is provided by precast frame system, with a precast wall system and gravity frames in the orthogonal direction (Figure 4-10). It was used four different beam-to-column connections details, used at the different levels as it is shown in Figure 4-8 and Figure 4-9. They are: Hybrid frame connections (Figure 4-8a), Pretension frame connection (Figure 4-8 TCY gap connection (Figure 4-9), TCY frame connection (Figure 4-9).

Figure 4-8 a: Hybrid frame connection – Beam-to-column frame connection is established with unbounded post tensioning through the centre of joint and field placement of mild steel reinforcement in ducts across the joint interface closer to the top and bottom beam surfaces. These ducts are grouted to ensure adequate bond for the reinforcement prior to post-tensioning.

Figure 4-8 b: Pretension frame connection – Continuous partially bonded pretension beams are connected to column segments extending from the top of beam at one floor level to the bottom of beam at the level above. The moment connection between the beam and column is established by extending the column mild steel reinforcement below the beam through sleeves located in the joint. The extended reinforcement is spliced to the column longitudinal reinforcement at the next level adjacent to the joint.

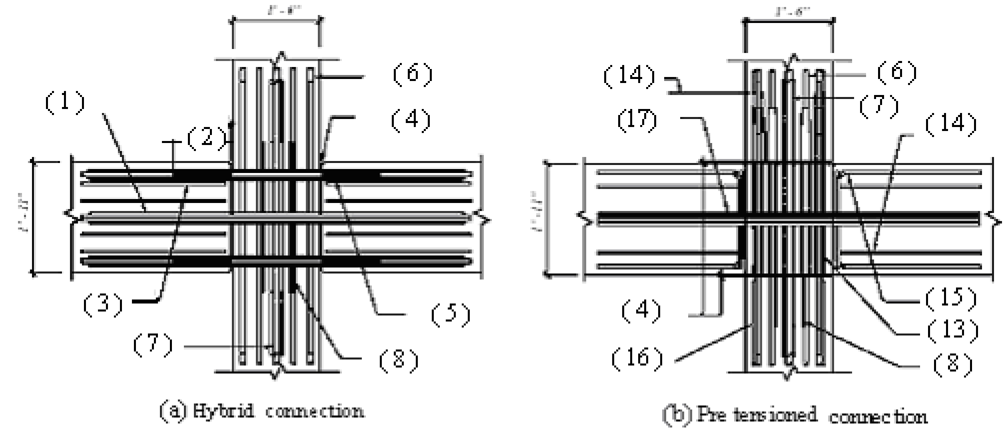


Figure 4-8: Hybrid and pretension frame connection

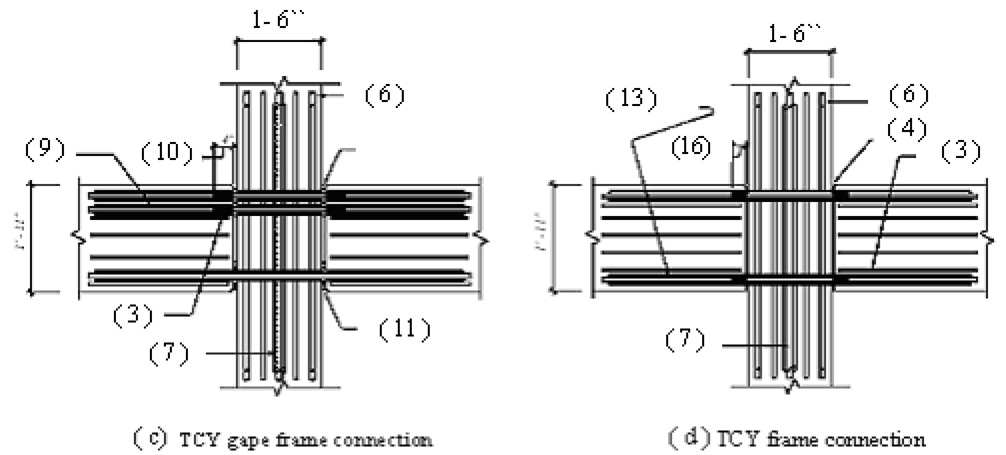


Figure 4-9: Tcy gape, and Tcy frame connections

Notation for Figure 4-8 and Figure 4-9 are as follows:

(1) 0.5” ϕ unbounded post-tensioned strands in PVC sleeve W/No grout
(2) Warp rebar (unbounded).
(3) Additional reinforcing not in sleeve per beam sections.
(4) ½” joint-fill w/grout prior to stressing.
(5) Mild reinforcing top and bottom in metal corrugated sleeves solid grouted
(6) Column longitudinal reinforcing
(7) Unbounded post-tensioning using Dywidag thread bars.
(8) 0.5”ϕ bonded prestressing strands.
(9) Mild reinforcing at top in metal corrugated sleeves solid grouted in beam and column.
(10) Warp rebar in sleeves.
(11) 1” joint-fill bottom6” of joint w/ fibre grout prior to stressing.
(12) Backer roos around PVC sleeves at joint to keep joint free of grout
(13) Mild reinforcing top and bottom in metal corrugated sleeves and grout.
(14) Terminal #5 filler bars at top of NMB splice sleeve.
(15) Main beam reinforcing top and bottom w/90^0 hooks at column face.
(16) # 5 filler bars terminated 1” clear from top of column.
(17) 0.5”ϕ prestressing strands (unbounded in beams).

Figure 4-9 c: TCY gap connection – Mild steel reinforcing bars placed in grouted sleeves at the top of the beam and unbounded post-tensioning at the bottom of the beam provide the necessary moment resistance at beam-ends. Beams and columns are separated by a small gap to avoid elongation of the beam due to seismic *1299 3* action. This gap is partially grouted at the interface over 6” at the bottom of the beam with the post tensioning force acting at the centre of grout.

Figure 4-9 d: TCY connection - Behaviour of monolithic reinforced concrete connections is emulated in this connection with top and bottom mild steel reinforcement in grouted sleeves across the beam-to-column interface.

Wall connection with special energy dissipating connectors located in a vertical construction joint between elements model is shown in Figure 4-10. The model includes two precast floor systems:

- The first three floors with pre-toped double tees.
- Hollow-core panels in the upper floors, with a cast in place topping.

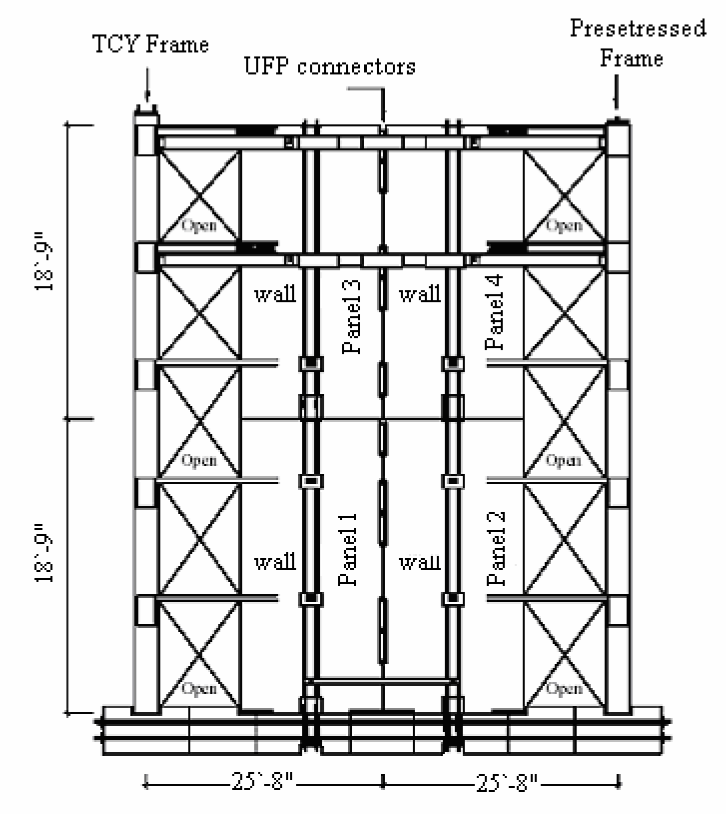


Figure 4-10: Jointed precast wall system

4.3.2 Design methodology

The PRESS building was designed using the direct displacement base design DBD [Priestley, 1998], to sustain maximum drift of 2% under a design level (EQ zone 4,soil type Sc acceleration spectrum UBC).

The reason for using (DBD) approach, for the designing was that force-based design does not sufficiently account for behaviour of jointed precast system. Furthermore, the R factors given in design codes as part of the force based design are also intended for precast systems emulating monolithic concrete connections, rather than for some of the connections incorporated in the PRESSS building.

4.3.3 Seismic test plan

Seismic testing are in two directions, with three different test schemes:

a) The stiffens measurement test

The stiffness measure test is a quasi-static loading test through which the stiffness matrix of the test building is formulated. That will be useful in

- Determining the appropriate integration time steps when explicit schemes are used in solving the equation of motion in the pseudo dynamic test procedure.
- Improving convergence of implicit integration schemes
- Characterizing structural behaviour consistent with the direct-displacement based approach
- Monitoring damage levels using stiffness as a damage indicator.

b) Pseudo dynamic test

Pseudo dynamic test in two directions was applied. The external dynamic load is applied–quasi-statically through ten on line controlled hydraulics actors. The numerical computation process, the experimental measurements, and the test concept is shown in Figure 4-11.

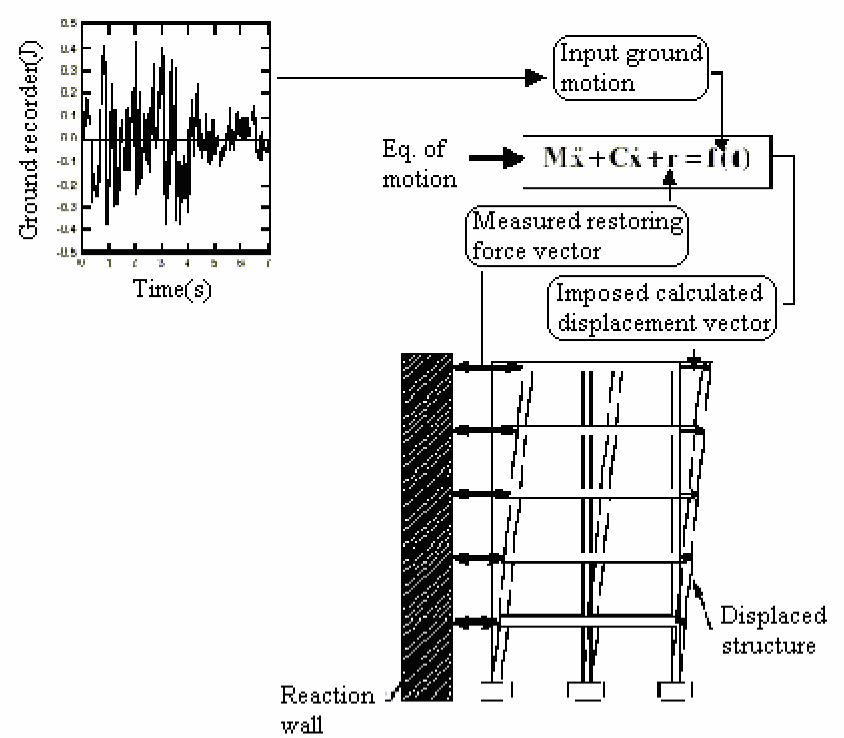


Figure 4-11: Pseudo dynamic test concepts.

Using the triangular load test, the building is subjected to full load reversal using a set of lateral forces distributed in an inverse triangular fashions, causing the structure to deform approximately to its first mode shape. The advantages of this procedure are the following:

- Response of the building from inverse triangular load test can be directly compared to the response assumed in the design procedure.
- The equivalent viscous damping of the building can be quantified.

4.3.4 Selection of the earthquake motion

The firs step in seismic testing is to formulate the stiffness matrix in the un-cracked state through a stiffness measure test, and then it follows by pseudo dynamic test, inverse load test, and stiffness measurement test. The sequence is repeated several times with intensity of input motion for the pseudo dynamic test increasing from EQ I to EQ IV refer to Table 4-1. These four levels represent, respectively, frequent, occasional, rare, and maximum credible earthquake.

The inverse triangular load test is performed for one cycle with full reversal at each level of input motion, such that the resulting maximum positive and negative roof drifts equal the maximum recorded drift in the preceding pseudo dynamic test. The first test is the Parallel to the jointed wall system and then in the orthogonal direction to examine the behaviour of seismic frames.

Table 4-1: Details of the original input record.

EQ Level	Record	Magnitude	PGA
EQ-I	Gilroy Array #1	ML=5.2	0.14 g
EQ-II	Hollywood Storage	Mw=6.6	0.21g
EQ-III	El Centro	Mw=6.9	0.35g
EQ-Iva	Sylmar	Mw=6.7	0.84g
EQ-Ivb	Tabas	Mw=7.4	0.94g

CONCLUSIONS

The following summarizes the response and conclusions available at the first stage.

(1) Damage to the building in the wall direction was minimal, despite being subjected to seismic intensities, 50 percent above the design level. Only minor spalling at the wall base, and fine cracking in floor slabs, and near the column bases were observed. The wall was essentially un-cracked except at the base during wall direction response. The wall showed additional flexural cracking when subjected to the 4.5 percent out-of-plane drift in the frame direction of response. However, these cracks closed up to be invisible to the naked eye at the end of testing.

(2) Damage to the building in the frame direction of response was much less than could be expected for an equivalent reinforced concrete structure, subjected to the same drift levels. The performance of the prestressed frame was particularly good, with damage being limited to minor spalling of cover concrete in the beams

immediately adjacent to the columns and some crushing of the fiber grout pads at the beam-column interfaces. Crack levels in the beam to- column joints due to shear were extremely small, and it was evident that the amount of joint shear reinforcement provided, which conformed to current code levels, could have been substantially reduced.

(3) Although the non-prestressed frame also performed well, the TCY gap connections showed more damage, occurring at an earlier stage, than the other connection types. This was due to the inadequate clamping force provided by the bottom post-tensioning, resulting in upward slip at the beam-column interface under seismic shear forces. Minor changes to the design criteria would correct this problem, but some attention will also be needed to improve detailing at the beam-ends to reduce the tendency for premature spalling of concrete cover.

(4) At high levels of response displacements, beam rotation about the longitudinal axis was noted, caused by the high torsion moment induced by the vertical load from the eccentrically supported double-tee floor members, and the reduced torsion resistance in the beam-end plastic hinges. Modified support details should be developed transferring the double-tee reaction closer to the support beam centerline.

(5) As anticipated, residual drift after the design level excitation was very low. In the wall direction, the residual drift was 0.06 percent after sustaining a peak drift of 1.8 percent. This corresponds to only 3 percent of the maximum drift. The low residual drift is a characteristic of the unbounded prestressing system used to provide strength in the wall direction, and is a significant advantage over conventional cast-in-place reinforced concrete construction, where very high residual drifts are possible. The low residual drift was also apparent in the prestressed frame, which was based on the same unbounded prestressed philosophy. The non-prestressed frame dissipated more energy, but suffered higher residual drifts and somewhat higher damage levels.

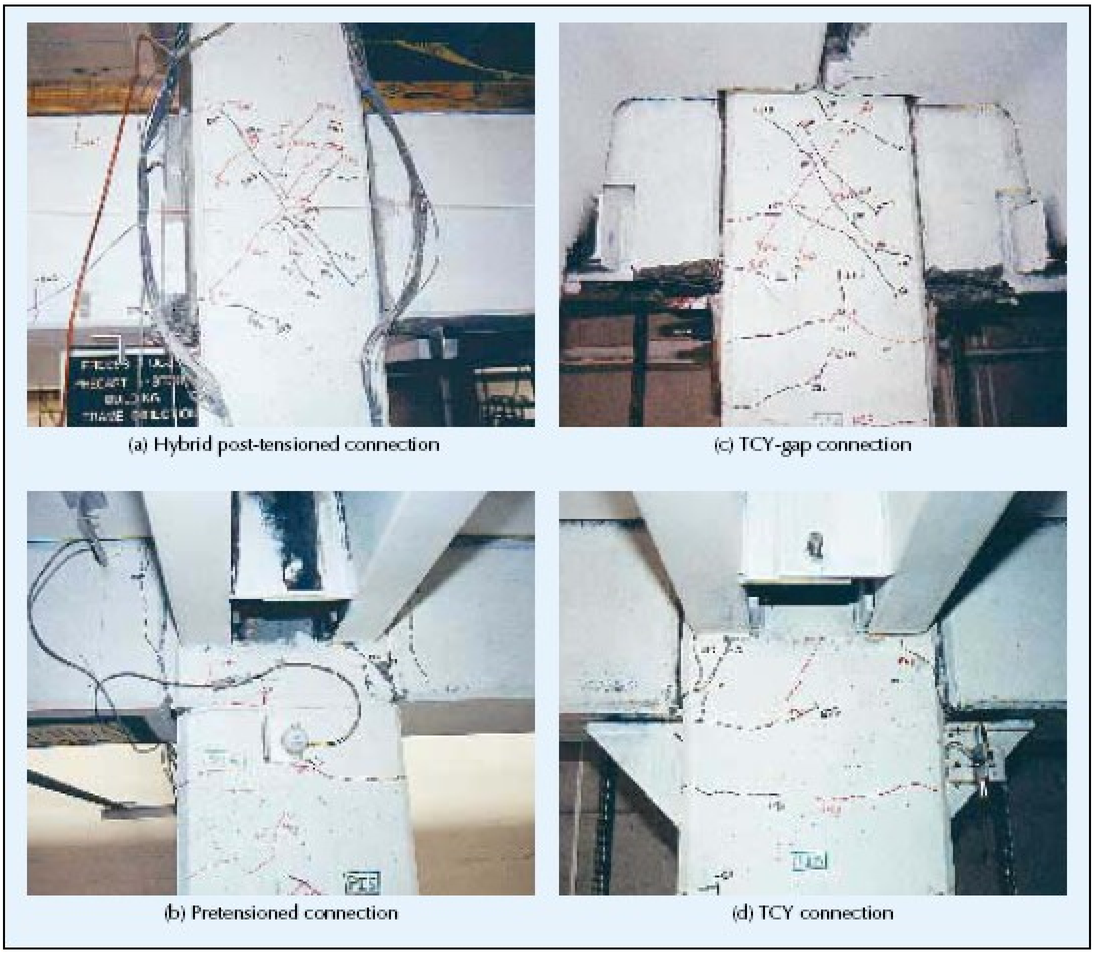


Figure 4-12: Condition of frame connection after being subjected to drift levels more than twice design level

(6) The test provided an excellent confirmation of the direct displacement-based design approach used to determine the required strength of the building. The structure was designed to achieve drifts of 2 percent under the design level earthquake (Zone 4, UBC intensity), and actually sustained average drifts of 1.8 and 2.2 percent in the wall and frame directions, respectively. This drift is within the accuracy of spectrum matching for the test accelerograms. The required base-shear strength using direct displacement based design was only 45 and 60 percent of the strength required by conventional force-based design using UBC provisions for wall and frame directions, reactions, respectively.2

(7) In the pseudo dynamic tests, much higher floor forces were experienced than anticipated. These high force levels were confirmed by analytical studies and were the result of higher mode effects. These high floor force levels represent diaphragm force levels that are significantly higher than currently considered in design and are a consequence of the comparative insensitivity of the higher mode force levels to ductility. Current designs, where force reduction factors are applied equally to higher modes as well as the fundamental mode result in a critical underestimation of the diaphragm force levels. The influence of higher mode floor forces is also translated into story shear force levels, and moment distributions that are more severe than currently considered in American designs using an inverted triangular distribution of the base shear force.

(8) Simple analytical models, suitable for the design office, were used to predict the response of the building. These were found to provide very good simulation of response, with excellent correlation between predicted and observed displacements, story shears, and overturning moments.

The hybrid prestressed connections is shown in Figure 4-12 a is performing extremely well at this stage, with only minor damage in the form of cover spalling, and some crushing and incipient break-down of the fibre grout pads between the beams and columns. No significant increase in width of the joint shear cracks above that sustained at the design level was observed. It is probable that the joint strain gauge results will show that substantial reductions in joint reinforcement will be possible with this detail. The performance of the pretension connections in the upper floors of the prestressed frame was also excellent (Figure 4-12 b). Damage in these connections was also limited to superficial cover spalling at, or adjacent to the beam-column interface.

Sliding of the TCY-gap connections had continued during testing levels above the design level. As a consequence of the high tension strains from the seismic response coupled with the dowel bending caused by the interface sliding, a few of the mild steel reinforcing bars crossing the interfaces fractured in the latter stages of testing. It is clear that sliding of the TCY-gap beams could have been avoided if the clamping force provided by the post-tensioned threaded bars at the base of the beams had been higher. It was also observed that early crushing of the cover concrete might be inevitable with this design detail because of the high compressive force across the grout pad when the top bars are in tension. Strengthening the beam-ends or some other detail to inhibit crushing may be advisable in future designs.

Figure 4-12 c shows the condition of a typical TCY-gap interior-column connection at the end of testing. Damage is still much less than would be expected from a conventional reinforced concrete beam-to-column joint at this level of drift. Design recommendations for improving this detail will follow in a subsequent paper. Damage to the TCY connections in the upper levels of the reinforced concrete frame was not significantly different from that at the design level. A small amount of interface sliding continued to occur at these higher levels of response, but crack patterns and spalling did not noticeably change. Some loss of bond between the top-level reinforcing bars in the grout ducts was noted at the exterior connections, with the headed reinforcing bar, which were exposed at the ends of the connection being pushed out by up to 1 in. (25 mm). There did not appear to be any significant strength reduction associated with this action.

The condition of the upper TCY connections was good, as illustrated in Figure 4-12 d, this despite the observation that incipient sliding was occurring at the interface. It should be emphasized that this sliding or shear deformation in the plastic hinge region has been commonly observed in tests of reinforced concrete beam-to-column connections at moderate ductility levels.

5 Capacity design

The capacity design method states that the ductile behaviour in a structure requires that the yield capacity reached first in ductile response modes rather than in brittle modes (chapter 5), and that can be achieved by suitable analysis to check the presence of the required strength in the members. The simple representation of the capacity linked a structure to a chain, in which each link represent a possible failure mechanism in an individual or group members with allowance for the designed material strength to be greater than the minimum specified strength, considering the over-strength factor. EC8 allows designing using three structural ductility classes in the frames; the high ductile class, the middle ductile class and the low ductile class. The higher the ductility leads to the lowering of the design force and then the required strength. The design spectrum is obtained by dividing the elastic spectrum by the behavior. Its approach aims at a well defined global mechanism, the so called beams sway mechanism, in which all beams at all storeys form plastic hinges, while all columns remain elastic for their entire height, with the exception of their base section at the ground storey. Weak beam strong column connection concept is applied and formulated for the seismic action performance.

In according to the New Zealand concrete code for assessing these allowance the chain of foundations, shear in columns, flexure in columns, flexure in beams, shear in beams, beam column joint, should be designed for the seismic action up to the ultimate limit state considering strength reduction factor of all elements. And for the capacity requirement the required strength for the other elements in the chain should be greater than magnification factor multiply by the effected force with a magnification factor as the following.

$$\mathrm{S}_i = \mathrm{F}$$
$$\mathrm{S}_i \geq \omega \cdot \phi_0 \cdot \mathrm{F} \tag{5.1}$$

Where ϕ is srength reduction factor, ϕ_0 is overstrength factor, and
ω is dynamic manification fact

The EC8 allows designing using three different balances between the strength and the ductility of the structure. The three levels, labeled as the high ductile class DCH, the middle ductile class DCM and the low ductile class DCL frame respectively. The higher the ductility leads to the lowering of the design force and then the required strength The design spectrum is obtained by dividing the elastic spectrum by the behavior q. In this study we consider the EC8 for the capacity design of the building.

The essential principles of the capacity design lead to the conclusion that, for this requirement a reasonable analyses procedure should be used for the structure that results in enough details of the loading on the member in the critical locations. In order to get controlled structure behaviour according to the analyses principles and results, the design of the member should include the tolerance in the material and the dynamic loading. The location that assumed to yield should be verified in strength that insures the continuity of the applied loadings and stiffness in a proportion sufficient for the distribution of the forces and deformation. In the reinforced concrete the flexure ultimate limit state of the cross section and the shear resistance is the effective measure for the strength factor, and the yield moment is the measure for the stiffness.

5.1 Capacity design of the frames

For the moment requirements, the action effects are those, which obtained from the analyses of the structure from the seismic force combinations. The ductile behaviour is insured by detailing rules, and especially concerning the maximum and minimum reinforcement ratio, the concrete confinement in the critical regions, the adopted length for such regions to be taken 2hw, 1.5hw, 1hw (hw; is the beam height) for DCH, DCM and the DCL frame respectively. For the shear requirements at least for DCH should not be that resulting from the analysis, but the shear corresponding to the equilibrium of the beam. That describes the actual bending of the cross section. EC8 specifies that every connection in the beam should be designed for a tension or compression normal force equal to the largest shear that the column should carry. The probability of the plastic hinge formation in the columns should be decreases. That should by achieve by satisfying the condition that at each beam-column joint the sum of the bending resistances of the columns end sections is larger than the corresponding one in the beams. The reason for these requirements is that, Columns have les available ductility than beams, avoiding columns failure is much more crucial for overall safety of the structure than the beam and the formation of the plastic hinges in the columns may leads to significant inter-story drift which leads to increase the second order effect and my cause the collapse of the structure. Therefore the equilibrium equations for the moments and shear includes inequalities with factors that accounts for the variability of the yield stress f_y and the probability of strain hardening effects in the reinforcement, the overstrength factor.

5.2 Capacity control according to EC8

The capacity design approach for EC8 aims at a well defined global mechanism, the so called beams sway mechanism, in which all beams at all storeys form plastic hinges, while all columns remain elastic for their entire height, with the exception of their base section at the ground storey. In this mechanism the global inelastic drift (plastic displacement at the top divided by the height of the frame) equals the plastic rotations at the end regions of al the beams, which corresponds to the most uniform possible spreading of inelasticity in the structure. The strength and the loading on the connection members are considered in two directions (Figure 5-1). In direction 1 the top of the right beam and the bottom of the left beam, are the resisting tension faces .The right side of the top column and the left side of the down columns side at the connections are the tension resistance faces. And the inverse will be applicable for the direction 2. The flexural strength of the beams in any section have the minimum resistance moment is never less than half the maximum with the opposite sign.

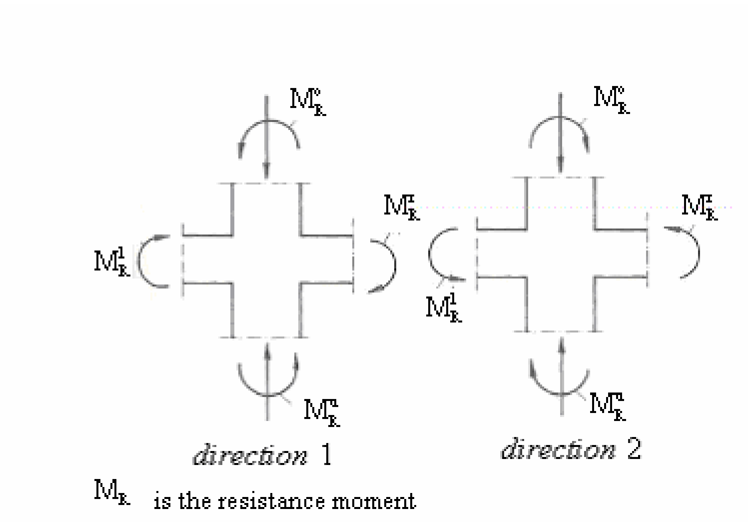


Figure 5-1: Interior connection details

The capacity design is satisfied if the columns are designed for the moments according to the equation (5.4), where:

$$\text{In direction 1} \quad \alpha_{cd1} = \gamma_{Rd} \cdot \frac{M_{R1\_L} + M_{R1\_R}}{M_{S1\_O} + M_{S1\_U}}$$
$$\text{In direction 2} \quad \alpha_{cd2} = \gamma_{Rd} \cdot \frac{M_{R2\_L} + M_{R2\_R}}{M_{S2\_O} + M_{S2\_U}} \tag{5.2}$$

Where M_S is the applied moment, which satisfies the seismic combinations, and M_R is the corresponding resistance moment as it is illustrated in Figure 5-1.

The relative importance of the gravitational with respect to seismic load is measured through the so-called moment reversal factor δ, which is given as follows:

$$\text{In direction 1} \quad \delta_1 = \frac{\left|M_{S1\_R} + M_{S1\_L}\right|}{\left|M_{R1\_R}\right| + \left|M_{R1\_L}\right|}$$
$$\text{In direction 2} \quad \delta_2 = \frac{\left|M_{S2\_R} + M_{S2\_L}\right|}{\left|M_{R2\_R}\right| + \left|M_{R2\_L}\right|} \tag{5.3}$$

The design bending moment in the columns to be calculated separately for the two sign of the seismic action, is given as follows:

$$\text{For direction 1 } M_{Sd1\_cd} = \min(\left|1 + (\alpha_{cd1} - 1) \cdot \delta_1\right| \cdot M_{Sd1},\ q \cdot M_{Sd1})$$
$$\text{For direction 2 } M_{Sd2\_cd} = \min(\left|1 + (\alpha_{cd2} - 1) \cdot \delta_2\right| \cdot M_{Sd2},\ q \cdot M_{Sd2}) \tag{5.4}$$

The strength of the beams in the top face and in the bottom face is considered for the reversal seismic actions i.e. the direction 2. The same described procedure used for the interior and the exterior column beam connections. For the exterior connections the values of the resistance moment of the beams to be consider for one side. It is convenient to notice that the column compatibility for the capacity design requirements needs more verifications and modifications for the beams reinforcements and the columns. There is a significant difference in the response of the exterior columns for the seismic actions in the two directions.

The capacity design shear forces at the columns assuming plastic hinges form at both ends, by means of the expression:

$$V = \gamma_{Rd} \frac{M_{Rc1} + M_{Rc2}}{h} \tag{5.5}$$

The capacity design for shear considering possible plastic hinge formation at the beam-ends is as follows:

$$\text{Shear strength} \geq \gamma \times \frac{M_{p1} + M_{p2}}{l_p} + \frac{W}{2} \text{ . With } W = (DL + 0.15 \cdot LL) \cdot L_{beam} \tag{5.6}$$

Where $M_{Rc1,2}$ are the flexural capacities of the end sections. γ_{Rd} =1.35 and 1.2 for DCH and DCM, respectively.

6 Nonlinear characteristics of R/C members

Dynamic response of a structure may be caused by different loading conditions such as: earthquake, ground motion, wind pressure; wave action; blast; machine vibration and traffic movement. The dynamic problems are different from static one in, the inertial force; damping; strain rate effect; and oscillation (stress reversals). Dynamic characteristics up to failure cannot be identified solely through a dynamic test or a real structure because it is difficult to understand the behaviour due to complex interactions of various parameters, it is expensive, and the capacity of loading devices insufficient to cause failure. Dynamic tests of real buildings are rather aimed toward obtaining data to: Confirm the validity of mathematical modelling techniques for a linearly elastic structure, and to obtain damping characteristics of different types of structures. The energy-dissipating mechanisms (Damping) in the structures are in the following forms: Inelastic hysteretic energy dissipation, radiation of kinetic energy through foundation, kinetic friction, viscosity in materials and aerodynamic effect. The speed of loading is known to influence the stiffness and strength of various materials, the strain rate during an oscillation is highest at low stress levels, and that the rate gradually decreases toward a peak strain. Cracking and yielding of a reinforced concrete member reduce the stiffness, elongating the period of oscillation. Furthermore, such damage is normally caused by the lower modes of vibration having long periods. Hysteretic model are able to provide the stiffness and resistance under any displacement history, which needs the definition of the basic characteristics of the member geometry and material properties in chapter 6 different hysteresis tests and rules are illustrated to define the flexural, shear and bond cyclic behaviour of reinforce concrete connections. Some of the available rules used in Ruaumoko program to solve the non-linear behaviour and the degrading of the building designed in this study.

Research on the subject of non-linear structural behaviour has been carried out in relation to earthquake problems. The dynamic problems are different from static one in the following factors: inertial force; damping; strain rate effect; and oscillation (stress reversals). These factors need to be clarified in order to analyse a structure under dynamic loading. Dynamic characteristics up to failure cannot be identified solely through a dynamic test or a real structure for the following reasons: It is difficult to understand the behaviour due to complex interactions of various parameters; It is expensive to build a structure, as a specimen, for destructive testing; and the capacity of loading devices insufficient to cause failure. Consequently, dynamic tests of real buildings are rather aimed toward obtaining data for the two main issues:

(a) To confirm the validity of mathematical modelling techniques for a linearly elastic structure.
(b) To obtain damping characteristics of different types of structures. A specifically designed laboratory test becomes inevitable in order to complement the weakness of full-scale tests and to study the effect of individual parameters.

6.1 Damping

Any mechanical system possesses some energy-dissipating mechanisms as follows:

(a) Inelastic hysteretic energy dissipation.
(b) Radiation of kinetic energy through foundation.
(c) Kinetic friction.
(d) Viscosity in materials.
(e) Aerodynamic effect.

Such capacity or energy dissipation is vaguely termed "damping," and is most often assumed to be of viscous type because of its mathematical simplicity. Damping capacity is often determined by the bandwidth of the response curve during a sinusoidal steady-state test. Figure 6-1 shows such acceleration response curves for a reinforced concrete building at different excitation levels（Jennings and Kuroiwa 1968). Note the shift of resonant frequencies and the change in amplitudes of damping with increase of excitation level despite low response amplitudes.

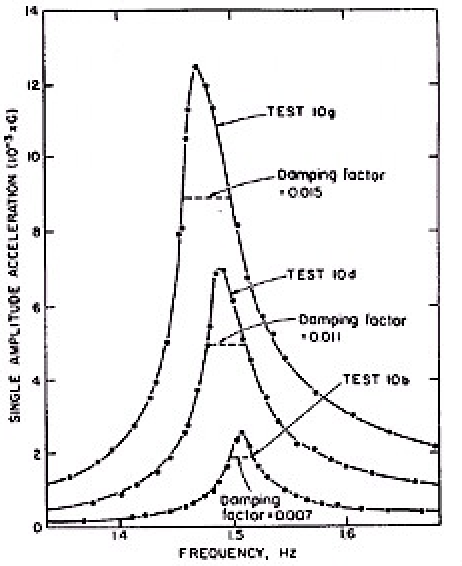


Figure 6-1: Observed acceleration amplitude from steady state test (Iennings and Kuroiwa)

6.2 Strain rate effect

It is technically difficult to test structural members under dynamic conditions in a laboratory. Before force-deformation relations already obtained from thousands of static tests can be studied for use in a dynamic analysis, the effect of strain rate on the force-deformation relation needs to be examined. The speed of loading is known to influence the stiffness and strength of various materials (Cowell 1965, 1966). Some member test results are available (Mahin and Bertero l972). Important findings from these investigations are as follows:

(a) High strain rates increased the initial yield resistance, but caused small differences in either stiffness or resistance in subsequent cycles at the same displacement amplitudes.
(b) Strain rate effect on resistance diminished with increased deformation in a strain-hardening range.
(c) No substantial changes were observed in ductility and overall energy absorption capacity.

Note that strain rate (velocity) during an oscillation is highest at low stress levels, and that the rate gradually decreases toward a peak strain. Cracking and yielding of a reinforced concrete member reduce the stiffness, elongating the period of oscillation. Furthermore, such damage is normally caused by the lower modes of vibration having long periods. Therefore, the strain rate is small in the case of earthquake response, and its effect on the response is small. Consequently, the static hysteretic behavior observed can be utilized in a nonlinear dynamic analysis of reinforced concrete structures.

6.3 Stiffness properties of reinforced concrete members

It is not feasible to analyse an entire structure using microscopic material models. It is more important to study the behaviour of isolated members and their subassemblies (beam-column, slab-column, and slab-wall connections) so that their analytical models can be developed for use in the analysis of a complete structure. A typical force-deflection curve of a cantilever column is shown in Figure 6-2. Otani et al. (1979) noted the following observations:

(a) Tensile cracking of concrete and yielding of longitudinal reinforcement reduced the stiffness.
(b) When a deflection reversal was repeated at the same newly attained maximum amplitude (for example, cycles 3 and 4) the loading stiffness in the second cycle was lower than that in the first cycle, although the resistances at the peak displacement were almost identical.
(c) Average stiffness (peak-to-peak) of a complete cycle decreased with a maximum displacement amplitude. For example, the peak -to-peak stiffness of cycle 5, after large amplitude displacement reversals, was significantly reduced from that of cycle 2 at comparable displacement amplitude. Therefore, the hysteretic behavior of the reinforced concrete is sensitive to loading history.

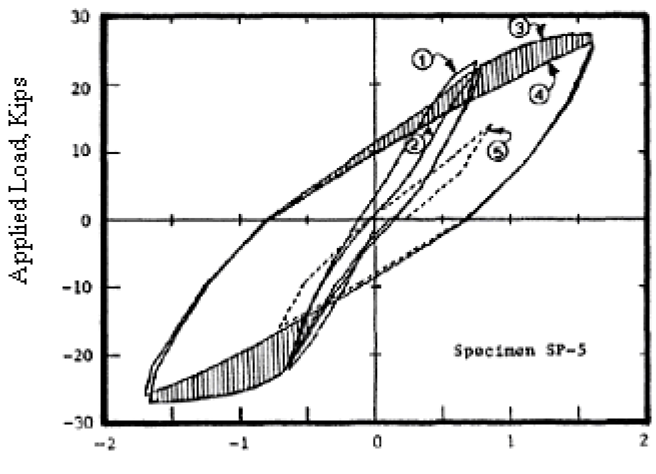


Figure 6-2: Hysteric characteristic of reinforced concrete member (Otani et al.1979)

6.4 Flexural characteristics

The flexural deformation index (average curvature) is obtained from longitudinal strain measurements at two levels assuming that a plane section remains plane. This flexural deformation index does not represent the flexural deformation in a strict sense because a plane section does not remain plane in a region where an extensive shear deformation occurs.

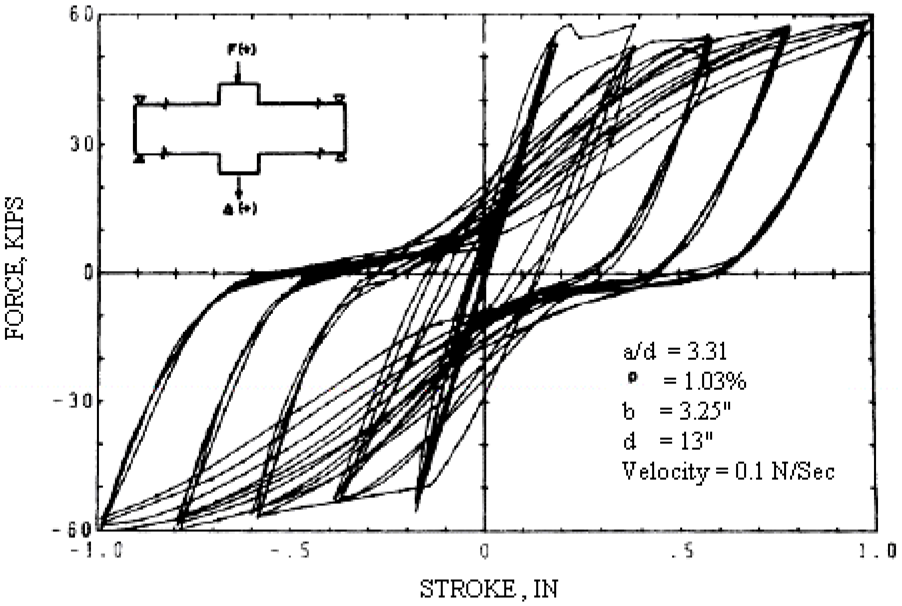


Figure 6-3: Flexural deformation characteristics (Celebi and Penzien 1973).

However, the index is useful for understanding flexural deformation characteristics qualitatively. A typical moment-flexural deformation index curve obtained from a simply supported beam test (Celebi and Penzien 1973) is shown in Figure 6-3. Note that the stiffness during loading gradually decreases with load, forming a fat hysteretic loop, and absorbing a large amount of hysteretic energy. The hysteretic loops remain almost identical even after several load reversals at the same displacement amplitude beyond yielding. Consequently, vibration energy can be efficiently dissipated through flexural hysteretic loops without a reduction in resistance**.**

The increase in axial force decreases the flexural ductility of a reinforced concrete member, but increases force levels corresponding to main two factors as follows:

(a) Tensile cracking of concrete.
(b) Tensile yielding of longitudinal reinforcement.

6.5 Shear characteristics

Similar to the flexural deformation index, a shear deformation index is defined from strain measurements in the two diagonal directions. Again, this index does not represent the true shear deformation because the interference of shear and flexure exists. A typical lateral load-shear deformation index curve (Celebi and Penzien 1973) is shown in Figure 6-4. Unlike what occurs in flexure, the stiffness during loading gradually increases with load, exhibiting a “pinching” in the curve. The hysteretic energy dissipation is smaller. The hysteretic loop decays with the number of load reversals, resulting in a smaller resistance at the same peak displacement in each repeated loading cycle. Although the curve shows a “yielding” phenomenon, it is important to recognize that the shear force of the member was limited by flexural yielding at the critical section rather than by yielding in shear. This yielding clearly indicates the interaction of shear and bending.

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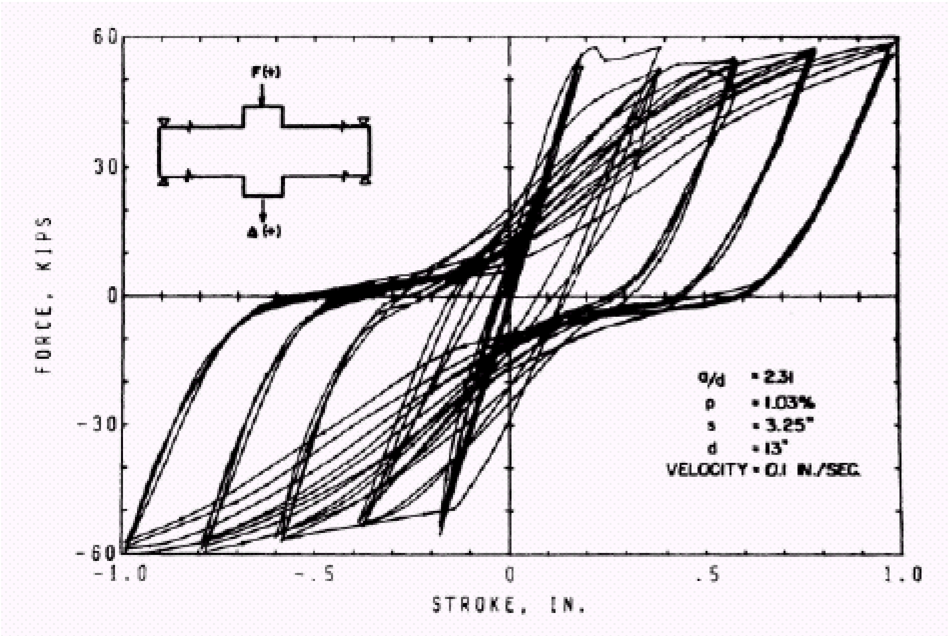


Figure 6-4: Shear deformation characteristics (Celebi and Penzien 1973).

The pinching in the force-deformation curve is obviously less desirable. The shear span to effective depth ratio is the most significant parameter. Decreasing the shear span to depth ratio causes a more pronounced pinching in the curve, and a faster degradation of the hysteretic energy-dissipating capacity. Considerable improvements in delaying and reducing the degrading effects can be accomplished by using closely spaced ties. Existence of axial force tends to retard the decrease in stiffness and resistance with cycles. However, it is hard to eliminate this undesirable effect when high shear stress exists. Consequently, it becomes important to include this degrading behavior in a behavioral model for a short, deep reinforced concrete member. The current state of knowledge is not sufficient to define the stiffness degrading parameters on the basis of the member geometry and material properties.

6.6 Bar slip and bond deterioration

When a structural element is framed into another element, some deformation is initiated within the other element. Consider a beam-column subassembly. Bertero and Popov (1977) reported a significant rotation at a beam end caused by the slippage (pullout) of the beam's main longitudinal reinforcement within the beam-column joint (Figure 6-5). The general shape of the moment-bar slip rotation curve is similar to that shown in Figure 6-3, demonstrating a pronounced pinching of a hysteresis loop. The contribution of bar slip to total deformation cannot be neglected, especially in a stiff member (short or deep).

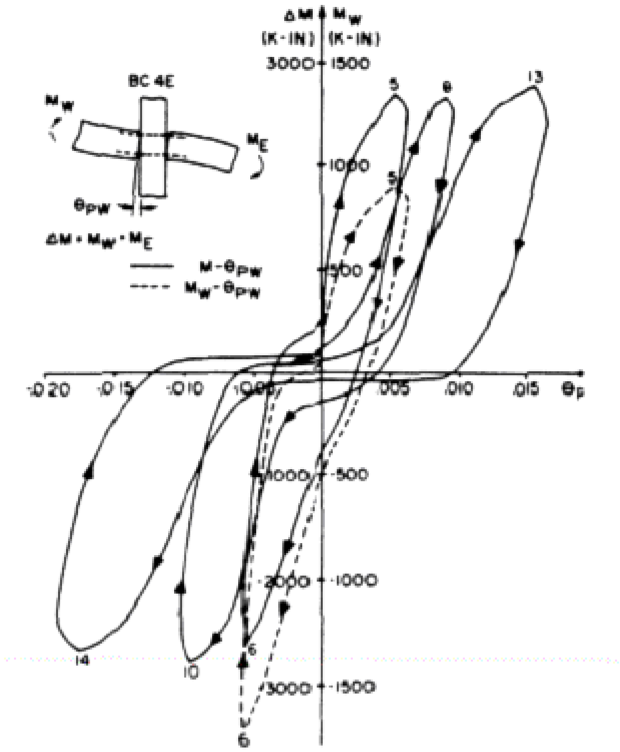


Figure 6-5: Rotation due to the bar slip (Bertero and Popov 1977).

6.7 Biaxial lateral load reversal

During an earthquake, columns of a framed structure must resist lateral forces simultaneously in longitudinal and transverse directions. Recent tests at the University of Toronto (Otani et al.1979) on reinforced concrete columns showed that the columns under lateral load reversals in two perpendicular directions exhibited the decay of resistance and stiffness at a faster rate than those under uniaxial lateral load reversals.

6.8 Hysteretic system

Most materials exhibit an hysteretic type of behavior when it strained beyond there elastic limit, so that system made of such material evolve as functions not just of the present values of state variables: displacement and velocity, but on the past history as well. Several laws of differential type for the modeling of hysteretic behavior are available; the most originally proposed by Bouc, and later generalized by Wen, Baber and Noor in conjunction with stochastic linearization. If u is the displacement and h the force, this law can be expressed as follows:

$$H = k \cdot [\alpha \cdot u + (1-\alpha) \cdot z] \tag{6.1}$$

Where α is a parameter defining the relative importance of the elastic term, and of the hysteric one; z, this latter being the solution of the differential equation (6.2).

$$z' = Au' - \gamma u' \beta|u'| |z|^{(n-1)} z \quad (6.2)$$

Where A, γ, β and n are parameters governing the shape of the hysteretic cycle. The last equation has been further extending in different directions: to describe material degradation, to cover vector stress state, and to include asymmetry of behaviour with inversion of sign of the sign of force [23].

6.9 Hysteretic Models for Reinforced Concrete

Nonlinear dynamic analysis of a reinforced concrete structure requires two types of mathematical modeling:

(a) Modelling for the distribution of stiffness along a member.
(b) Modelling for the force-deformation relationship under stress reversals.

A hysteretic model must be able to provide the stiffness and resistance under any displacement history. At the same time, the basic characteristics need to be defined by the member geometry and material properties. The current state of knowledge is sufficient to define flexural hysteretic models. However, it is not sufficient to determine the degree of stiffness degradation due to the deterioration of shear-resisting and rebar-concrete bond mechanisms.

6.9.1 Bilinear Model

Many investigators used the elastic- perfectly plastic hysteretic model because the model was simple. The maximum displacement of an elasto-plastic simple system was found (Veletsos and Newmark 1960) to be practically the same as that of an elastic system having the same initial period of vibration as long as the period was longer than 0.5 s. A finite positive slope was assigned to the post yield stiffness to account for the strain-hardening characteristic, and the model was called a bilinear model. The bilinear model (Figure 6-6) does not represent the degradation of loading and unloading stiffness with increasing displacement amplitude reversals, and the model is not suited for a refined non-linear analysis of a reinforced concrete structure.

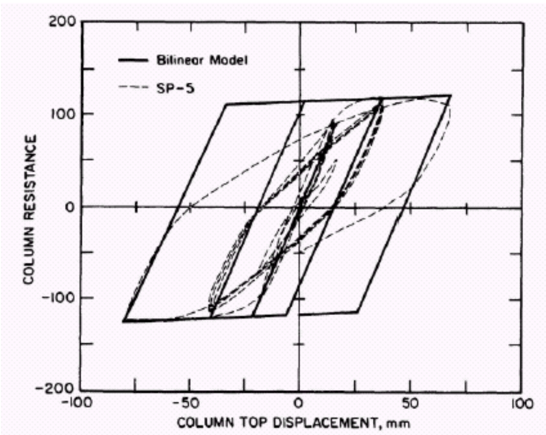


Figure 6-6: Bilinear hysteresis model

6.9.2 Degradation and hysteresis rules

Degradation rules and criteria are used for the modelling of the reinforced concrete member behaviour. Many of the rules allow for degradation of the strength of the members. The Yield Forces or Yield Moments may degrade as a function of the member ductility or the number of cycles of inelastic action. This is independent of the stiffness degradation associated with a hysteretic rule itself. There are, however, some hysterics rules that have their own built-in degradation of strength [27]. The degradation of the strength model is shown in Figure 6-7. The coefficients used in these models depend on the material concrete and steel types and the details of the cross sections, the confinement of the core.

Some of the available rules used in Ruaumoko program are shown in Figure 6-8 .For more information about these rules and its parameters refer to Ruaumoko manual.

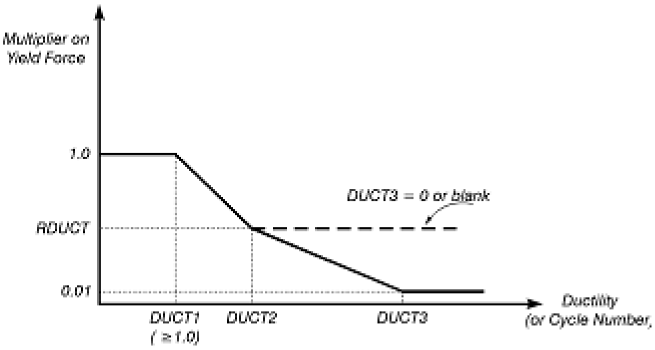


Figure 6-7: Degradation of the strength model

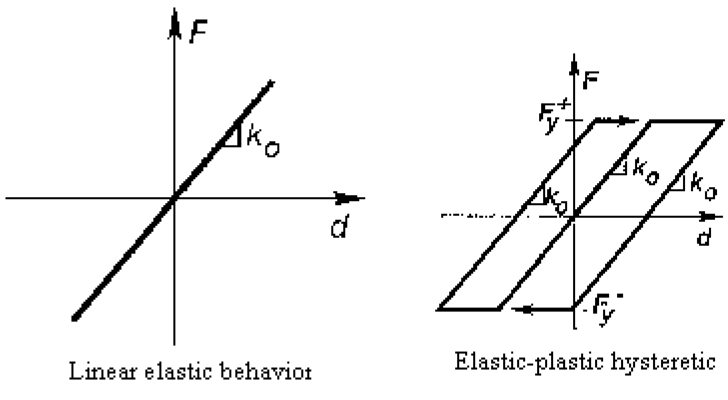


Figure 6-8: Linear elastic behaviour, no hysteretic behavior occurs. And Elastic-plastic hysteretic, the simplest hysteretic model with no stiffness shown after yielding

7 Nonlinear rectangular-section behavior of R/C members

For the seismic performance of the connections deferent materials required to be investigated, the conventional concrete and the Hybrid fibre concrete, which posses high ductility and performs good under cyclic loading. The behaviour definition of the R/C section is required in the solution of the strength, stiffness, and ductility. To represent the properties of the post yield response of the members, the complex behaviour of the steel and concrete must be considered, such as the no linearity of the concrete and steel, crushing and cracking of the concrete, and bond bar slip between concrete and bars. A non-linear behaviour model for rectangular R/C cross section is built for concrete B55, and for hybrid fibre concrete. The compression and tension behaviour of the concrete are defined by parameters enables the substitution of the stress strain relations into the equilibrium equations

The non-linear continuum modelling of reinforced concrete includes models applicable in structural engineering for problems that can be simplified into assemblage of beams and bars. To represent the properties of the post yield response of the members, the complex behaviour of the steel and concrete must be considered, such as the no linearity of the concrete and steel, crushing and cracking of the concrete, and bond bar slip between concrete and bars.

For the design of the concrete elements in our study, a non-linear behaviour model for rectangular R/C cross section is built. This model is based on the non-linear behaviour of the concrete and the steel materials. The concrete stress strain relation is represented with Hogenstad's parabola. Other model for the stress strain relations for ultra high performance concrete, hybrid fibre concrete HFC has been developed. These models are given in the following pages and they are considered as the consistent relations for the non-linear analysis.

In our model the compression and tension behaviour of the concrete and steel are studied separately. That enables not the identification of the compression and tension only, but it also enables to apply factors concerns the crushing, cracks and bar slip. The formulation of the compression and tension zone is addressed in parameters. These parameters are solved for rectangular section the solution can be extended for other shapes of conventional reinforced concrete, or HFC sections.

The concrete material is B55 and the steel of grade 50 is used in the analysis. The method used in the non-linear analysis, is a general method for monotonic loading. It is applicable for the different concrete types and for the HFC, The solutions of the parameters are also general and it is solved in our study for the specific type of concrete B55, and for the hybrid fibre concrete (HFC).

7.1 Nonlinear analyses of the rectangular R/C sections

The non-linear analyses use the equilibrium equations applied on transverse cross section under moment and normal force. The stress stain relations and material parameters are substituted in the equilibrium equations. The material parameters α and β are solved and defined locally for the given materials, this solution includes the limitations at yield, and ultimate limit state for a typical rectangular section. The parameters are given at the end of the results as a function of the concrete strain at the top and the bottom face of the element section. The representations of these parameters as a function of the strain enables not only use it directly in the equilibrium equations in the static cases, but it enables to solve the dynamic problems. In the following sections a brief description of this method. The equilibrium equations can be formulated recognizing the acting forces on the R/C section (Figure 7-1).

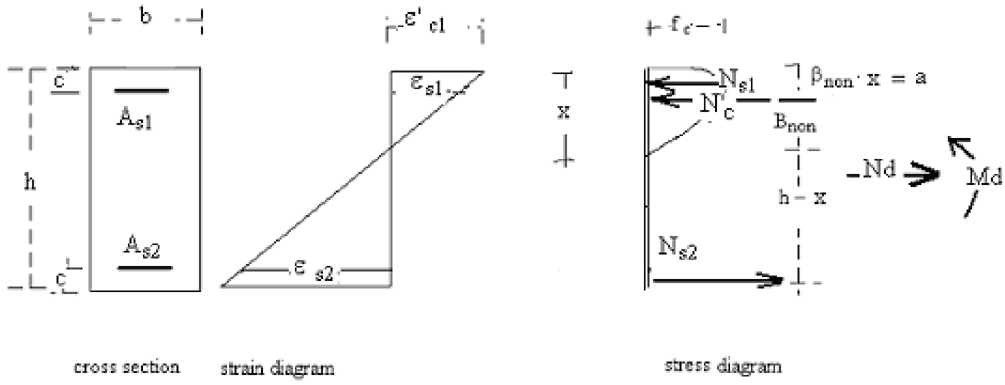


Figure 7-1: Stress strain diagram for rectangular reinforced concrete cross section subjected to moment and axial force.

The force and the moment equilibrium equations may be built as follows:

$$N_{s2} + N_d - N'_c - N_{s1} = 0 \tag{7.1}$$

$$M_d - N_{s1} \cdot \left(\frac{h}{2} - c\right) - N'_c \cdot \left(\frac{h}{2} - a\right) - N_{s2} \cdot \left(\frac{h}{2} - c\right) = 0 \tag{7.2}$$

From the strain diagram the neutral axis from the top concrete compression face may be formulated as follows:

$$x = \frac{\left|\varepsilon'_{c1}\right| \cdot (h - c)}{\left|\varepsilon'_{c1}\right| + \varepsilon_{s2}} = k_x \cdot (h - c) \tag{7.3}$$

And with:

$$\varepsilon_{s1} = \varepsilon'_{c1} \cdot \frac{x - c}{x} \qquad \kappa = \frac{\varepsilon'_{c1}}{x} \tag{7.4}$$

The acting forces on the cross section may be given as follows:

$$N'_c = \alpha \cdot b \cdot x \cdot \sigma'_{c1}$$

$$N_{s1} = A_{s1} \cdot \sigma_{s1}$$

$$N_{s2} = A_{s2} \cdot \sigma_{s2}$$

(7.5)

With α is the compression intensity factor. And β is the eccentric compression parameter.

Substitute the values of the normal forces; equations (7.5) in the equilibrium equations (7.1) and (7.2) .We obtain the following relations:

$$\frac{Nd}{b \cdot h \cdot f'_c} = \frac{A_{s1} \cdot \sigma_{s1} + \alpha \cdot b \cdot x \cdot \sigma'_{c1} \cdot - A_{s2} \cdot \sigma_{s2}}{b \cdot h \cdot f'_c} \tag{7.6}$$

Define the steel contribution factors as follows:

$$\omega_1 = \frac{A_{s1}}{b \cdot h} \qquad \omega_2 = \frac{A_{s2}}{b \cdot h} \qquad \psi_1 = \frac{\omega_1 \cdot f_s}{f''_c} \qquad \psi_2 = \frac{\omega_2 \cdot f_s}{f''_c} \tag{7.7}$$

Substitute for the steel area A_{s1} and A_{s2} of the equations (7.6) and (7.7) in equation (7.6), the moment and the normal force parameters n_d and m_d may be obtained. They are expressed in equation (7.8) and (7.9). These equations are applicable for the rectangular reinforced concrete suctions subjected to bending moment and axial force. The intensity and eccentric compression parameters α, and β can be solved for non-linear behaviour indicated as (α_{non}, and β_{non}). These parameters are solved in paragraph (7.3.2), and they are given as a function of the strain (Figure 7-5).

$$n_d = \frac{N_d}{b \cdot h \cdot f'_c} = \psi_1 \cdot \frac{\sigma_{s1}}{f_s} + \alpha_{non} \cdot k_x \cdot \left(1 - \frac{c}{h}\right) \cdot \frac{\sigma_{c1}}{f''_c} - \psi_2 \cdot \frac{\sigma_{s2}}{f_s} \tag{7.8}$$

$$m_d = \frac{M_d}{b \cdot h^2 \cdot f'_c} = \psi_1 \cdot \frac{\sigma_{s1}}{f_s} \cdot \left(\frac{1}{2} - \frac{c}{h}\right) \ldots$$
$$+ \alpha_{non} \cdot k_x \cdot \left(1 - \frac{c}{h}\right) \cdot \frac{\sigma_{c1}}{f'_c} \cdot \left[\frac{1}{2} - \beta_{non} \cdot k_x \cdot \left(1 - \frac{c}{h}\right)\right] ..$$
$$+ \psi_2 \cdot \frac{\sigma_{s2}}{f_s} \cdot \left(\frac{1}{2} - \frac{c}{h}\right) \tag{7.9}$$

Equation (7.8) and (7.9), can be built in interaction diagram, and in this study it have been solved through an iteration process after the substitute for the values of the concrete and steel stresses in terms of the strain. While the non-linear compression factors are given as a functions of the strain. The solution of the compression parameters is given in section 7.3.

7.2 Nonlinear analyses of rec. cross-section of hybrid fiber concrete HFC

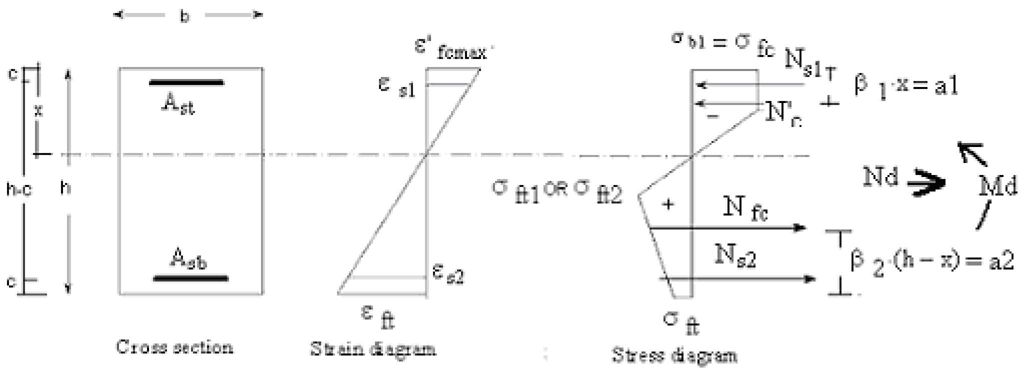


Figure 7-2: Stress strain diagram for (HFC) rectangular cross section subjected to moment and axial force.

The tension force of the fibre concrete works in the same direction of the tension reinforcement force. In the solution of the equilibrium equations we consider in this situation the tension parameters $\alpha 2$ and $\beta 2$ rather than the compression parameters $\alpha 1$ and $\beta 1$. The equations (7.1) and (7.2) can be extended to represent the equilibrium for moment and forces acting on the HFC cross section (Figure 7-2). The equations (7.10) and (7.11) are obtained.

As we see the relations in these equations, leads to use of the same procedure as in the conventional reinforced concrete which enables not only to solve the specific problem but to the benefit from the progress that have been made in the former concrete works, definitions, tables, procedures. The solution for HFC is a specific application, of this general formulation, and after taking in the considerations the material properties, and the strain limitations. The compression parameter (α_1 and $\beta_{1)}$, and the tension parameters ($\alpha 2$, $\beta 2$) are solved in section (7.4). They are shown as a function of the strain in Figure 7-12

The force and the moment equilibrium equations can be built recognizing the acting forces on the HFC section (Figure 7-2) as the follows

$$\left(N_{s2} + N_{ft}\right) + N_d - N'_c - N_{s1} = 0 \tag{7.10}$$

$$M_d - N_{s1} \cdot \left(\frac{h}{2} - c\right) - N'_b \cdot \left(\frac{h}{2} - a\right) - N_{s2} \cdot \left(\frac{h}{2} - c\right) - N_{ft} \cdot \left(\frac{h}{2} - \beta 2 \cdot x\right) = 0 \tag{7.11}$$

From the stress and strain diagram (Figure 7-2) the relations in equations (7.3) and (7.4) are obtained. Consider the compression parameters ($\alpha 1$,$\beta 1$), and the tension parameters ($\alpha 2$, $\beta 2$) and the acting force on the cross section may be obtained as follows:

$$a1 = \beta 1 \cdot x \qquad a2 = \beta 2 \cdot (h - x) \tag{7.12}$$

$$N'_c = \alpha 1 \cdot b \cdot x \cdot \sigma'_{c1}$$

$$N_{s1} = A_{s1} \cdot \sigma_{s1}$$

$$N_{ft} = \alpha_2 \cdot b \cdot (h - x) \cdot \sigma_{ft}$$

$$N_{s2} = A_{s2} \cdot \sigma_{s2} \tag{7.13}$$

Substitute the values of forces in equations (7.13) in the equilibrium equations (7.10) and (7.11), the following relation can be obtained:

$$\frac{N_d}{b \cdot h \cdot f'_c} = \frac{A_{s1} \cdot \sigma_{s1} + \alpha 1 \cdot b \cdot x \cdot \sigma'_{c1} \cdot - A_{s2} \cdot \sigma_{s2} - \alpha_2 \cdot b \cdot (h - x) \cdot \sigma_{ft}}{b \cdot h \cdot f'_c} \tag{7.14}$$

And

$$\frac{M_d}{b \cdot h^2 \cdot f'_c} = \frac{A_{s1} \cdot \sigma_{s1} \cdot z_{s1} + \alpha 1 \cdot b \cdot x \cdot \sigma'_{c1} \cdot \left(\frac{h}{2} - a\right) + A_{s2} \cdot \sigma_{s2} \cdot z_{s2} + \alpha_2 \cdot b \cdot (h - x) \cdot \sigma_{ft} \cdot z_{ft}}{b \cdot h^2 \cdot f'_c} \tag{7.15}$$

The definitions of the steel ratio and steel ration and the steel contribution factors ψ in equation (7.7) are applicable also for the HFC concrete. Substitute for these values in equation (7.14) and (7.15). We obtain equation (7.16), (7.17), which expressed the compression and the moment equilibrium relations for the HFC cross-section. These equations are applicable for the rectangular HFC cross suctions subjected to bending moment and axial force. The compression and tension parameters $\alpha 1, \beta 1, \alpha 2, \beta 2$ are solved to describe the non-linear behaviour of the steel and the HFC in section 7.4, Equation.(7.16) And (7.17), can be built in interaction diagram, and in this study it have been solved through an iteration process after the substitute for the stress and of the concrete and the steel in terms of the strain. While the non-linear compression factors are given as a function of the strain. The solution of these equations can be carried out simply by direct substitution of the compression and the tension parameters after obtaining its values in Figure 7-12.

$$n_{df} = \frac{N_d}{b \cdot h \cdot f'_c} = \psi_1 \cdot \frac{\sigma_{s1}}{f_s} + \alpha 1 \cdot k_x \cdot \left(1 - \frac{c}{h}\right) \cdot \frac{\sigma_{c1}}{f'_c} - \psi_2 \cdot \frac{\sigma_{s2}}{f_s} - \alpha_2 \cdot \left[1 - kx \cdot \left(1 - \frac{c}{h}\right)\right] \cdot \frac{\sigma_{ft}}{f'_c} \tag{7.16}$$

$$\begin{aligned} m_{df} = \frac{M_d}{b \cdot h^2 \cdot f'_c} = {} & \psi_1 \cdot \frac{\sigma_{s1}}{f_s} \cdot \left(\frac{1}{2} - \frac{c}{h}\right) + \alpha 1 \cdot kx \cdot \left(1 - \frac{c}{h}\right) \cdot \frac{\sigma'_{c1}}{f'_c} \cdot \left[\frac{1}{2} - \beta 1 \cdot kx \cdot \left(1 - \frac{c}{h}\right)\right] \ldots \\ & + \psi_2 \cdot \frac{\sigma_{s2}}{f'_c} \cdot \left(\frac{1}{2} - \frac{c}{h}\right) + \alpha_2 \cdot \left[1 - kx \cdot \left(1 - \frac{c}{h}\right)\right] \cdot \frac{\sigma_{ft}}{f'_c} \cdot \left[\frac{1}{2} - \beta_2 \cdot kx \cdot \left(1 - \frac{c}{h}\right)\right] \end{aligned} \tag{7.17}$$

7.3 Model for the concrete type B55

7.3.1 Unconfined stress strain relation

The concrete stress model is built using the stress strain relations based on Hogenstad's parabola. This formula describes the stress strain relations for confined concrete. And it can be calibrated for various degree of confinement. In our study the purpose of this model is to study the non-linear behaviour at the yield locations of the members ends. The Hogenstad's parabola is given in equation (7.18) and Figure 7-3. It have been applied for concrete type B55 for the strength calculations using the cylinder concrete strength f_c = 45 N/mm^2 and based on the CEB requirements. And it have been calibrated for unconfined concrete using the average stress of the concrete f_c = 55 N/mm^2, and the average elasticity of B55 for the stiffness calculations.

$$\sigma c(f_c, \varepsilon_{c1}, \varepsilon_c) = f_c \cdot \left[\left(\frac{2\varepsilon_c}{\varepsilon_{c1}}\right) - \left(\frac{\varepsilon_c}{\varepsilon_{c1}}\right)^2\right] \qquad \text{CEB 1993}$$

With $\varepsilon_{c1}(f_c, E_{c0}) = 2 \cdot \dfrac{f_c}{E_{c0}}$ Where f_c is the concrete cylinder strength

$$E_{c0}(f_c) = 2.15 \cdot 10^4 \cdot \left(\frac{f_c}{10}\right)^{\frac{1}{3}} \qquad \text{Suggested by CEB}$$

And $E_{c0}(45) \cdot \dfrac{N}{mm^2} = 3.55 \times 10^4 \dfrac{N}{mm^2}$

The strain corresponding to the stress $\sigma = fc$

$$\varepsilon_{c1}\left(45\frac{N}{mm^2}, 35500\frac{N}{mm^2}\right) = 2.535 \times 10^{-3} \qquad \text{For strength calculations}$$

$$\varepsilon_{c1}\left(55\frac{N}{mm^2}, 36000\frac{N}{mm^2}\right) = 3.056 \times 10^{-3} \qquad \text{For stiffness calculations}$$

(7.18)

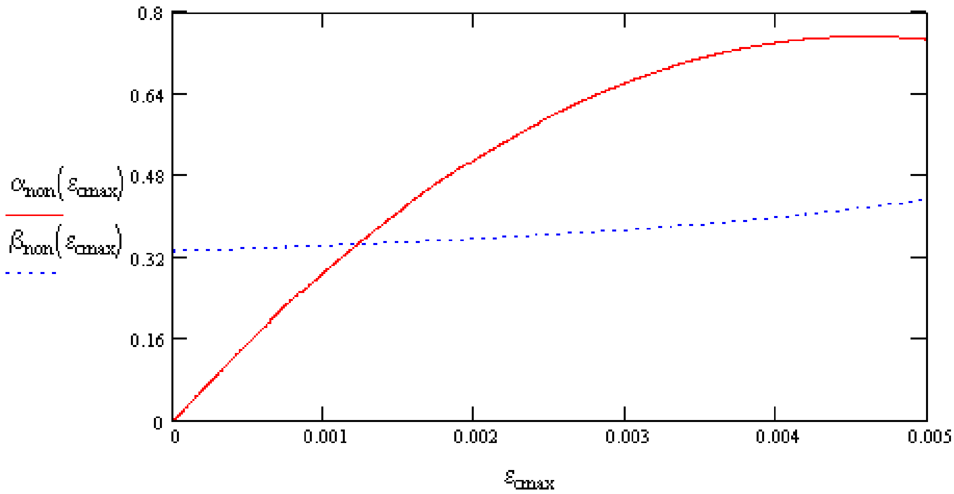


Figure 7-3: Stress strain diagram according to Hogenstad's parabola for B55

The stress strain relations are applied for the reinforced concrete section (Figure 7-4). The aim of this application is to obtain the compression parameters of the R/C rectangular concrete cross section for the non-linear analyses.

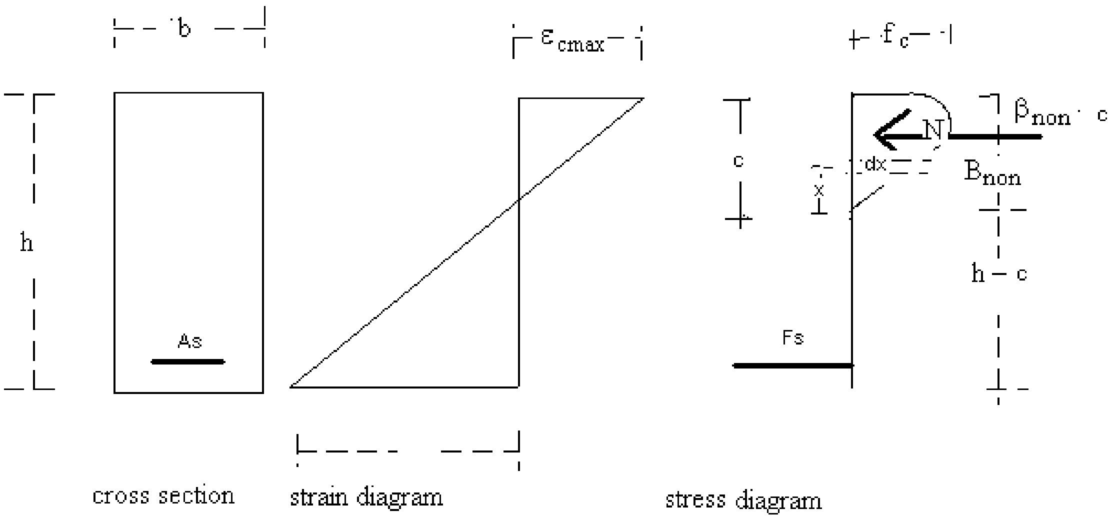


Figure 7-4: Stress strain diagrams at rectangular reinforced concrete cross-section

7.3.2 Compression parameters

The stress strain relations are applied for the reinforced concrete rectangular section (Figure 7-4) .The aim of this application is to obtain the compression and the tension parameters. To obtain the compression force of the concrete block apply the stress integration over the section as follows:

$$\frac{N}{b \cdot f_c} = \int_0^c \left[\left(\frac{2\varepsilon_c}{\varepsilon_{c1}} \right) - \left(\frac{\varepsilon_c}{\varepsilon_{c1}} \right)^2 \right] dx \qquad \text{and with} \qquad \varepsilon_c = x \cdot \frac{\varepsilon_{cmax}}{c}$$

$$\frac{N}{b \cdot f_c} = \int_0^c \left[\left(\frac{2\varepsilon_{cmax}}{\varepsilon_{c1} \cdot c} \right) \cdot x - \left(\frac{\varepsilon_{cmax}}{\varepsilon_{c1} \cdot c} \right)^2 \cdot x^2 \right] dx \tag{7.19}$$

Where ε_{cmax} is the maximum strain of the concrete and c is the depth of concrete compression block.

The compression α parameter may be defined as the proportion of the average stress to the maximum concrete stress; it is given in as non-linear parameter as follows:

$$\frac{N}{b \cdot f_c} = \frac{\varepsilon_{cmax}}{\varepsilon_{c1}} \cdot c - \frac{1}{3} \cdot \frac{\varepsilon_{cmax}^2}{\varepsilon_{c1}^2} \cdot c = c \cdot \alpha_{non} \tag{7.20}$$

And to obtain the distance of the concrete compression force N, from the neutral axis apply the momentum integration on the compression stresses of the concrete compression block and divided by the value of compression force N

$$B_{non} = \frac{1}{\frac{\varepsilon_{cmax}}{\varepsilon_{c1}} \cdot c - \frac{1}{3} \cdot \frac{\varepsilon_{cmax}^2}{\varepsilon_{c1}^2} \cdot c} \cdot \int_0^c \left[\left(\frac{2\varepsilon_{cmax}}{\varepsilon_{c1} \cdot c} \right) \cdot x - \left(\frac{\varepsilon_{cmax}}{\varepsilon_{c1} \cdot c} \right)^2 \cdot x^2 \right] \cdot x \, dx$$

$$B_{non} = \frac{c}{4} \cdot \frac{(3 \cdot \varepsilon_{cmax} - 8 \cdot \varepsilon_{c1})}{(-3 \cdot \varepsilon_{c1} + \varepsilon_{cmax})}$$

For the concrete B55 the strain $\varepsilon_{c1} = 2.535 \times 10^{-3}$ For strength calculations

$\varepsilon_{c1} = 3.056 \times 10^{-3}$ For stiffness calculations

Summary:

The Compression stress parameter in the rectangular concrete cross section can be given as a function of the maximum compression strain, using γ_m =1.2, and substitute for the strain $\varepsilon_{c1,}$ as follows:

For strength calculations:

$$\alpha_{non}(\varepsilon_{cmax}) = \frac{1}{1.2} \cdot \left(\frac{\varepsilon_{cmax}}{0.002535} - \frac{1}{3} \cdot \frac{\varepsilon_{cmax}^2}{0.002535^2} \right)$$

$$\beta_{non}(\varepsilon_{cmax}) = 1 - \frac{1}{4} \cdot \frac{(3 \cdot \varepsilon_{cmax} - 2.0280 \cdot 10^{-2})}{\left[-(7.605 \cdot 10^{-3}) + \varepsilon_{cmax}\right]} \tag{7.21}$$

And for stiffness calculations

$$\alpha_{non}(\varepsilon_{cmax}) = \frac{1}{1.2}\cdot\left(\frac{\varepsilon_{cmax}}{0.003056} - \frac{1}{3}\cdot\frac{\varepsilon_{cmax}^{\ 2}}{0.003056^2}\right)$$

$$\beta_{non}(\varepsilon_{cmax}) = 1 - \frac{1}{4}\cdot\frac{\left(3\cdot\varepsilon_{cmax} - 2.4448\cdot 10^{-2}\right)}{\left[-(9.168\cdot 10^{-3}) + \varepsilon_{cmax}\right]} \tag{7.22}$$

With:

$$\alpha_{non} = \frac{N}{b\cdot f_c\cdot c} \quad \text{And} \quad \beta_{non} = \frac{x_N}{c} \tag{7.23}$$

The parameters (α and β) are given as a function of the maximum compression strain, and can be used for the solutions of the reinforced concrete cross sections. They can be introduced in the moment and the normal force equilibrium equations. Figure 7-5 shows the parameters (α and β) for reinforced concrete B55

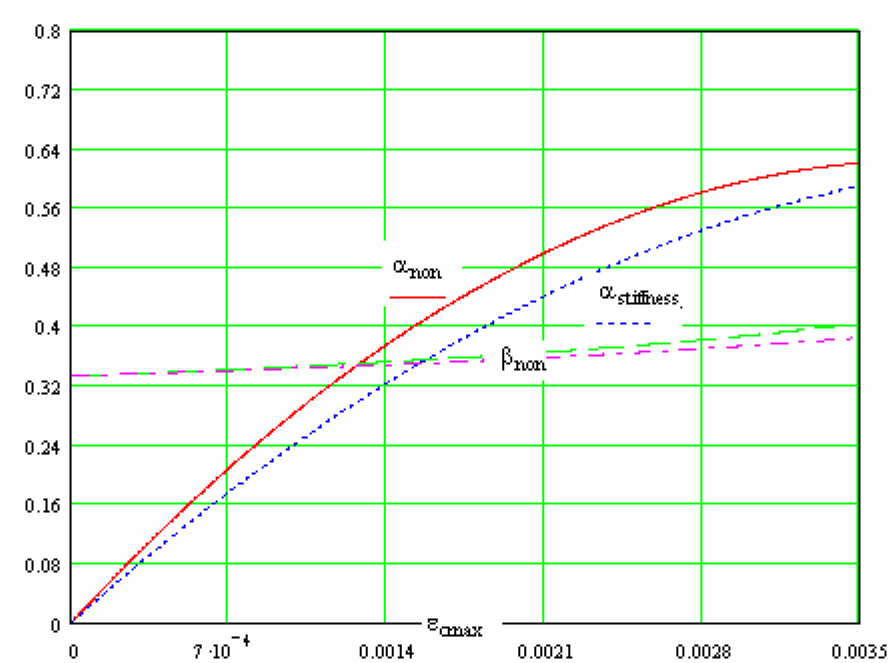


Figure 7-5: Compression parameters for confined concrete B55

7.4 Model for the hybrid fiber concrete, HFC

7.4.1 Stress strain relations

The stress parameters α and β are defined as a function of the strain in the tension and the compression block. In this paragraph the stress strain relations of the hybrid fibre concrete are defined. The data of the stress strain are laboratory test results on HFC (Figure 7-6 though Figure 7-7).

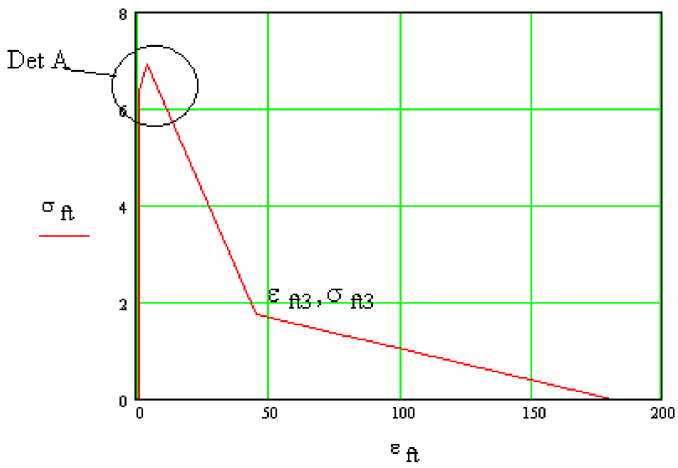


Figure 7-6: Tension stress strain relation of the HFC, stress [N/mm^2], strain in [mm/m]

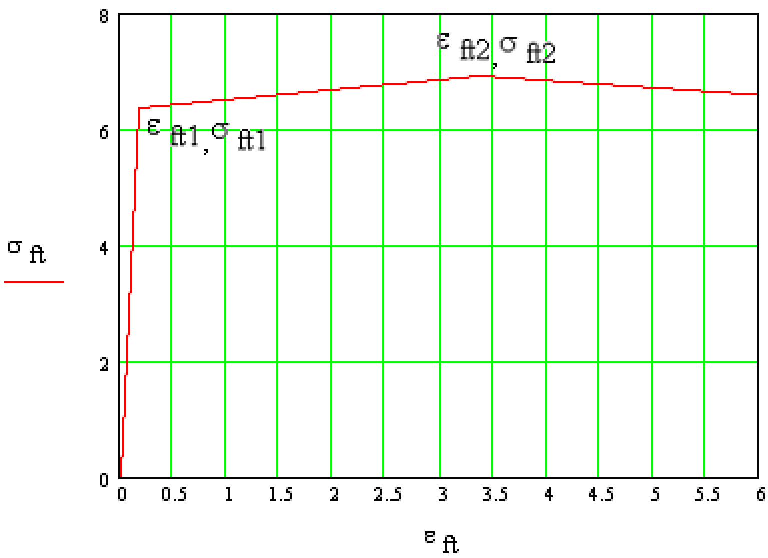


Figure 7-7: Tension stress strain relation of the HFC, stress [N/mm^2], strain [mm/m]

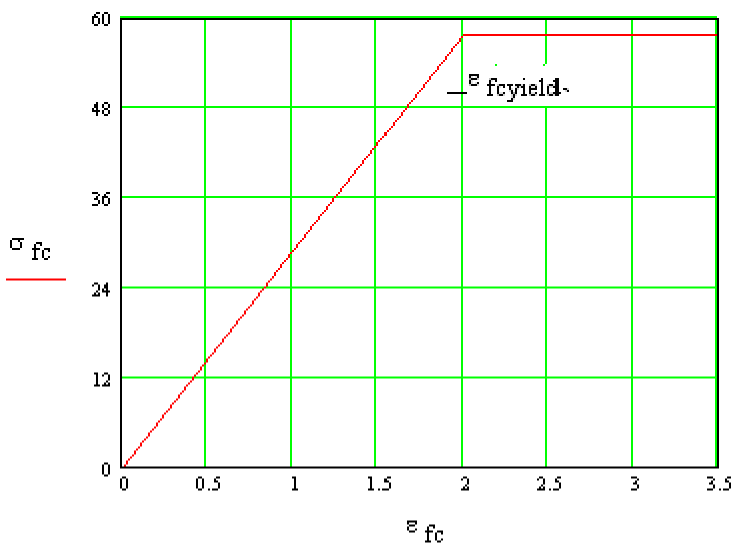


Figure 7-8: Compression stress strain relation of the HFC, stress [N/mm^2], strain [mm/m]

The stress strain relations as it is seen in the figure are continuous between each two points. The change of the slope of the lines between each two lines that intersect in an inflection point is a sudden change and for a given stress we get different values of strain that means that the concrete stress undergoes sudden change in the strain for a given loading level. Exceeding the tension strength at point 2 of (6.35 N/mm^2). The stress decrease rapidly, that may occur by repeating loading inside the section .for that the stress of the point of inflection 1 is considered as the maximum tension stress that will insure that the design tension stress value are bellow the material test values according to the AFGC group regulations for the ultra high performance fibre reinforced concrete UHPRC. The maximum compression strain is 3.5 mm/m, and the yield strain is 2 mm/m.

7.4.2 Tension and compression parameters

For the description of this behaviour the interpolation of the stress strain values is used, the limitations for the inflection points are required and it is applied as follows:

For the calculation of the parameters (α and β) it is needs to fix the stress and strain values at the inflection points and define its limitations, which can be recognized in the stress strain curves. Based on this information the parameters are calculated as a function of the strain through the internal relations in the compression stress block for the compression parameters and the tension stress block for the tension parameters, using the information provided in Figure 7-9 through Figure 7-11) as follows:

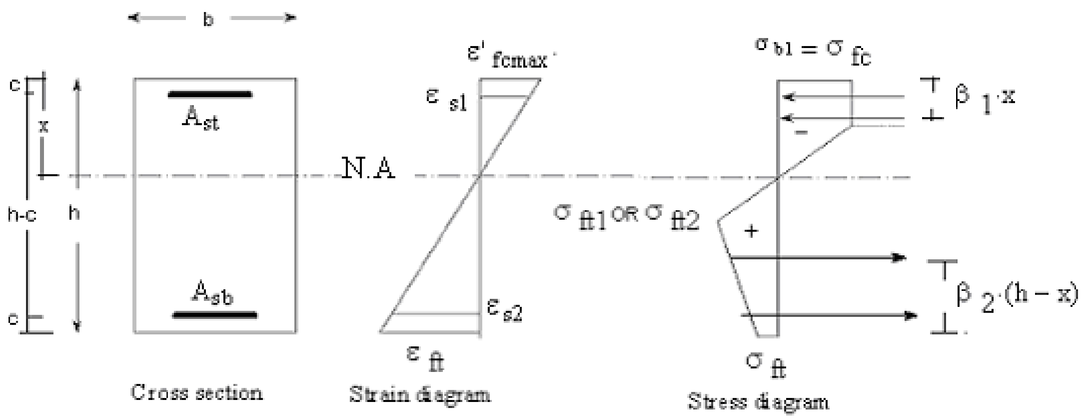


Figure 7-9: Stress and strain diagrams at the hybrid fiber concrete section exerted to bending moment

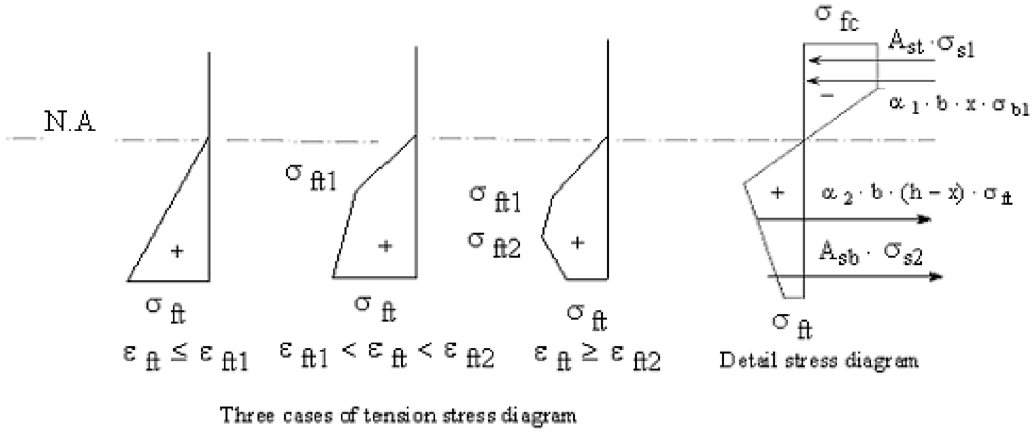


Figure 7-10: Stress and strain diagrams at the hybrid fiber concrete section exerted to bending moment

In order to define the parameters (α and β) in the tension part, integrate the stress at the tension area and divided it by the maximum stress multiplied by the tension the area (Figure 7-11), the parameters $\alpha 2$ and $\beta 2$ in the tension zone are defined in (7.24) to (7.30) using the stress strain relations in Figure 7-11 as follows:

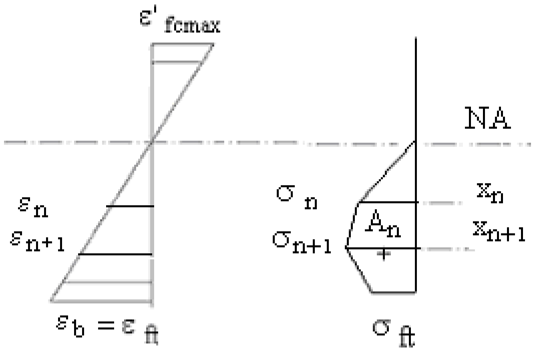


Figure 7-11: Stress and strain diagram at the tension zone

$$\alpha 2 = \frac{\sum_n A_n}{x_b \cdot \sigma_{ft}}$$

Or

$$\alpha 2 = \frac{1}{x_b \cdot \sigma_{ft}} \cdot \sum_n \frac{(\sigma_{n+1} + \sigma_n)}{2} \cdot (x_{n+1} - x_n) \tag{7.24}$$

$$\beta 2 = \frac{\sum_n M_n}{x_b \cdot \sum_n A_n} \tag{7.25}$$

The strain and the stress coordinate can be obtained from the strain diagram is defined as follows:

$$x_n = \frac{\varepsilon_n}{\varepsilon_b} \cdot x_b \qquad \text{And} \qquad x_{n+1} = \frac{\varepsilon_{n+1}}{\varepsilon_b} \cdot x_b \tag{7.26}$$

Introduce x in the above equations. That leads to define the parameters α and β as a function of the stresses and the strain at any distance x, which can be read from the stress strain diagram. And the strain at the bottom face

Apply that also for the compression zone. That leads to the relations and the limitations as follows:

The compression parameters are:

$$\alpha_1 = \begin{cases} 0.5 & \text{if } \varepsilon'_{fcmax} < (\varepsilon_{fcyield}) \\ \left(1 - \dfrac{\frac{\varepsilon_{fcyield}}{2}}{\varepsilon'_{fcmax}}\right) & \text{otherwise} \end{cases} \tag{7.27}$$

$$\beta_1 = \begin{cases} 0.33 & \text{if } \varepsilon'_{fcmax} < (\varepsilon_{fcyield}) \\ \left[\dfrac{0.5\left(\varepsilon'_{fcmax} - \varepsilon_{fcyield}\right)^2 + \left[0.33 \cdot \varepsilon_{fcyield} + \left(\varepsilon'_{fcmax} - \varepsilon_{fcyield}\right)\right] \cdot \frac{\varepsilon_{fcyield}}{2}}{\varepsilon'^{\,2}_{fcmax} \cdot \left(1 - \dfrac{\frac{\varepsilon_{fcyield}}{2}}{\varepsilon'_{fcmax}}\right)}\right] & \text{otherwise} \end{cases} \tag{7.28}$$

The tension parameters are given as follows:

$$\alpha 2 = \begin{cases} 0.5 & \text{if } \varepsilon_{ft} < (\varepsilon_{ft1} \leftarrow 0.239) \\ \left[0.5 \cdot \dfrac{\varepsilon_{ft1}}{\varepsilon_{ft}} \cdot \dfrac{\sigma_{ft1}}{\sigma_{ft}} + \dfrac{\sigma_{ft1} + \sigma_{ft}}{2 \cdot \sigma_{ft}} \cdot \left(1 - \dfrac{\varepsilon_{ft1}}{\varepsilon_{ft}}\right)\right] & \text{if } \varepsilon_{ft1} < \varepsilon_{ft} < \varepsilon_{ft2} \\ \left[0.5 \cdot \dfrac{\varepsilon_{ft1}}{\varepsilon_{ft}} \cdot \dfrac{\sigma_{ft1}}{\sigma_{ft}} + \dfrac{\sigma_{ft1} + \sigma_{ft2}}{2 \cdot \sigma_{ft}} \cdot \left(\dfrac{\varepsilon_{ft2}}{\varepsilon_{ft}} - \dfrac{\varepsilon_{ft1}}{\varepsilon_{ft}}\right) + \dfrac{\sigma_{ft2} + \sigma_{ft}}{2 \cdot \sigma_{ft}} \cdot \left(1 - \dfrac{\varepsilon_{ft2}}{\varepsilon_{ft}}\right)\right] & \text{if } \varepsilon_{ft} \geq \varepsilon_{ft2} \end{cases} \tag{7.29}$$

$$\beta 2 = \begin{cases} \frac{1}{3} & \text{if } \varepsilon_{ft} \le \left(\varepsilon_{ft1} \leftarrow 0.239\right) \\ \dfrac{\frac{\varepsilon_{ft1} \cdot \sigma_{ft1}}{2 \cdot \varepsilon_{ft}} \cdot \left(1 - \frac{2}{3} \cdot \frac{\varepsilon_{ft1}}{\varepsilon_{ft}}\right) + \frac{\sigma_{ft1}}{2} \cdot \left(1 - \frac{\varepsilon_{ft1}}{\varepsilon_{ft}}\right)^2 + \frac{\sigma_{ft} - \sigma_{ft1}}{2 \cdot 3} \cdot \left(1 - \frac{\varepsilon_{ft1}}{\varepsilon_{ft}}\right)^2}{\left[0.5 \cdot \frac{\varepsilon_{ft1}}{\varepsilon_{ft}} \cdot \sigma_{ft1} + \frac{\sigma_{ft1} + \sigma_{ft}}{2} \cdot \left(1 - \frac{\varepsilon_{ft1}}{\varepsilon_{ft}}\right)\right]} & \text{if } \varepsilon_{ft1} < \varepsilon_{ft} < \varepsilon_{ft2} \\ \dfrac{\left[\begin{array}{l} \frac{\varepsilon_{ft1} \cdot \sigma_{ft1}}{2 \cdot \varepsilon_{ft}} \cdot \left(1 - \frac{2 \varepsilon_{ft1}}{3 \varepsilon_{ft}}\right) + \frac{\sigma_{ft2}}{2} \cdot \left(\frac{\varepsilon_{ft}}{\varepsilon_{ft}} - \frac{\varepsilon_{ft1}}{\varepsilon_{ft}}\right)^2 - \frac{\sigma_{ft2} - \sigma_{ft}}{2 \cdot 3} \cdot \left(1 - \frac{\varepsilon_{ft2}}{\varepsilon_{ft}}\right)^2 \dots \\ + \frac{\sigma_{ft1} - \sigma_{ft2}}{2} \cdot \left(\frac{\varepsilon_{ft2}}{\varepsilon_{ft}} - \frac{\varepsilon_{ft1}}{\varepsilon_{ft}}\right) \cdot \left(1 - \frac{2 \cdot \varepsilon_{ft2}}{3 \cdot \varepsilon_{ft}} - \frac{\varepsilon_{ft1}}{\varepsilon_{ft}}\right) \end{array}\right]}{\left[0.5 \cdot \frac{\varepsilon_{ft1}}{\varepsilon_{ft}} \cdot \sigma_{ft1} + \frac{\sigma_{ft1} + \sigma_{ft2}}{2} \cdot \left(\frac{\varepsilon_{ft2}}{\varepsilon_{ft}} - \frac{\varepsilon_{ft1}}{\varepsilon_{ft}}\right) + \frac{\sigma_{ft2} + \sigma_{ft}}{2} \cdot \left(1 - \frac{\varepsilon_{ft2}}{\varepsilon_{ft}}\right)\right]} & \text{if } \varepsilon_{ft} \ge \varepsilon_{ft2} \end{cases} \tag{7.30}$$

The parameters (α and β) are given as a function of the strain at the cross section exerted to bending moment. α1 and β1 are given as a function of the top compression strain, while (α2 and β2) are given as a function of the bottom tension strain They are independent of the axial force while the relations are obtained according to the stain relations, considering the rotation of the plane and do not consider the equilibrium condition at the cross section. These parameters may use for the flexure moment and normal forces equilibrium equations. Their values can be obtained as a function of the strain and introduced into these equations. And they can be obtained directly from the curves and used in simple formulas to solve the bending moment problem for reinforced hybrid fibre concrete cross section.

The compression and tension parameters for the hybrid fibre concrete α and β are shown in Figure 7-12. Those are constant until the top strain at the section reaches 2mm/m and it grows rapidly up to a constant value when the strain reaches 10mm/m. The compression parameters at the yielding of concrete are as follows: α1 = 0.50, β1 = 0.33 up to the strain of 2mm/m, α1 grouse up to 0.85 at strain of 10 mm/m, while the value of β1 gross with a small increment up to 0.42 at strain of 10 mm/m.

The compression parameters at the yielding of concrete are as follows: α2 = 0.5, β2 = 0.33 up to the cracking bottom tension strain of 2mm/m, α2 gross up to 1 at strain of 5 mm/m, while the value of β1 gross with a small increment up to 0.49 at tension strain of 4.7 mm/m.

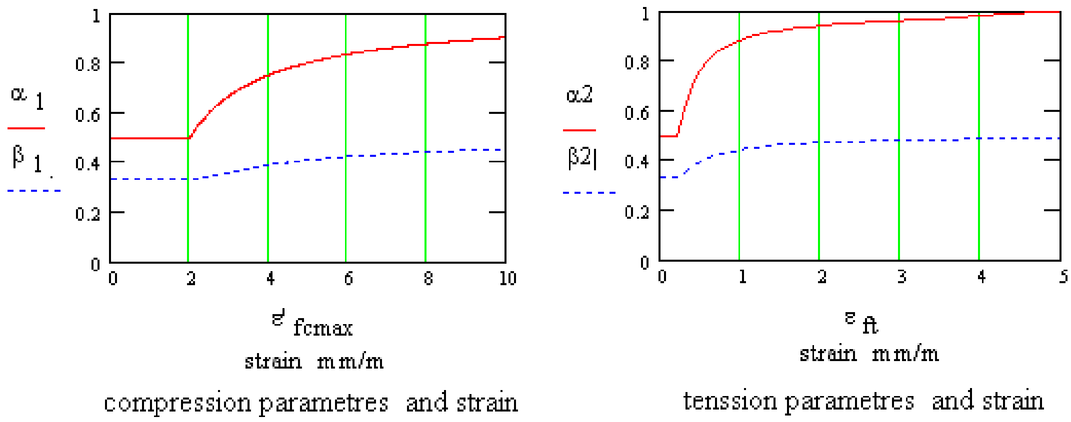


Figure 7-12: The tension and the compression parameters of the hybrid fiber concrete versus the strain

7.5 Model for the steel grade 50, stress strain relations

A simplified stress strain model is adopted for both tensile and compressive loads. The stress strain relations are described in Figure 7-13. The monotonic stress strain relation for the reinforced concrete is described through a suitable set of equations (7.31) as follows:

The steel stress is given as a function of the strain. The steel elasticity of Es=200000 N/mm^2 and yield stress of 435 N/mm^2, the strain hardening ε_{sh} = 0.02, and the yield strain εy = 0.002175,

$$\sigma_s(\varepsilon_s) = \begin{cases} (E_s \leftarrow 200000)\cdot\varepsilon_s & \text{if } \varepsilon_s \le \left(\varepsilon_y \leftarrow \dfrac{435}{200000}\right) \\ (E_s \leftarrow 200000)\cdot\varepsilon_y\cdot\left[1+\dfrac{0.1\cdot(\varepsilon_s-\varepsilon_y)}{\varepsilon_{sh}\leftarrow 0.02-\varepsilon_y}\right] & \text{if } \varepsilon_y \le \varepsilon_s < (\varepsilon_{sh} \leftarrow 0.02) \\ (f_{ye} \leftarrow 435)\cdot\left[1.5-0.5\cdot\left(\dfrac{0.12-\varepsilon_s}{0.112}\right)^2\right] & \text{otherwise} \end{cases} \tag{7.31}$$

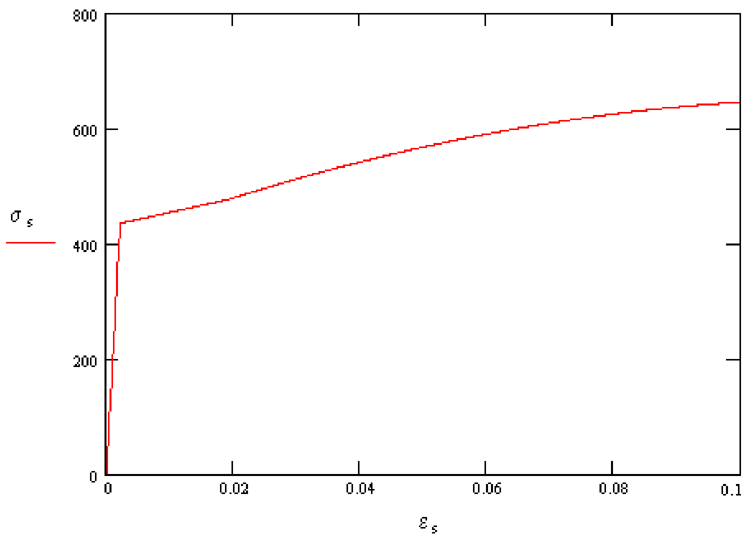


Figure 7-13: Stress strain diagram for steel grade 50

8 Analyses method approaching the nonlinear structural behavior

The stiffness of the concrete members is a factor influence the distribution of the applied loads on the structure, and it is subjected to the change under the loading and the degradation influence during the seismic action. The stiffness of the R/C section increased with the increase of the reinforcement ratio. The demand influence the stiffness of the cross section, and in the member ends that influence is high under seismic action. The application of the relation load-stiffness-reinforcement ratio-ductility- enables building model for the precast concrete beam. Introducing that in a concrete structure subjected to seismic action shows that the use of the effective stiffness obtained from the local section properties can provides description for the structure under seismic loading, and may be used in the design as a redistribution method for the applied bending moment in the beam ends. The redistribution of the seismic force varies between 20% for the post yielding condition up to 35% when the degradation is considered. Description for the stiffness, its relations, and use are given in this chapter. The non-linear analyses are applied for the most significant moments in the beams at each level of the building structure in this study. The analyses results are given in interactive tables information, which enables for the design and control at the yield moment and at the ultimate limit state for the reinforced concrete cross sections. These results are given in figures. This application provides the relation between the stiffness and the reinforcement ratio, which enables in building a model for a typical beam for the precast use. Structural analysis model with modified stiffness is introduced.

8.1 The ultimate and yield moment

For the control of the yield location, the behaviour of the beams needed to be defined. The most needed information is the ultimate and the yield moments. In our study we have a precast concrete building. It has three typical beams cross-sections. One of the applications for the non-linear analysis (chapter 7) is to built curves for the moment and the reinforcement ratio for the different cross section. In the seismic design we have minimum reinforcement ratio of 50% of the tension reinforcement in the compression zone. This analysis is curried out for B55 r/c sections.

The ultimate and the yield moments (M_u, M_y) verse the reinforcement ratio (ω) are shown in Figure 8-1 for three beams types with the dimensions as follows: 700×500mm, 600×500mm and 600×400mm.

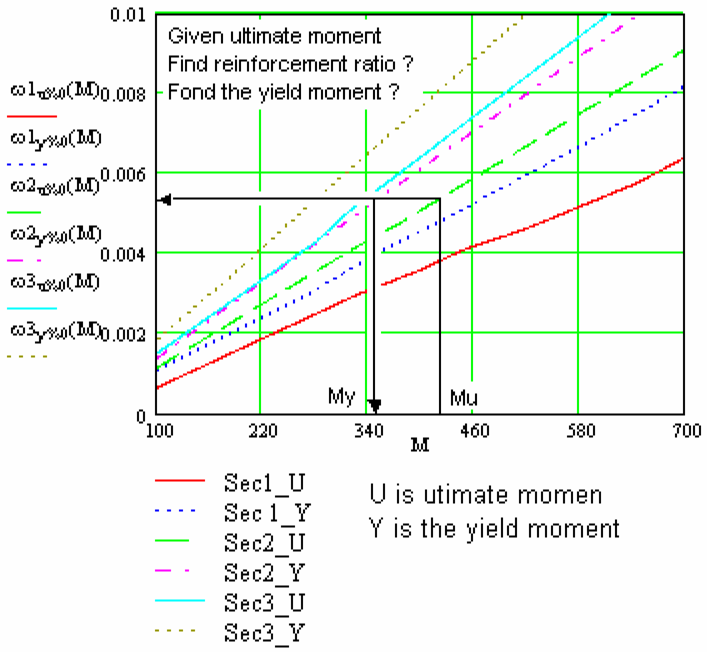


Figure 8-1: The ultimate and yield moment verse reinforcement ratio for the B55, rectangular R/C section

8.2 The effective stiffness

In order to build representative model link the linear solutions, for the non-linear analyses, the definition of the effective stiffness of the members (K_{eff}) is required. K_{eff} may be obtained through the moment curvature relations (Figure 8-2).

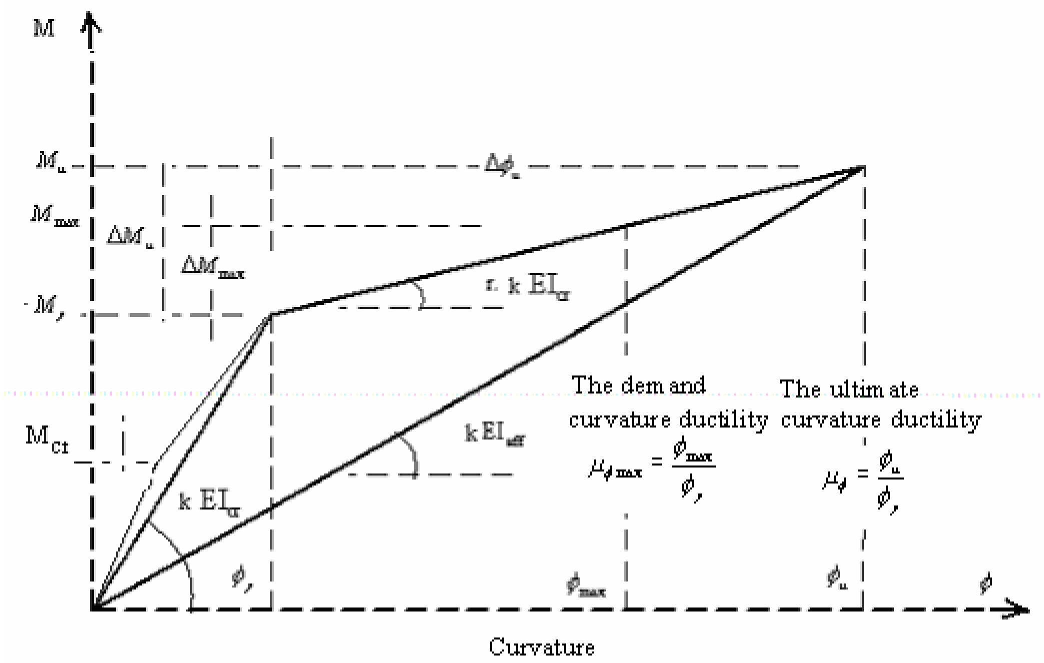


Figure 8-2: Typical beam moment curvature response

The relation between the initial stiffness and the post yield stiffness shown in Figure 8-2 are expressed in equation (8.1).

$$EI_{eff} = \frac{EI_{cr}(1+ r(\mu_b -1))}{\mu_b} \qquad (8.1)$$

Where:

μ_b is the expected beam rotation ductility.

EI_{cr} is the cracked section stiffness at the effective yield.

r is the ratio of postyield to preyield stiffness.

The internal relations in the moment curvature diagram (Figure 8-2) enable defining the relations of the moment, curvature, stiffness and the ductility in equations (8.2) through (8.9). The difference between the ultimate and the yield curvature is given as follows:

$$\Delta\phi = \phi_u - \phi_y \qquad (8.2)$$

And for a certain local maximum moment, M_{max}, the effective stiffness is calculated as follows:

$$\Delta M = M_{max} - M_y \qquad (8.3)$$

$$\Delta M_u = M_u - M_y \qquad (8.4)$$

The curvature as a function of the maximum moment is formulated as follows:

$$\phi_{max} = \phi_y + \Delta\phi_u \frac{\Delta M_{max}}{\Delta M_u} \qquad (8.5)$$

The demand curvature ductility is defined as the required curvature ductility in the section so that the section reaches certain curvature ϕ_{max} corresponds to the moment M_{max}. The ultimate curvature ductility is defined as the available ductility in the section so that the curvature reaches the ultimate curvature ϕ_u corresponds to the ultimate moment M_u; those are formulated in equation (8.6) and (8.7) as follows:

$$\mu_{\phi\max} = \frac{\phi_{max}}{\phi_y} \quad \text{And} \quad \mu_{\phi} = \frac{\phi_u}{\phi_y} \qquad (8.6) \text{ and } (8.7)$$

Consider that the stiffness is given as follows: K= C.EI

With:

$$r = \frac{\phi_y \cdot \Delta M_u}{M_y \cdot \Delta\phi_u} \quad , \qquad (8.8)$$

The effective stiffness may be formulated as follows:

$$EI_{eff} = \frac{EI_{cr}(1+r(\mu_b-1))}{\mu_b} \quad \text{For } \mu_{\phi max} \geq 1$$

Or (8.9)

$$EI_{eff} = K_y \quad \text{For } \mu_{\phi max} \text{ p } 1$$

Using the relations in (8.6) through (8.9), remember that the ultimate and the yield moments are functions of the reinforcement ratio the ductility can be built as a function of the reinforcement ratio, the loading level and the effective stiffness can be built as a function of the reinforcement ratio.

Beyond the post yield of the members, the effective stiffness is increased rapidly with the increase of reinforcement ratio, and the demand ductility decreases as the reinforcement ratio increased. The demand ductility for certain reinforcement ratio increased as the loading on the member increase. These relations are shown in Figure 8-3; the effective stiffness versus the reinforcement ratio, and the ductility versus the applied moment, for a certain local ductility the design load level can be obtained.

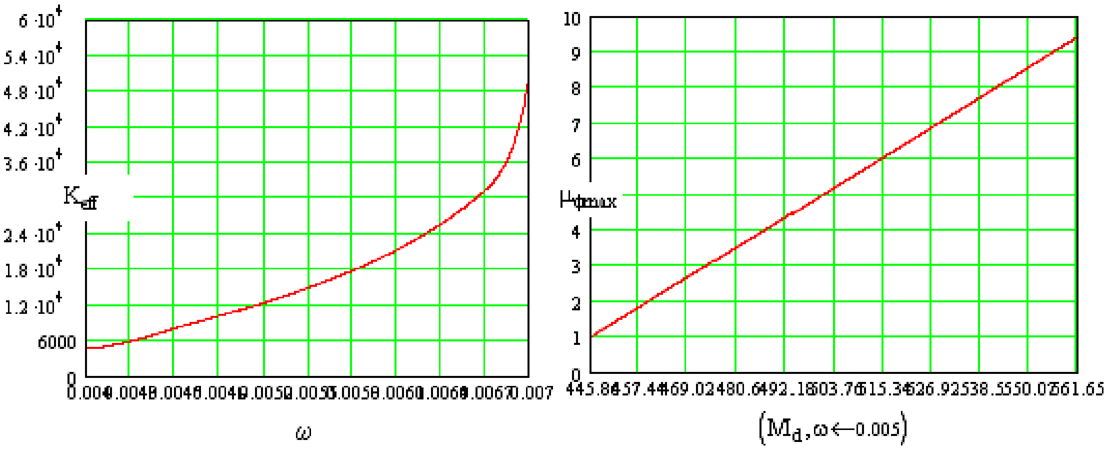


Figure 8-3: The effective stiffness versus the reinforcement ratio, and the ductility verse the loading level

8.3 The stiffness of the rectangular R/C section, and the ductility

The relations of the stiffness factor in this section are applied for the reinforced concrete sections using the most explicit relations and limitations of the reinforced concrete sections, the rotation, curvature and ductility.

The behaviour of the reinforced concrete section at its yield and post yield are build with the curvature and the ductility for different reinforcement ratio. The significant influence of the reinforcement ratio is built in curve according to the loading levels. This relation is extended to the behaviour of the members beyond the crack section to define the general relations of the stiffness.

The effective stiffness, ductility, material property for a rectangular R/C section are given as follows:

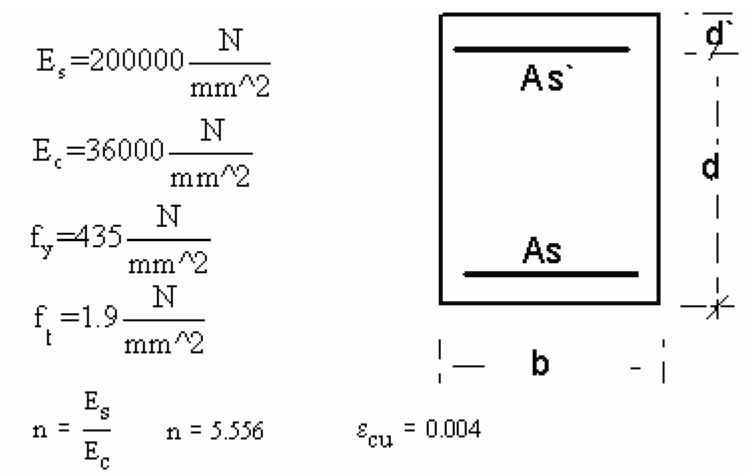


Figure 8-4: B55, R/C cross-section

The section dimension is the following: b = 0.5m, d = 0.6 m and d`=0.05 m,

$\beta_1 = 0.85 - 5 \times 0.04$, $\beta_1 = 0.65$ and $f`_c = 35 \dfrac{N}{mm^2}$

The neutral axis depth factor k and the arm Z are calculated as follows:

$$k = \sqrt{(\rho + \rho`)^2 \cdot n^2 + 2n \cdot (\rho + \frac{\rho` \cdot d`}{d})} - n \cdot (\rho + \rho`) \qquad (8.10)$$

$$Z = d - \frac{k \cdot d}{3} \qquad (8.11)$$

The steel area: $A_s = \rho \cdot b \cdot d$ And $A_s` = \rho` \cdot b \cdot d$.

$$M_y = A_s \cdot f_y \cdot Z \quad \text{And} \quad \varphi_y = \frac{\frac{f_y}{E_s}}{d \cdot (1-k)} \qquad (8.12) \text{ and } (8.13)$$

Assume that both the tension and compression reinforcement has reached the yield strength, the compression zone depth `C` and the depth of the compression zone from the top face (a) can be obtained as follows:

$$C = \frac{a}{\beta}$$

$$a = \frac{(A_s - A_s`) \cdot f_y}{0.85 \cdot f`_c \cdot b} \qquad (8.14)$$

The ultimate moment and the ultimate curvature will be as follows:

$$M_u = 0.85 \cdot f_c` \cdot a \cdot b \cdot (d-a) + A_s \cdot f_y \cdot (d-d`) \quad \text{And} \quad \phi_u = \frac{\varepsilon_{cu}}{C} \qquad (8.15)\ (8.16)$$

The incremental moment and curvature beyond the yield moment up to the ultimate moment are formulated as follows:

$$\Delta M_u = M_u - M_y \quad \text{And} \quad \Delta\phi_u = \phi_u - \phi_y \qquad (8.17)\ (8.18)$$

The yield and the ultimate stiffness are calculated as follows:

$$K_y = \frac{M_y}{\phi_y} \quad \text{And} \quad K_u = \frac{M_u}{\phi_u}$$

Define that $rK = \dfrac{\Delta M_u}{\Delta\phi_u}$ (8.19)

For a given moment M_{max}, the curvature and the effective stiffness may be expressed as follows:

$$\phi_{max} = \phi_y + \frac{M_{max} - M_y}{rK} \qquad (8.20)$$

$$K_{eff} = \frac{M}{\phi_{max} \cdot C_s} \qquad (8.21)$$

Where C_s is a coefficient depends on the structural member fixation. With C = 1.The maximum ductility due to the applied moment M_{max} and the available ductility μ_u due to the ultimate moments are:

$$\mu_{\phi max} = \frac{\phi_{max}}{\phi_y} \qquad \text{And} \quad \mu_u = \frac{\phi_u}{\phi_y} \qquad (8.22)$$

For the crack stiffness calculations the cracking section properties definition is required as follows:

The area and the moment of inertia of the section are calculated as follows:

$$A = b \cdot d \ \text{ and } I = \frac{b \cdot d^3}{12}$$

The gross section area and the section moments are formulated as follows:

$$A_g = A \cdot d \cdot (1 + n \cdot (\rho + \rho`)$$
$$A_{gy} = A \cdot \frac{d}{2} + n \cdot A \cdot (\rho \cdot d` + n \cdot A \cdot (\rho \cdot d` + \rho` \cdot (d-d`)) \qquad (8.23)$$

The gross section inertia is $I_g = (n-1) \cdot A \cdot (\rho \cdot (y_t - d`)^2 + \rho` \cdot (d - y_t - d`)^2) + \dfrac{b \cdot d^3}{12}$ (8.24)

The cracking moments, the cracking curvature, the cracking stiffness and the ductility are calculated as follows:

$$M_{cr} = \frac{f_\tau \cdot I_g}{y_t}\ ,\ \phi_{cr} = \frac{M_{cr}}{E_c \cdot I_g} \qquad (8.25)\ (8.26)$$

$$K_{cr} = \frac{M_{cr}}{\phi_{cr} \cdot C_s} \quad \text{And} \quad \mu_{cr} = \frac{\phi_{cr}}{\phi_y} \qquad (8.27)\ (8.28)$$

8.4 The stiffness and the ductility of the rectangular R/C section

The stiffness and the ductility discussed in paragraph (8.1) are applied for R/C section in this section with different reinforcement ratio and loading levels. A simple computer program is used to solve equation (8.10) through (8.28). As a result it is recognised that the effective stiffness decreases with the increase of the applied moment and increases with the increase of the reinforcement ratio (Figure 8-5) and the demand ductility is increased significantly with the increase of the applied moment from the yield to the ultimate moment (Figure 8-5), it decreases with the reinforcement ration.

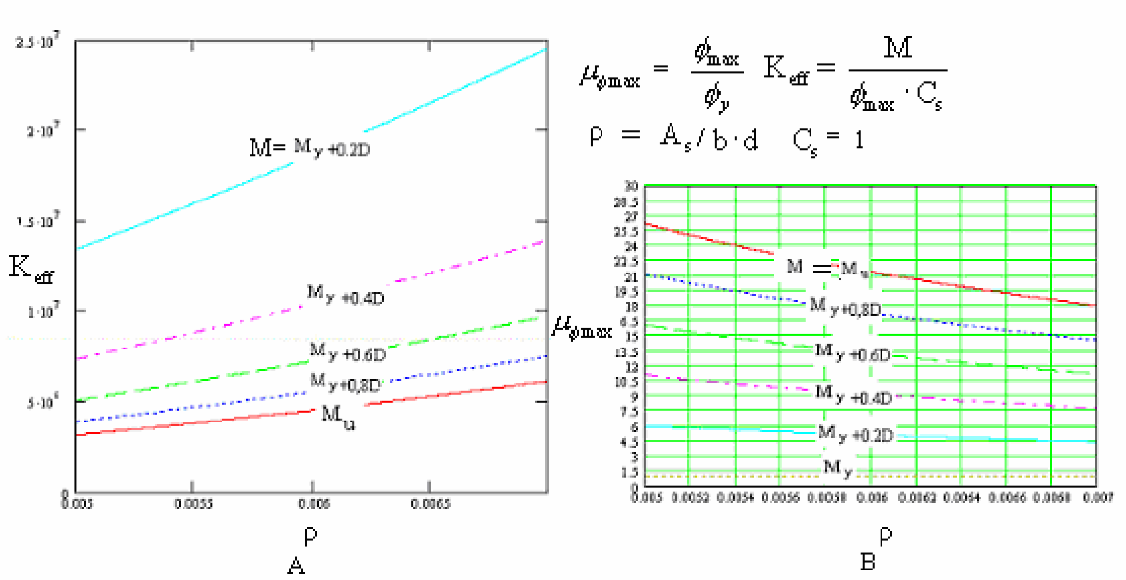


Figure 8-5: The effective stiffness (A), and the demand ductility (B) versus the reinforcement ration

For rectangular B55 R/C

Conclusion

When the load on the connection lays beyond the yield point the stiffness of the connection is reduced and that reduce the load exerted on the structure applied due to the redistribution of the loads on the connection, the ductile joint equilibrium required to design the connection so that the design moment is less than the ultimate moment and more than the yield moment, taking into the considerations the capacity requirements. The effective stiffness will be justified through the choice of the adequate reinforcement ratio, and the ductility demand is lays within the available ultimate ductility. Careful study of the relations given in this chapter leads to the following conclusion:

(1) The moment resistance, the ductility, the stiffness at the beams ends depends on the reinforcement. The continuity of the previous properties of the concrete cross section depends on its extension in the column beam connection. The connection in

this case is considered as the column beam connection and the member ends. As we have seen in the previous sections (6.4) through (6.8) and (8.1) trough (8.4) the stiffness, and the reinforcement is highly dependent on the reinforcement ratio at the column core.

(2) Thus the reinforcement and its continuity in the column core, has significant influence on the effective stiffness of the concrete structure, and then on it's the response.

(3) The stiffness in the beam end is highly dependent on the loading level and especially near the column beam intersection region.

(4) The stiffness, strength at the column beam connection reduces during the time and it may be regards as material softening and deterioration causes rotation at the beam-ends. Rotation occurs or allowed during the erection process of the precast concrete elements. This rotation should be considered in connecting members' behaviour and its stiffness.

(5) Premier estimation of the effective stiffness can be carried out for certain member, which needs enough information for its cross section at its ends and along its length. As the dimensions the reinforcement ratio. The material properties; the elasticity, strength. The information may be given in two ways as follows:

(a) Using the analysis methods as it discussed in section (8.2) through (8.4), using the relations between the reinforcement ratio, the ductility, the loading level and the stiffness (section (8-3), and (8.5)).

(b) Using experimental results of the R/C productions, data test available or can be built for the desired r/c elements, for sections of various dimension and reinforcement ratio. The information concern the stiffness may be given as a function for the loading level and the ductility. It takes in the considerations the detail of the connections, and the properties of the ductile or the connecting members used.

(6) The estimation of the effective stiffness may be carried out with the following steps:

(a) Chose reinforcement ratio level for the external, and internal connections for the structure. That may be obtaining considering the structural analyses result for the moment and shear at the connection region using the elastic method and considering certain distribution factor for the forces at the connection. Code of practice enables the premier dimensioning and the choice for distribution factor and the reinforcement ratio.

(b) Estimate the local ductility of the members. That can be done through the relations of the global ductility and the internal the structure geometry relations, Using structural program or geometrical relations, considering that the global ductility provides general description for the non-linear behaviour of the structure.

(c) The effective stiffness can be estimated from the standard moment curvature diagrams of the members. Using the loading level find the demand ductility for that load (Figure 8-5), then use this ductility value and find the effective stiffness.

(d) Apply the structural analysis, after the substitution of the obtained effective stiffness in the model and examine that the demand ductility of the member lies within the available ductility. The demand ductility is a function of the structure geometry, the degree of the member fixation, and the loading. While the available ductility of the member is a function of the member properties; the dimensions, and the reinforcement ratio. The application of the effective stiffness required trial and prior estimation for the structural elements (Figure 8-13).

(e) In the As it is recognized in connections, which uses ductile members the stiffness depends on the unbounded length of the ductile member, by which the stiffness can be justify and control. That may be used for the DDC connections, which used unbounded lengths at the ductile members that enable control the rotation at the section according the applied loading.

8.5 The stiffness, the ductility, of a model with simple fixed end beams

The stiffness of a simple R/C beam loaded with a uniform load, and it is connected to fixed end supports model is built in this section. The fixed end moment and loading can be modified to allow the representation of simple beams with different dynamic loadings. The beam allows different value of loading in the middle region part in order to represent a precast beam unit with different effective loadings. It may represent the real behaviour by changing the end moment's fixation degree. The model reflects the behaviour of the concrete sections according to the loading levels and the concrete material properties. Takizawa (1973) developed a model that assumed a prescribed distribution pattern of the cross-sectional flexural flexibility along the member length, instead of dividing a member into short segments. The model recognized the influence of the stress in the cross sections .Our model properties (Figure 8-6) are given as follows:

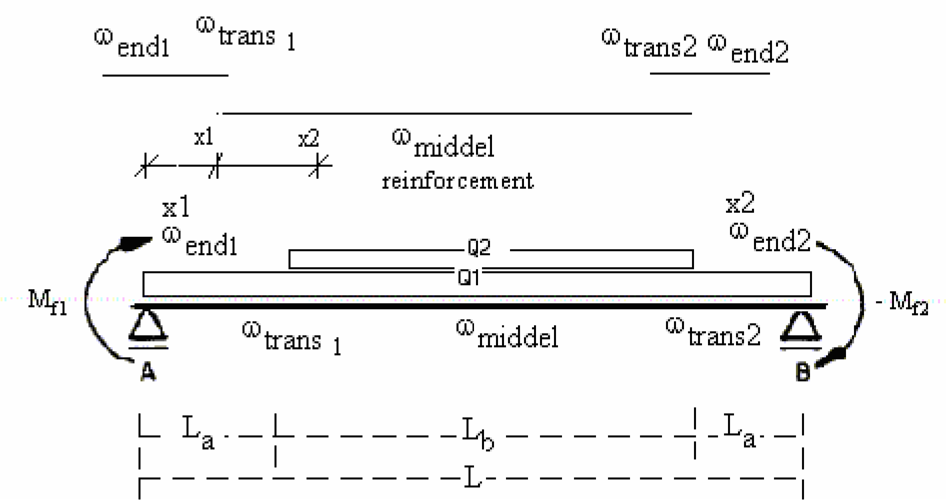


Figure 8-6: Simple precast concrete beam subjected to fixed end moment and uniformly distributed load

- The model represents the actual loading that obtained from the structural analyses.

- The model represents the actual fixation moments and the connection with the total structure.
- The model represents the connection place and the degree of fixation of the precast unit.
- The model describes the location and connection properties of the precast unit and it allow for the choice of the preferable situation.
- The dynamic loading effect is included and its high influence is recognized especially in the critical connection region.
- The material properties of the reinforced concrete are included, and that is for concrete and for the reinforcement, as strength, stiffness and reinforcement ratio.
- The model can describe the influence of the reinforcement ratio and it's variation along the element, and that allows for the choice of the preferable yielding location and describes explicit the variation of the flexibility according to the dynamic loading.
- The stiffness along the member is divided into three categories. First the stiffness beyond the yield moment, second the stiffness beyond the cracked moment, and third the stiffness in the non-cracked section.
- The model can be used for the capacity design for the choice of the yielding locations and the description of the stiffness along the member
- The model is applicable for the representation of the behaviour of the concrete element under dynamic loadings, in which the material properties and the dynamic loading are considered.

In order to build this model, we should define the beam reinforcement. The distribution of the flexure reinforcement is defined along the beam, considering the top reinforcement in both ends and the bottom reinforcement in the middle of the beam as follows:

Consider that the reinforcement ratio at the end is ω_{end} works up to the distance x_1, the reinforcement ratio at the middle of the beam is ω_{middle} works from the distance x_2. The reinforcement between the ends and the middle of the beam is ω_{trans}. The reinforcement ratio works at between the ends and the middle area, the transmission, varies linearly according to the distance x from the supports, the reinforcement ratio may be expressed as a function of it's distance from the support x as .The distance x_1 and x_2 is the distance of the beginning and end of the steel reinforcement work in the transmission zone from the support, these distances to be considered by the designer depends on the bars diameters and the encrage length.

In case that the bending moment $M_{f2}>0$, the reinforcement ratio is formulated as follows:

$$\omega 1(x) = \begin{vmatrix} \dfrac{\omega_{end}(x2 - x1) + \left(-\omega_{end} + \omega_{trans}\right)(x - x1)}{x2 - x1} & \text{if } (x \geq x1) \text{ and } (x \leq x2) \\ \omega_{middel} \text{ if } [x \geq x2 \wedge x \leq (L - x2)] & \\ \dfrac{\omega_{end}(x2 - x1) + \left(-\omega_{end} + \omega_{trans}\right)(L - x - x1)}{x2 - x1} & \text{if } (x \leq L - x1) \text{ and } (x \geq L - x2) \\ \omega_{end} \text{ if } (x \leq x1) \vee (x \geq L - x1) & \\ \omega_{trans} \text{ otherwise} & \end{vmatrix} \quad (8.29)$$

And in case that M_{f2} □0, then, the reinforcement ratio is formulated as follows:

$$\omega 2(x) = \begin{vmatrix} \dfrac{\omega_{end}(x2 - x1) + \left(-\omega_{end} + \omega_{trans}\right)(x - x1)}{x2 - x1} & \text{if } (x \geq x1) \wedge (x \leq x2) \\ \omega_{middel} \text{ if } [x \geq x2 \wedge x \leq (L - x2)] & \\ \dfrac{\omega_{end2}(x2 - x1) + \left(-\omega_{end2} + \omega_{trans2}\right)(L - x - x1)}{x2 - x1} & \text{if } (x \leq L - x1) \wedge (x \geq L - x2) \\ \omega_{end} \text{ if } (x \leq x1) & \\ \omega_{end2} \text{ if } (x \geq L - x1) & \end{vmatrix} \quad (8.30)$$

The reinforcement ratio along the beam is defined as follows:

$$\omega(x) = \begin{Vmatrix} \omega 1(x) \text{ if } M_{f2} \geq 0 \\ \omega 2(x) \text{ otherwise} \end{Vmatrix} \quad (8.31)$$

The influence of the fixed end moments on the member is calculated as follows:

$$M_f(x) = \left(1 - \frac{x}{L}\right) \cdot M_{f1} + \frac{x}{L} \cdot M_{f2} \quad (8.32)$$

The bending moment due to the dead and the live load can be described as follows:

The reaction at point A is calculated as follows:

$$R_a = \frac{Q2 \cdot (L - 2L_a)}{2} \tag{8.33}$$

The moment at the region near the support is calculated as follows:

$$M_a(x) = R_a \cdot x \tag{8.34}$$

The moment at the middle region is calculated as follows:

$$M_b(x) = R_a \cdot x - \frac{Q2 \cdot (x - L_a)^2}{2} \tag{8.35}$$

The moment at the region near the other end support is calculated as follows:

$$M_c(x) = R_a \cdot (L - x) \tag{8.36}$$

The free static moment of the load on the middle beam is formulated as follows:

$$M_d(x) = \frac{-Q2 \cdot L_b}{2} \cdot \left[(x - L_a) - \left[\frac{(x - L_a)^2}{L_b} \right] \right] \tag{8.37}$$

The bending moment due to the middle load is calculated as follows:

$$M_{b2}(x) = \begin{cases} M_a(x) & \text{if } x \le L_a \\ M_b(x) + M_d(x) & \text{if } (x > L_a) \wedge [x < (L - L_a)] \\ M_c(x) & \text{otherwise} \end{cases} \tag{8.38}$$

The total bending moment due to the uniform load is calculated as follws:

$$M_b(x) = M_{b1}(x) + M_{b2}(x) \tag{8.39}$$

The bending moment along the beam is calculated as a function of the distance x from the support as follows:

$$M(x) = M_b(x) - M_f(x) \tag{8.40}$$

The bending moment and the reinforcement ratio of the beam for a beam with 6 m length are shown in Figure 8-7, the effective stiffness that is stated by the equation are applied for this beam. The results of the effective stiffness along the beam are shown in Figure 8-8, where a significant reduction of the effective stiffness at the support due to the applied fixed end moment. In this model the cracked stiffness was not considered.

Figure 8-7: Bending moment diagram and reinforcement ratio

Figure 8-8: The effective stiffness along the precast R/C beam

Using the similar method used in section (8.2), which covers the relations between the yield and the ultimate moment. And apply it for the cracking and the yield moment. The slope of the line between the cracked moment M_{cr} and the yield moment M_y is $r \times K_{cr}$ (Figure 8-9).

The effective stiffness is built as a function of the distance x from the support in equation (8.41) through equation (8.46). By this concept the properties of the member can be defined due to the cross section area, the reinforcement ratio, the end fixation, and the loading taking into the considerations the un cracked, the cracked, and post yield situation .The beam can be divided in three regions, the end fixation of post yield section behaviour with high loading level, which depends on the degree of fixation and dynamic loading, the transition zone with un cracked section and low loading level varies from positive to negative moment, and the middle zone with cracked section and positive moment level .The beam as it is idealized may behave as rectangular section, while in the reality at the middle region the contribution of the floor can't be avoided. As the stresses at the middle span of the beams increased the compression area of the concrete extended for and the width of the compression concrete increases. And that leads to more stable stiffness at the middle of the beam than in the supports, rather than it will be increased due to more contribution of the floor in case of increase in the positive moment. The stiffness in the support will be reduced due to the increase in the loading level.

Thus the idealization of the structure using general factor multiplied by the inertia forces is farther to describe the behaviour of the structural members, and for that a local definition of the stiffness is needed for better approach for the non-linear behaviour of the structure. From this point of view is the advantage of our beam model and the structural analysis method used the study with redistribution of the loading by using semi rigid connections model and a spring constant (Figure 8-9).

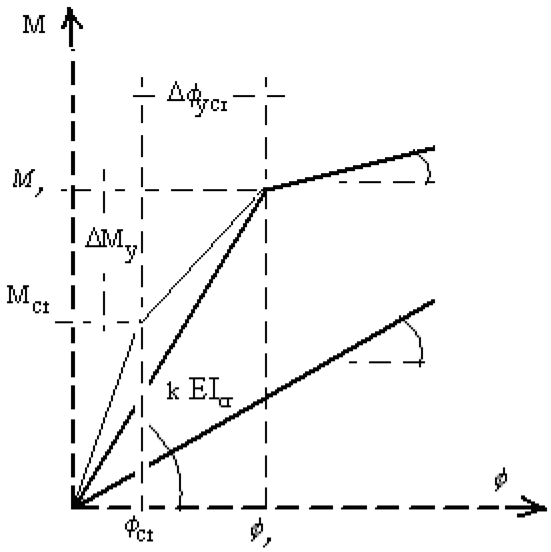


Figure 8-9: Typical beam moment curvature response detailed for the crack

Formulation of the moment curvature at the cracked region

The increment of the moment up to the yield moment may be calculated as follows:

$$\Delta M_y = M_y - M_{cr} \tag{8.41}$$

The increment in the curvature beyond the crack moment up to the yield moment is calculated as follows:

$$\Delta\phi_{ycr} = \phi_y - \phi_{cr}$$

The slope line between M_y and M_{cr} is calculated as follows:

$$rK_{cr}(x) = \frac{\Delta M_y(x)}{\Delta\phi_{ycr}(x)} \tag{8.42}$$

The incremental moment beyond the crack is calculated as follows:

$$\Delta M_{cm}(x) = |M(x)| - M_{cr}(x) \tag{8.43}$$

The incremental curvature beyond the crack is calculated as follows:

$$\Delta\phi_{crm}(x) = \frac{\Delta M_{cm}(x)}{rK_{cr}(x)} \tag{8.44}$$

For a given moment the curvature along the beam may be formulated as follows:

$$\begin{aligned} &\phi_{c\max}(x) = \phi_{\max}(x) \quad \text{For } M \geq M_y \text{ and } M \leq M_u \\ &\phi_{c\max}(x) = \phi_{cr}(x) + \left|\Delta\phi_{crm}(x)\right| \quad \text{For } M \geq M_{cr} \text{ and } M \text{ p } M_y \end{aligned} \tag{8.45}$$

Summary:

The general effective stiffness is applicable for the beam, includes the cracked the untracked and the post yield stiffness can be formulated as follows:

$$\begin{aligned} &k_{geff}(x) = K_{eff} \quad \text{For } M \geq M_y \\ &k_{geff}(x) = \frac{M(x)}{\phi_{crmax} \cdot C_s} \quad \text{For } M \leq M_y \text{ and } M \geq M_{cr} \\ &k_{geff}(x) = \kappa_{cr} \end{aligned} \tag{8.46}$$

The application of the above equation in a 6m span beam (Figure 8-10) in a structure shows that the moment distribution at the beams by semi rigid connection in comparison with rigid connection; results in les negative moments at the supports and more positive moments at the middle of the beam members (Figure 8-10). It may be concluded that the behaviour of the beams using rigid connection and semi rigid connection can be described as follows:

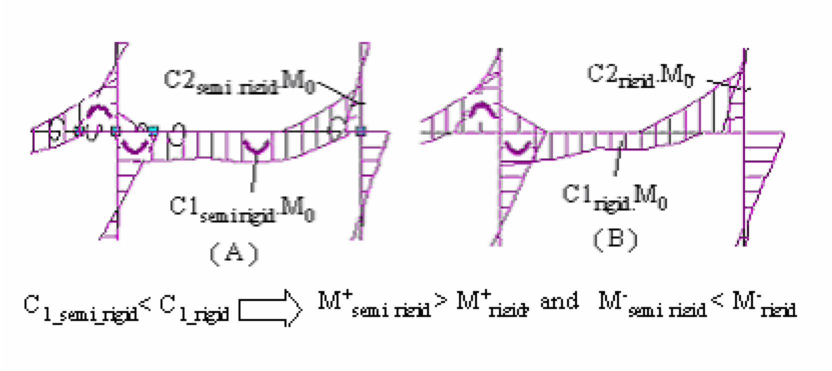


Figure 8-10: B.M. diagram at a beam in a structure with rigid and with semi rigid end fixation

(1) Beam with semi rigid connection:

The static moment due to DL + LL = M_0

The fixed end and at the middle of the span may be formulated as follows:

$C_{1\_semirigid} \times M_0$, $C2\_{semirigid} \times M0$.

(2) Beam with rigid connection:

The static moment due to DL + LL = M_0.

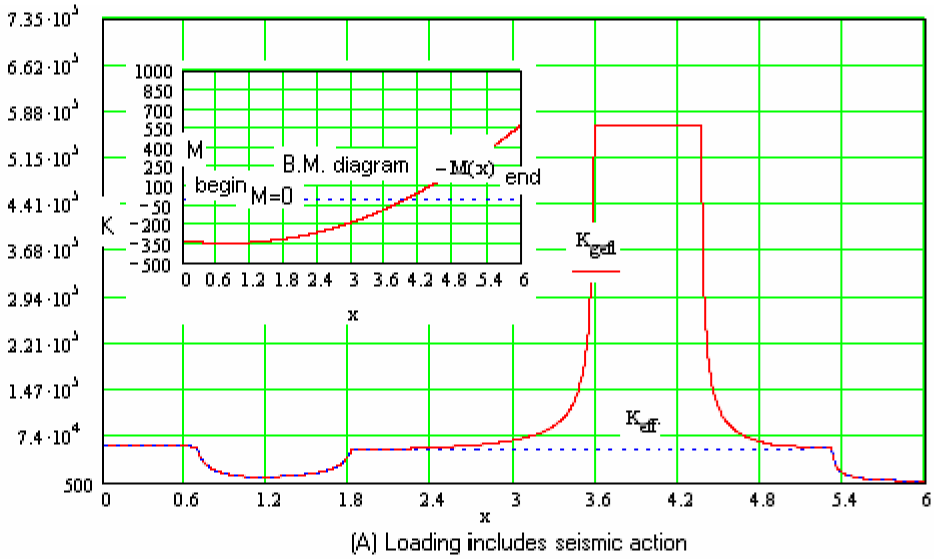
The fixed end moment and at the middle of the span may be formulated as follows:

$C_{1\_rigid} \times M_0$, $C2_{\_rigid} \times M_0$

Due to the flexibility in the semi rigid connection the following conclusion may be obtained:

$$C_{1\_semi\ rigid} < C_{1\_rigid} \quad \Longrightarrow \quad M^{+}_{semi\ rigid} > M^{+}_{rigid} \text{ and } M^{-}_{semi\ rigid} < M^{-}_{rigid} \tag{8.47}$$

Seismic load exerted on the connection is distributed on the members according to the stiffness proportion of the connecting members. As the load increase, the beam stiffness decrease more than that in the column where the beams are in the yield situation and that leads to the decrease of the seismic bending moment due to the decrease of the beams stiffness proportion in the connection. As the seismic load increase the end beam stiffness reduced and that leads to increase the positive bending moment in the beam middle span and decrease the negative bending moment in the beams end due to the DL+LL according to equation (8.47) Figure 8-11. Then introduce this member into a structure leads to semi rigid structural model (Figure 8-12). This model is closer to the real behaviour of the structure under seismic loading. The reduction of the rigidity under dynamic loading in the beam-ends and the degradation in the yielding locations leads to redistribution of the loading on the structural members. The model with semi rigid connection is used in this study as a link approach to the non-linear behaviour of the structure. It is applied as an elastic model with modified elastic stiffness connections. These modified stiffness used at the connections are obtained for the different R/C cross-section relations as a function of the reinforcement ratio and the loading level (The bending moment- at the connecting beam end).



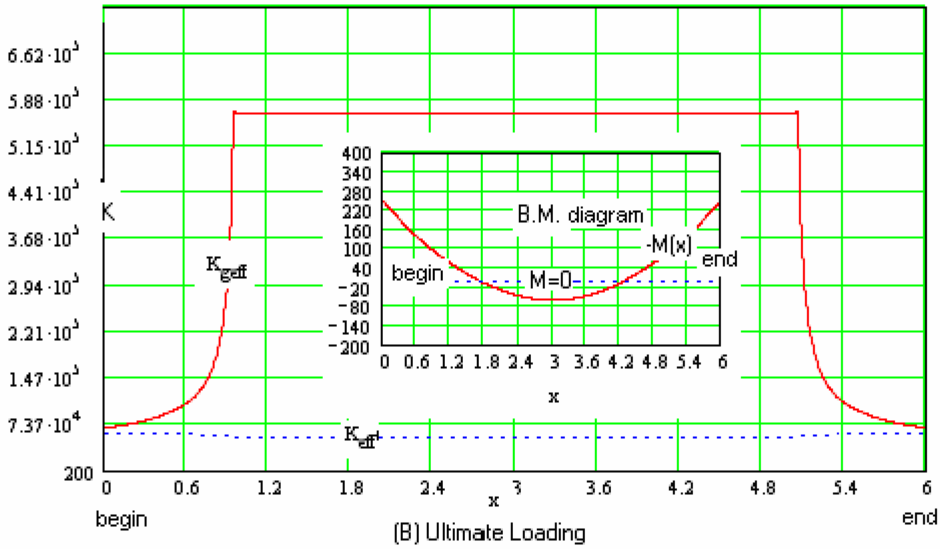


Figure 8-11:The effective stiffness of the reinforced concrete beam, with seismic action at A and ultimate loading at B

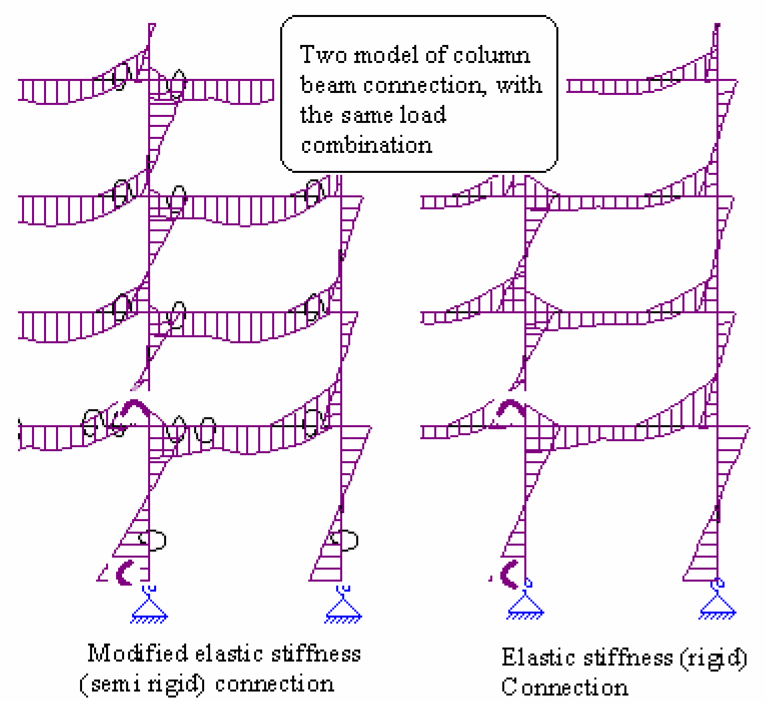


Figure 8-12: Loading includes seismic action, with semi rigid and rigid connection analysis model

8.6 The connection stiffness

The stiffness of the connections in the precast constructions is decided by many factors. The factors cover the concrete and the steel material properties, the section dimensions, and the member geometrical relations. The use of the ductile bars in the connections enables designing for stiffness in the connection: Determine the development bar length, the unbounded bar length, describe the yield length, and the angel of rotation. The use of the prestressed enables the reversal resistance at the connection. In the knee connections and the outside columns beams connection. The detail of the reinforcements influences the connection stiffness. The attention should be made to prevent sharp changes in the stiffness, by recognizing the moment curvature diagrams. And that is to prevent brittle behaviour at the connection.

In our model (Figure 8-11), the emulated monolithic connections may take use of the monolithic structures model, for the precast construction with the following considerations:

- The degree of fixation of the precast units, and the percentage of the initial permanent load of the members.
- The properties and the dimensions of the used installation elements

In the structural analysis the solution should be carried out for the elastic and the modified stiffness model as follows:

(1) The structural analysis for the building with elastic stiffness connections, using the linear analysis. The loadings are the dead, live, and wind load, and the

combinations should full fill the serviceability and the ultimate limit stat as it is shown in Figure 12-3

(2) The structural analyses with modified stiffness connection, the loading are, the dead, live, and seismic loads. The combinations may be carried out according to the EC8 or other regulations. In the decision of the degree of fixation at the mode; the stiffness of the connection, the attention should be made for the following considerations:

- Choice of mechanism so that the yield locations should be mobilized. Models from another analysis and from pushover analysis can be used to help in this choice.
- The degree of rotation of the members should be tolerated to permit the initial loading of the members
- The design of the element used the ultimate limit state for the member's resistance, and the results of the seismic combinations. Then the absolute value of the moment at the end fixation of the beams, and at it' span in (Figure 12-4) should be grater that that which are obtained from the analysis with rigid connections in (Figure 12-3).
- The resultant applied moment at the end fixations should lays between the yield, and the ultimate moments of the members. The premier decision may be taken considering reinforcement ration for the members.
- The choice of local ductility level and the stiffness of the members can be obtained using (Figure 8-13). These curves may be built in structural analysis program .The local ductility may be obtained through the global ductility of the structure.

For a given member and certain ductility the applied load will be decided, using the relations between the curvature, the applied load and the demand ductility in equation (8.1) to (8.9). The stiffness factor can be obtained using the relations between the applied load and the stiffness and as it is shown in Figure 8.12. The effective stiffness is a function of the reinforcement ratio and the applied load (Figure 8.5).

The effective stiffness a reinforced concrete (B55) member is given in Figure 8-13 through Figure 8-14. The effective stiffness in these figures are a function of the R/c sections dimensions, the reinforcement ratio, and they are independent of the location x from the supports.

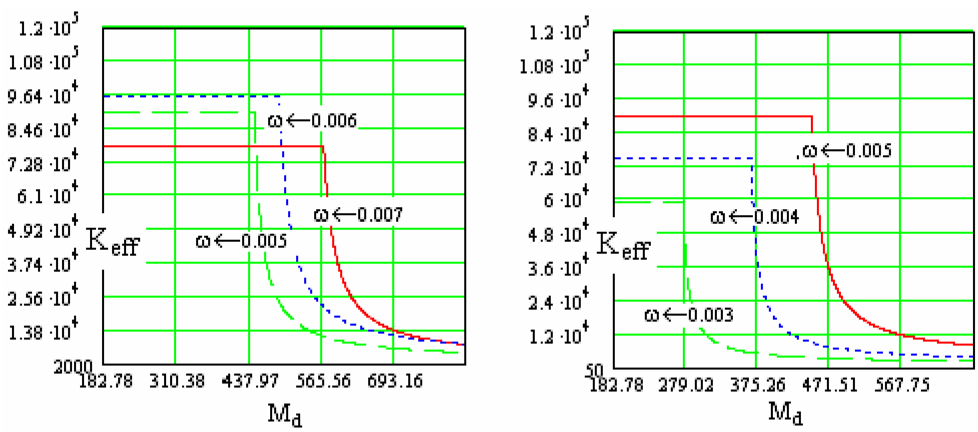


Figure 8-13: Stiffness versus the applied moment for B55 R/C section with h = 700mm, b =500 mm

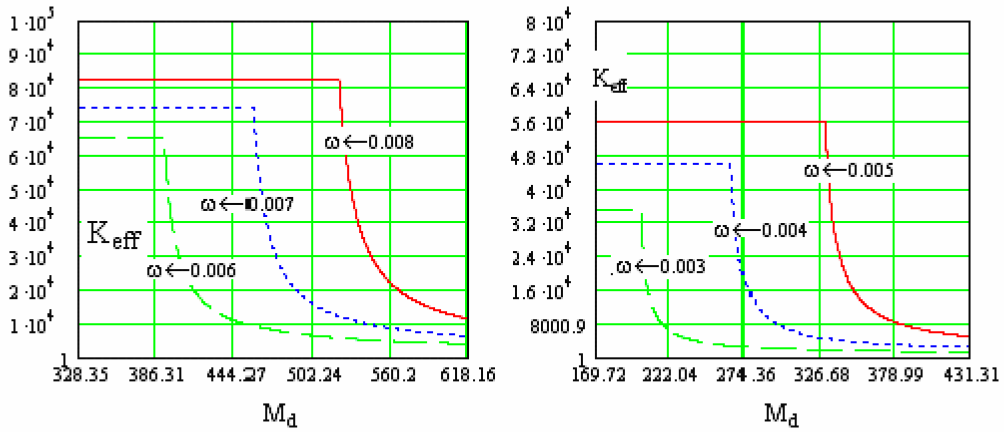


Figure 8-14: Stiffness versus the applied moment for R/C section with h= 600mm, b=500 mm

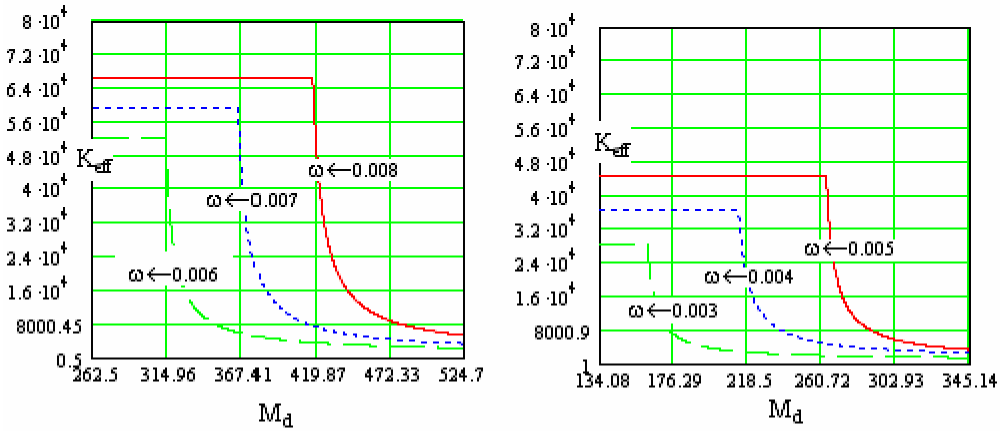


Figure 8-15: Stiffness versus the applied moment for R/C section of h = 600mm, b = 400 mm

The above figures are built for beam length of L = 6m and its value Keff = $EI_{eff.}$ In order to obtain the stiffness in a member, use the stiffness moment figures to find the plastic angle with the following relation θp = (φu-φy) /Lp. Where φu and φy are the values obtained from the diagrams for the ultimate and the yield moment. Use the above curves to find φu and φy and apply the member effective stiffness. As it is seen the effective stiffness may be verified in a member using different plastic lengths, and by cut the reinforcement in proportion at the yield locations.

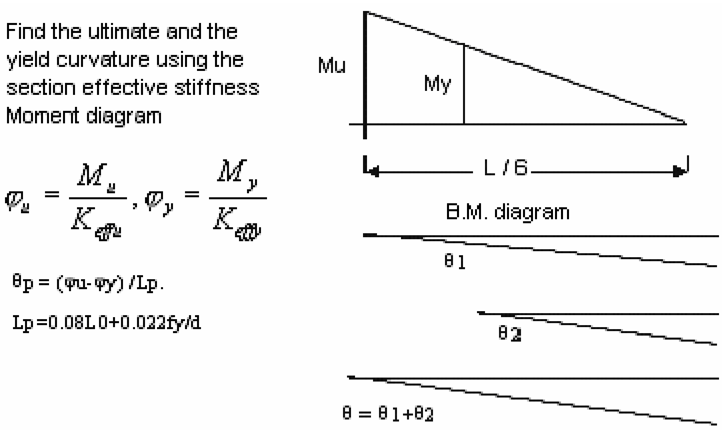


Figure 8-16: Bending moment diagram in a member end

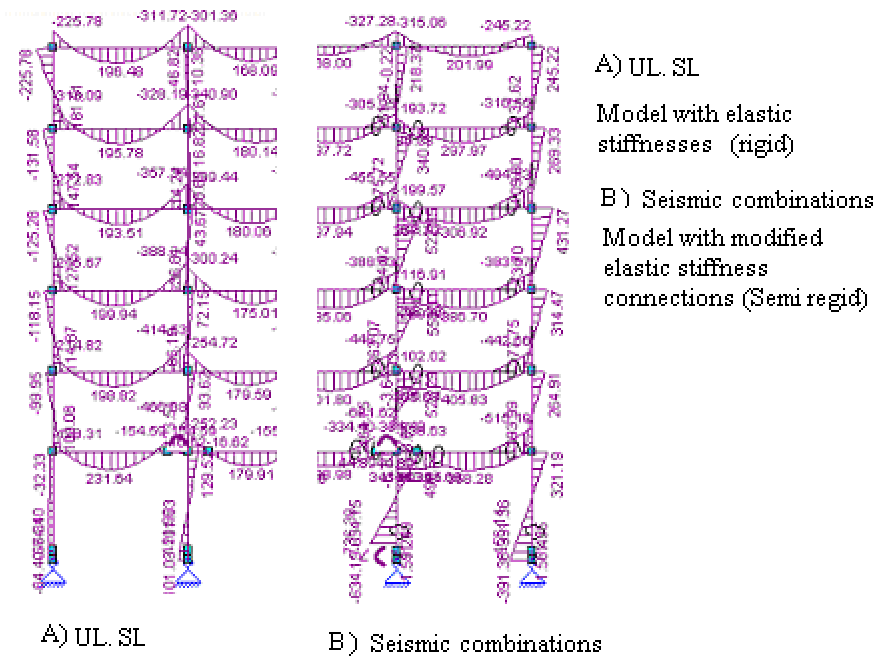


Figure 8-17: Bending moment diagram in a structure building, (A) with serviceability and ultimate limit state, and (B) with Seismic loading combinations.

8.7 Modified effective stiffness structural model

Due to the flexibility in the semi rigid connection the bending moment at the end member fixation reduces and increase the positive moment at the span as follows:

$$C_{1\_semi\_rigid} < C_{1\_rigid} \quad \Longrightarrow \quad M^{+}_{semi\ rigid} > M^{+}_{rigid} \text{ and } M^{-}_{semi\ rigid} < M^{-}_{rigid} \tag{8.48}$$

The bending moments due to the seismic loading exerted on the connection are distributed over the connected members according to their rigidity thus their stiffness proportion. As the load increases the stiffness of the beams decreases more than that in the columns where the beams are in the yield situation and that leads to decrease the bending moment of the seismic force due to the decrease of the beams stiffness proportion in the connection. As the seismic load increase the end beam stiffness reduced and that leads to the increase of the positive moment at the middle span and decrease of the negative moment at the beams end due to the applied dead, live and the seismic load, DL+LL+S (equation(8.48) and Figure 8-18). Introducing this member into a structure leads to semi rigid structural model. This model is closer to the real behaviour of the reinforced concrete structure under seismic loading. The increase of the bending moment at the beam-ends decreases its stiffness (Figure 8-19). As the loading level increased on a certain reinforced concrete section the effective stiffness decreases. When apply that on a beam the effective stiffness at the beam ends decreases and especially under the seismic loading due the yield at the top beams ends, and especially in the precast concrete constructions, where the floor is constructed with partially rigid. This influence is limited at the middle span where the top compression zone width increases as the load increase.

The reduction of the rigidity under dynamic loading at the beam-ends, and the degradation at the yielding locations leads to the decrease of the moments in the member ends and the redistribution of the loading in the structural members. The model with semi rigid connection is used in this study .as a link approach to the non-linear behaviour of the structure. It is applied as elastic model with modified elastic stiffness connections. The modified stiffness is obtained for the different R/C cross-section as a function of the reinforcement ratio and the bending moment at the connecting beam ends (Figure 8-19).

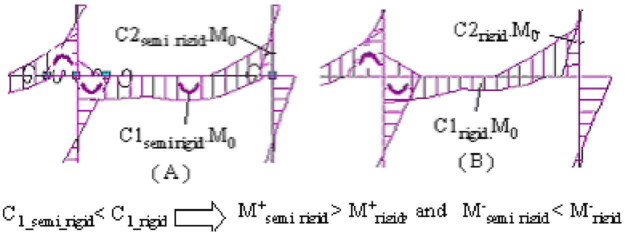


Figure 8-18: B.M. diagram at a beam in a structure with rigid and semi rigid end fixation

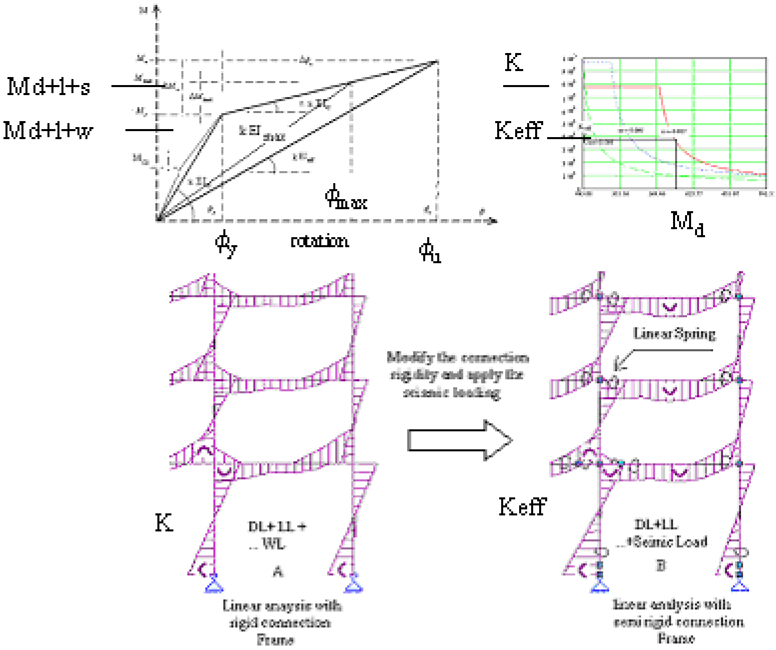


Figure 8-19: linear analyses with rigid connection frame and modified frame with semi rigid connection. As the B.M. on the R/C section increase decrease its effective stiffness. That is considerable at the member ends, and limited at the span.

8.8 The modified elastic stiffness and the seismic performance

Members incorporated with precast concrete elements and subjected to gravity load shall satisfy the following two criteria

- Demand performance for serviceability limit state subjected to gravity load.
- Demand performance for ultimate limit state subjected to gravity load.

The member incorporating precast concrete elements subjected to seismic load should satisfy in addition the followings performance requirements:

(1) Demand performance for ultimate limit state subjected to seismic load.
(2) The entire member incorporating precast concrete elements should satisfy the following criteria, Demand performance for durability and fire resistance and Redundancy.

The equivalence in performance may be verified on rational basis using mathematical models with ensured reliability and investigation including correlation studies between tests and analyses. In other case, newly developed connection detail may be evaluated by test under simulated earthquake load of specimen. Such as building model with different types of connections subjected to simulated earthquake loading.

When the load on the connection lays beyond the yield point the stiffness of the connection is reduced and that reduces the load exerted on the structure applied due to the redistribution of the loads on the connection, the ductile joint equilibrium required to design the connection so that the design moment is less than the ultimate moment and more than the yield moment, taking into the consideration the capacity requirements. The effective stiffness will be justified through the choice of the adequate reinforcement ratio, so that the ductility demand lies within the available ductility of the member. We may conclude that:

(1) The reinforcement in the member's end -including the connections- has significant influence on the effective stiffness of the concrete structure, and then on its response.
(2) The effect is high in the region of the connections while it depends on the loading level.
(3) First estimation of the effective stiffness can be carried out for certain R/C section that have enough information, concerns the moment curvature behaviour, ductility and strength as follows:

- Chose reinforcement ratio level for the connections; premier reinforcement design for the applied moment.
- Estimate the local ductility rotational demand of the members. That can be done for the structure through the global ductility and the internal the structure geometry relations.
- The effective stiffness (K_{eff}) of the member ends can be estimated using standard moment curvature diagrams.
- Apply the structural analysis, and examine that the demand ductility of the member lies within the available ductility of the member. The demand ductility of

the member is obtained from the structural analysis, while the available ductility is a function of the reinforced concrete section property.

- When using ductile members, the unbounded length on the ductile bars may be used for the control the elongation of the bars in the unbounded places, which cause rotation and influences the stiffness of the member. Thus in a controlled design the stiffness at the connection can be justified and verified.

8.9 Analysis with modified elastic stiffness model

The analysis of a structure for seismic loading may be carried out using linear program. This analysis used the modified elastic stiffness, in a model describes the characteristics and the behavior of its members, such as the material properties, dimensions, degree of fixation at the connections, continuity and discontinuity of its members. This approach may be built in two main models (Figure 8-19), the elastic model, and the modified stiffness model.

8.9.1 Monolithic construction with modified elastic stiffness model

Modified elastic stiffness model may be applied on monolithic constructions; in this case the attention should be made for the sharing of the transverse beams, its torsions effect on the columns and the successive yielding of the concrete floor surrounding the connections. When the yield locations lays at a considerable distances from the connection faces the floor reinforcement will yield, other than the reinforcement belonging to the beams. This reinforcement may be taken in account for the stiffness calculations. In precast concrete constructions the influence of the transverse beam is limited where the precast concrete elements do not function in flexure bending strength with the supporting transverse beams. The use of this method as intervention procedure in the analysis and the design of the monolithic construction required various solution that links between the loading, yielding material and technical solutions. The technical solutions in the monolithic constructions are limited when it is compared with the precast construction; such solution used in the floor yield locations considering technical solutions and details as follows:

- Using special arrangement of the reinforcement in the floor at the connection and yield region, an equivalent steel bundles works as hidden beams outside the yield locations, relocate the yield lines by using unbounded spaces to create tension softening at some suitable places.
- Using special details for the floor connections of the floor with the transverse beams. Some details in the floor used as shear resistance, and limiting moment resistance while the principle steel at the yield situation work as traction force.
- Use moment limiting joints consist of areas of reduced effective depth achieved by the artificial discontinuity of the compression zone. Three main issues need to be considered in the design of elements incorporating pined or limiting joints as follows:

(1) Shear transfer: Detailed reinforcement to transmit shear.

(2) Kinematics: Insure that the shear force does not increase due to kinematical incompatibility resulting from the primary lateral force resisting system.
(3) Fatigue: Some attention to be paid to fatigue because such joints often yield under serviceability limit state.

- Proportional dimensioning of the floor, beams and columns.

8.9.2 Factors influences monolithic seismic design

The application of the modified effective stiffness model on the monolithic reinforced concrete, may consider the influence of three major factors distinguish it from the precast construction:

- Distribution of the loads of the floor diaphragm on the supporting beams. In the precast concrete the load transfer to one direction in the most preferable cases, using precast elements of standard width. The supporting beams and the columns work as frame which take the loading of the diaphragm the DL+LL and the seismic load, while in the monolithic constructions these loads are distributed on the surrounding beams according to the diaphragm dimensions, its degree of fixation on the supporting beams, and the continuity of the floor on the two sides of the beam.
- The diaphragm stresses of the monolithic are different than the precast floor, and accordingly the reinforcements. In most cases in the monolithic concrete constructions the use of the top reinforcement is at the corners of the floor elements, as for the two-way slabs, and that mean the high-stiffened region surrounding the columns. In cases of extreme loading the proposed yield locations at the beams, will be extended to the floor and as the loading increase increased the steel area, which shares the beam to resist the loading. The external surrounding beams bears torsion moments due to the floor action.
- In the precast concrete the dead load of the beams and some times apart of the floor may be considered supported by the simply supported beam. While in the monolithic constructions the total load is used for the design of the end fixation according to its degree of fixation.

Considering the above-mentioned factors we may take into account the following considerations in the application of the monolithic construction.

(1) Distribute the floor load on the beams according to an analysis method, or tables. Considering the dimension of the included floor units. And estimate the floor reinforcement and especially the reinforcement at the columns location.
(2) Define the yield location in the beams; typically they are at a distance of the beam depth d to d/2 from the column face.
(3) For the calculation of the beams effective stiffness take into account the reinforcement of the floor, in the locations where they are subjected to yielding.
(4) The attention should be made that the supposed yield location can be verified; it should be mobilized for the yielding and the demand ductility.

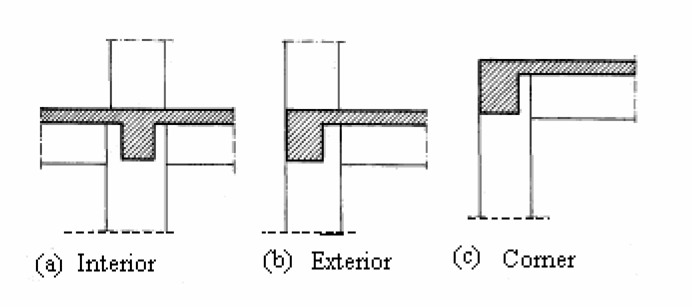


Figure 8-20: Joints in monolithic R/C structures (a) interior, (b) exterior, (c) corner.

8.9.3 The effect of transverse beams and floor slabs

In the forgoing discussion no explicit account has been taken place for the presence of transverse beams in the joint (Figure 8-20). Meinheit and Jirsa found the shear resistance of the core increase. The transverse beams are part of the seismic system and in this case it is quit possible that plastic hinges form at their ends close to the joint face. The presence of a slab monolithic with the rest of the members was found to affect favourably the behaviour of the joint by increasing both its strength and stiffness. According to tests Pauly and Park noted that the contribution of the slab reinforcement is effected by the level of inelasticity induced at the joint, the maximum strength, may be the minimum of the following limits:

- One quarter of the beam span at each side from the beam centreline.
- One half or one quarter of the distance between adjacent beams, at each side of the beam centreline, at interior or exterior columns, respectively.

Durrani and Zerbe, based on results from tests of exterior joint floor slab have suggested effective slab width equal to one depth of transverse beam at each side of the column face. Durrani and Zerbe found that transverse beams were initially effective in confining the joint, but once they reached their (tensional) cracking strength, the effeteness was drastically reduced. In the design of column beam connection for the precast and the monolithic construction, the core is considered confined with the transverse beams. And the reduction due to the tensional cracking may considered.

9 The project of prefabricated building structure

For the study of the precast connections, a precast concrete of 6-storey building is designed with reinforced concrete type B55 (chapter 9). The column beam connections are designed as emulated monolithic R/C B55, Hybrid-fibre concrete HFC, Dywidag ductile connectors DDC, and Post-Tension assemblage. In the analysis of the building structure a modified elastic model is used as an approach for the non-linear behaviour of the reinforced concrete structure. A model for rectangular reinforced concrete in non-linear behaviour is used in the design to control the yield locations at the columns and the beams. The connection design performs the seismic requirements, the different material and connection types. Capacity design control is carried out for the connecting elements according to EC8.The building provides a representative structure to be studied and different seismic performance problematic to be applied. The building is of flexible and permits the choice of different types of column beams connection. The height of the building is enough to investigate the influence of the seismic effects on each level. The precast concrete building consist frames o (3×6m span) beams, at 7.8m centre to centre. The frames support floor of 350mm thick, and 7.8m spans. The structural system concept in principle based on that the precast units should be as long as possible, the short spans (6m) frames supports seismic and service load, while the beams the long beams of (7.8m) span supports partially the service loads and the seismic force. Load and material characteristic of the R/C are considered according to EC8. Hybrid fibre concrete properties are obtained of laboratory test, and the stress strain relations are described in chapter 7. The building location is in zone III in Greece with seismic acceleration A_g =0.24g. In the calculation and distribution of the seismic force on the building the equivalent shear method is used. The seismic and wind loads are distributed on the transverse frames. The floor system serve as rigid diaphragm (18m deep) between the vertical elements of the lateral force resisting system, the columns with equal inertia and lengths undergo the same drift, these frames are considered resist equal seismic forces, the resulted seismic force is reported for one transverse frame.

9.1 Work plan

The work on the project considers main principle activities as follows:

(1) Study and apply analyses procedures in the precast concrete constructions.
(2) Investigate factors affecting the characteristic of the connection, apply and define the regulation for the choice of the different types of the connection.
(3) Collect some standard experimental hysteresis for connections that can be compared with the prefabricated. The project can build in models, which represent the resistance, the rotation capacity, and the ductility.
(4) Study the possibility for using the different types of concrete materials in the connection.
(5) Model typical system connections, use it in structure and verify its response; the model will be built in efficient program as MathCAD to obtain coefficient and explicit calculations. And in Ruaumoko to control the response as follows:

- After changing the degree of fixation, which leads for redistribution of the seismic effects

- Change the fixation position, lump the non-linear members behaviour as flexible joint near the system joint
- The joints can be distinguished in two main categories

(1) The system connection: connection between members and direct influence in the system as the column beams, wall floors-diaphragms-
(2) The connection between the elements as column-column connection, beam-beam connection, floor-floor connections.

The target of the study will be to obtain the response, ductility, and degradation due to the change of the degree of fixation, stiffness, and strength, Verify using different type of connections in the system for obtaining inherent dampers due to the varying of the propagated waves.

And for that the choice of one structure will be more useful. After the collection of the information, the Items in the work plan loops parallel. The building of the models with the different tools as MathCAD, Ruaumoko program is associated with the verification and the choice of the connection types and elements.

9.2 Project: Six story precast concrete building

The project includes the design of 6 stories building in precast concrete. The building forms of 8 transverse frames at 7.8 m each frame consist of three 6 m spans, and of 6 floors at 4.8 m for the first floor and 3.6m other storey floors. The design of the building is given in chapter 9 through chapter 13, a schema for the design process is illustrated in Figure 9-3.

9.2.1 Concepts obtained from the monolith building.

The monolith building provides useful information as follows:

- Mass: Lumped loads for the dynamic analyses of the building,
- Frequencies: The fundamental frequencies of the building in the transverse and longitudinal directions
- The study of the building as precast resistance frame with different possible solutions for the connection types.

9.2.2 Form of the building

The plan of the 6-story building and the vertical transverse cross section are given in Figure 9-1 and Figure 9-2 respectively. The building of length = 7 × 7.8 = 54.6 m, and width = 3×6 = 18m.The interior column are of 700×700 mm, the exterior columns of 600×600 mm. The floor is a one-way hollow core precast unit of 350 mm thick, with 7.8 m span, which is supported on the transverse beams.

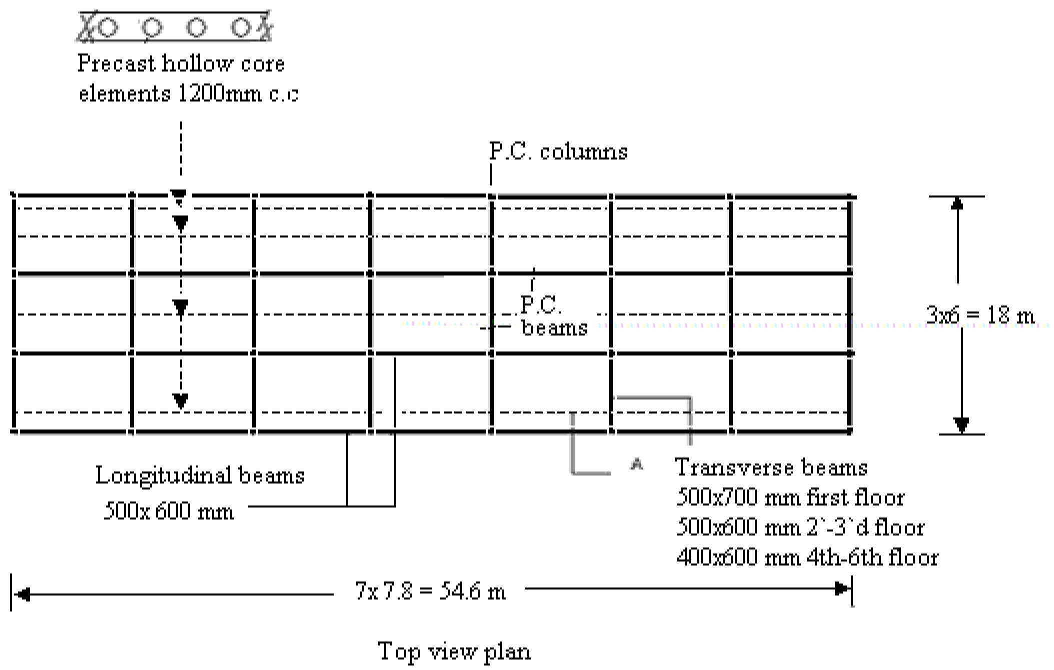


Figure 9-1: Six story-precast concrete building, top view of the current floor

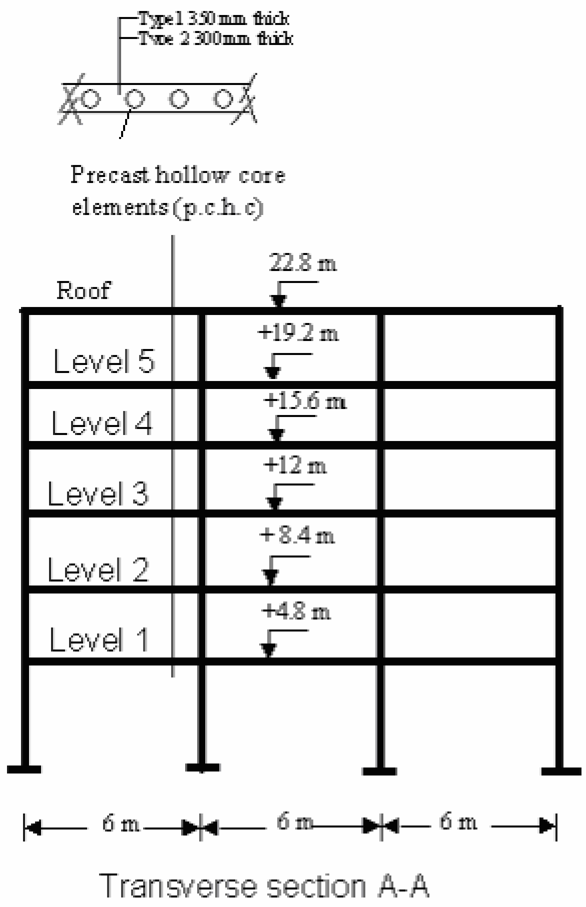


Figure 9-2: Vertical transverse section in the 6 story precast concrete building

Table 9-1 Reinforced concrete structure dimensions

Storey level	Outer columns [m]	Inner columns [m]	Beams [m]	Precast h. c. type
1	0.6×0.6	0.6×0.6	0.5×0.7	1
2	0.5×0.5	0.6×0.6	0.5×0.6	1
3	0.5×0.5	0.5×0.5	0.5×0.6	1
4,5	0.5×0.5	0.5×0.5	0.4×0.6	1
6	0.5×0.5	0.5×0.5	0.4×0.6	2

9.2.3 Requirements

The load and the material properties are reported in Table 9-2 and

Table 9-3

Table 9-2 Superimposed dead load, and live loads on the building wind and seismic load

Locations	Dead load and live load		Seismic and wind load	
	DL[kN.m]	LL[kN/m]	Seismic load	Wind load
Basic		2.4	Considered in subsoil soil class B, zone III in Greece. $A_g = 0.24$ g	$0.8[kN/m^2]$
Allow for corridor		1.2		
Superimposed dead load	1			
Ceiling	0.5			
Total				
Current floor	1.5	3.6		
Roof	0.5	1		

Table 9-3 Material properties [N/mm²].

Concrete	Stress		Elasticity
	f_{ck}	f_c	
B55	55	33	36000
HFC	75		50000
Reinforcement	.$f`_y$		$E`_s$
Fe B 500	435		200000

9.2.4 Design process:

The design of the building has been carried out in chapter (9) through chapter (13). The design process chart is given in Figure 9-3.

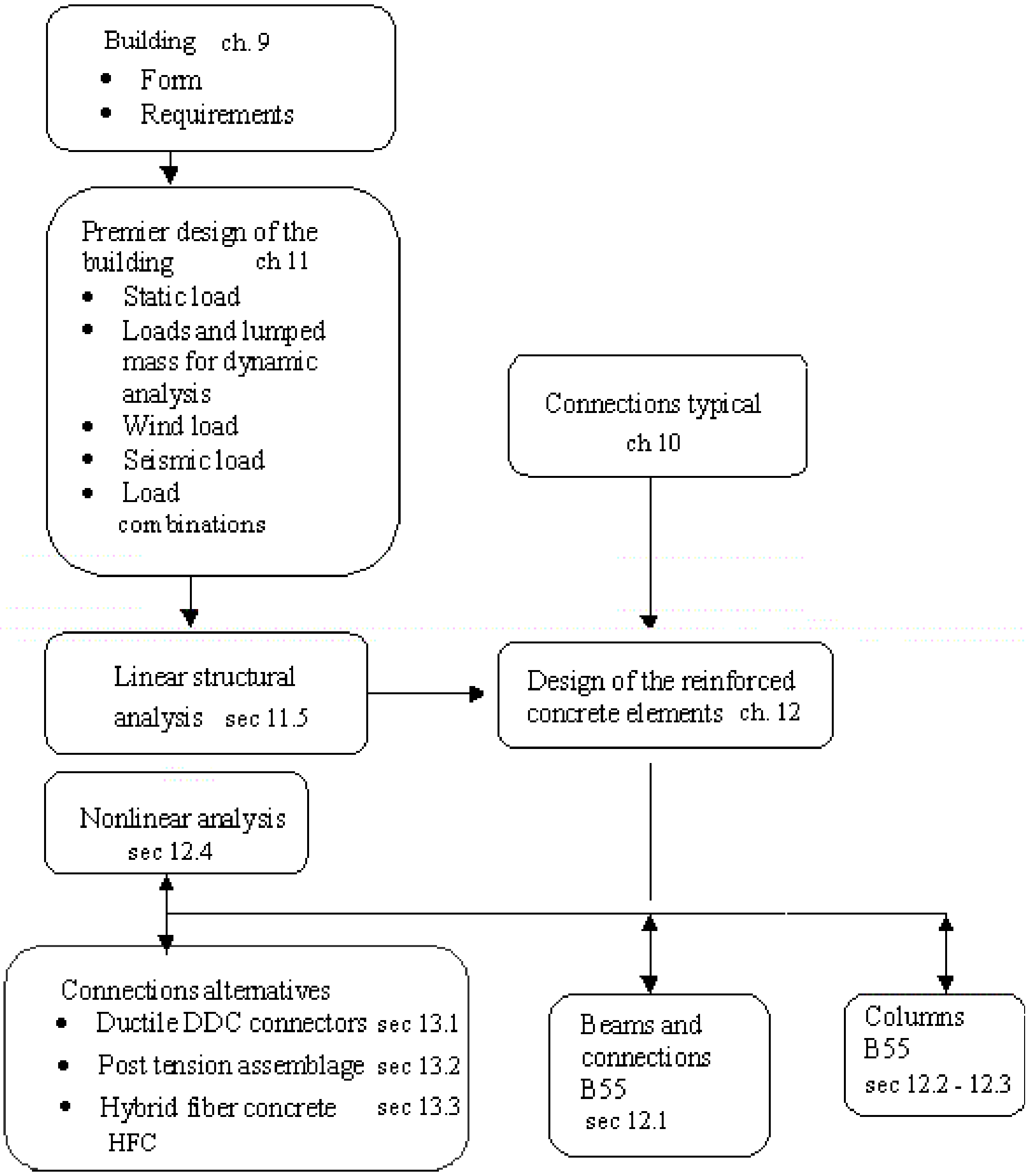


Figure 9-3: The building design process.

10 The connection alternatives of the precast building

Precast concrete elements in general built in equivalent monolithic or jointed system. Special materials, ductile members, and special technique are used in the connection to improve its shear, moment resistance and ductility during the seismic loading. The connection it self as an intersection between at least two elements has its specific problematic to be solved. Solutions are involved in shear, and moment resistance, the integrity with the supports and the ductility. It may be studied in related with the member ends and the connection core. The connections can be divided into system connection, in which factors as moment, shear, dimension to be considered and member connection where the performance for the previous mentioned factors may simplify. Splitting the problem enables finding economical solution, where local solution can be applied as the use of the ductile members is to increase the connection ductility, and the use of the prestressed elements is to solve the shear problem. The analysis of the building structure provides important information for the design of the column beams connections. The negative bending moment in the beams ends extends a distance from the columns faces. That enables the choice in the design for alternatives allowing the usage of stiff connections at the column beams intersection. While the dissipated zone (yield locations) can be extend some distance from the columns. This distance can be estimated as 0.3 m. The reverse moments at these locations ensures the compression of the already tensioned steel in the previous cycle. The dissipating zone can be chosen some distance of 0.6 m from the centre of the columns. For the further discussion the definition the system and member connections is required as follows (Figure 10-1):

System connection: These are the connection of the column and beams. They have direct influence of the structure. The problems to be solved here is the shear, the bending moment. The diagonal shear in the core of the connection is an important problem to be solved.

Member connection: These are the connection between members as column-to-column connection or beam-to-beam connection. The problem to be solved is the shear force.

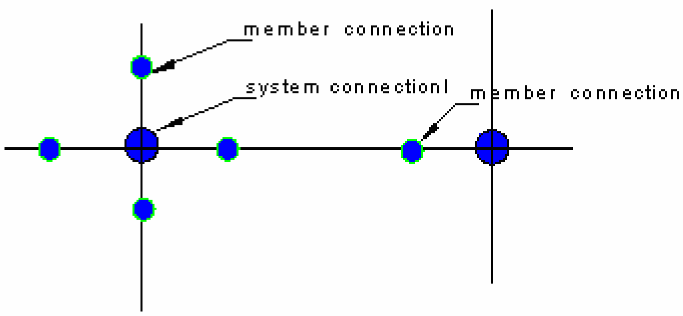


Figure 10-1: System and member connection

10.1 Design Alternative

The design of the seismic connection may be carried out in different alternatives. The connections should perform the seismic requirements as the shear resistance, bending moment, ductility and the rotation capacity. Splitting the functions and the problematic of the members in the connection can help in finding different solutions for the performance according to these requirements. The choice for the yield locations in places outside the column face enables suitable solution for the bar slips and stress penetration in the core of the connection. The choice of the beam connections (The column strip beam and the interiour beam) in a distance from the column, solve the shear problematic in the column face where high moment and shear occurs.

For the realisation of the above-mentioned design considerations there are different alternatives. Each of them has its particular advantage and usage as we have seen in chapter 2. Design alternatives are introduced as follows:

(3) Columns beams connections:

- Emulated monolith R/C connection: The columns and the beams are of B55, using continuous bottom reinforcement in the beams (Figure 10-2).
- Connection type emulated monolith R/C, B55 columns and HFC beams with wet members connection (Figure 10-3)
- Hybrid post tension assemblage or ductile rod Dewadig assemblage (Figure 10-5).

(2) Columns foundation connections

- Column foundation connection type column base plate connection (Figure 10-6)
- Column foundation connection type pocket connection (Figure 10-7).

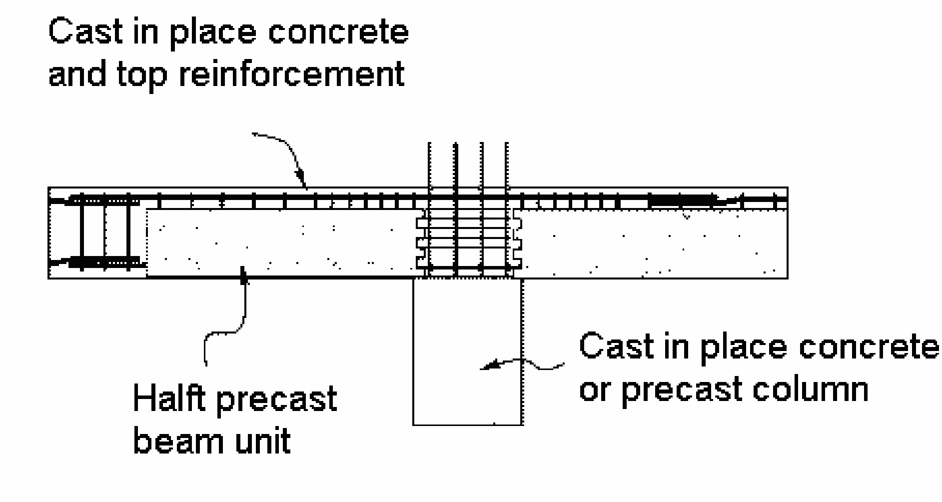


Figure 10-2: Connection, type emulated monolith R/C, using continuous bottom reinforcement

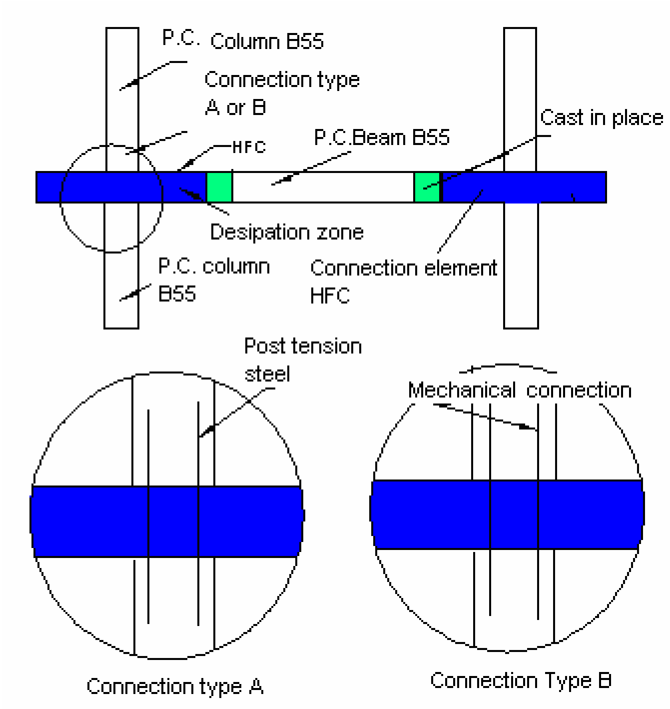


Figure 10-3: Connection type emulated monolith R/C, B55 columns and HFC beams with wet member's connection

Figure 10-4: Connection type hybrid post tension assemblage or ductile rod Dywidag assemblage

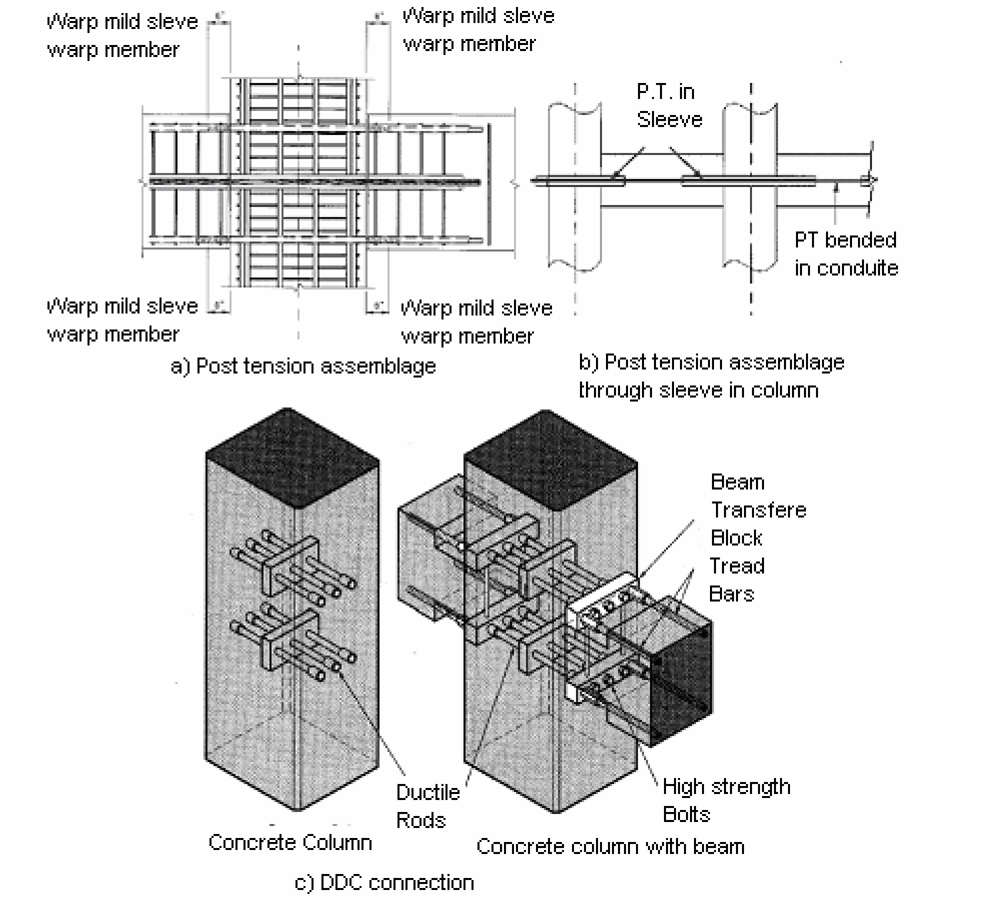


Figure 10-5: DDC connection, post tension assemblage

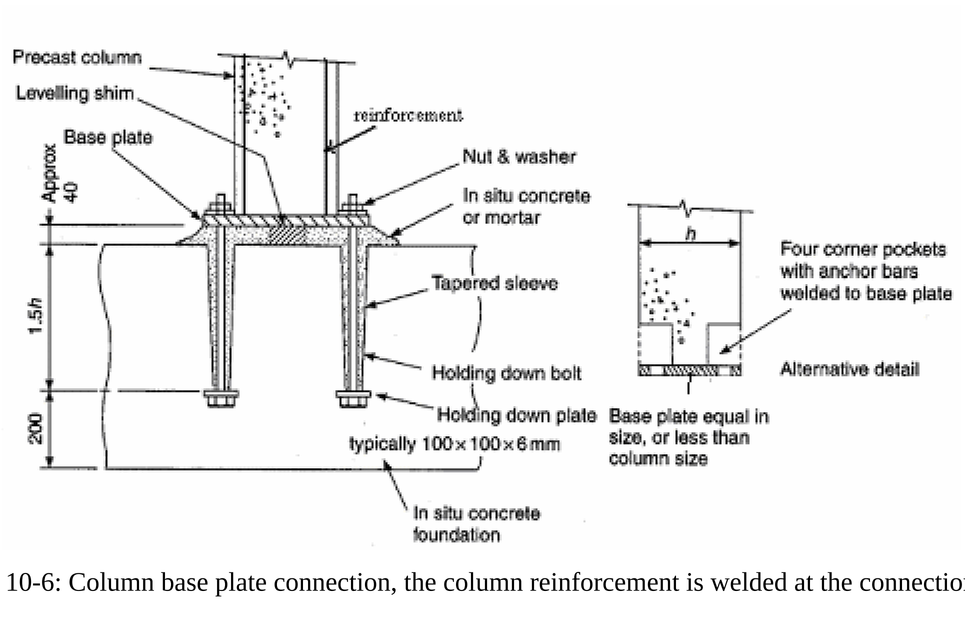


Figure 10-6: Column base plate connection, the column reinforcement is welded at the connection plate

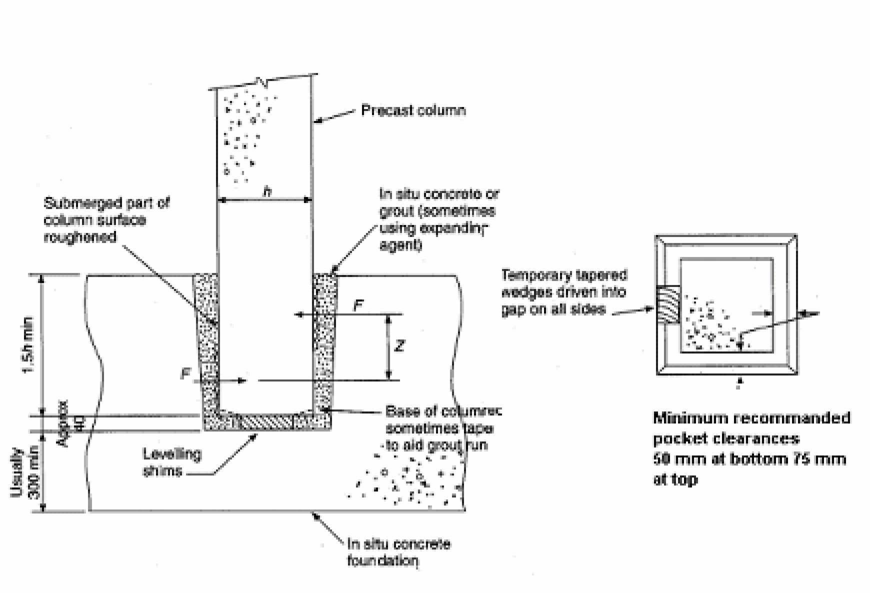


Figure 10-7: Column pocket connection

11 Preliminary design and analysis of the building

The design of the building addresses two major items, Preliminary design and analyses of the building structure, and the design of the reinforced and precast concrete elements of the building. The building designed according to the EC8 requirement. The premier design of the building for the seismic is curried out using simplifying response analyses .The analyses of the building and the response of the concrete members used the equivalent base shear method.
The premier design and analysis can be summarizes in the following steps: Find the action of the dead and the live loads. Design seismic actions. Determine the lateral forces, due to the seismic forces, and the wind loads, the design Loads and combinations. Structural analyses.

In the structural analysis two models are used (chapter11); the elastic model considering the serviceability limit state (wind, dead, live load) and seismic combinations (Dead, live, and seismic load), and the elastic with modified stiffness connection model considering the seismic combinations. The effective stiffness used in the later model are obtained after the design of the concrete structure with the elastic model with 20% reduction of the resultant beams end moment, where the stiffness of the beams end varies according to the reinforcements and the applied load. According to the design requirements the drift of the current floors and the drift of the top floor is less than the permitted drift $\Delta/h<3\%$. The shear force on the building is distributed on the interior column and the exterior columns in a manner lead to decrease the moments at the exterior columns. The yield location at the first floor beam provides flexibility in the structure. The bending moments at the first floor beams are high and in order to get suitable beams depth at the different floors, yield locations at the first floor are applied. The design of the reinforced concrete elements is carried out using the elastic model with 20% reduction of bending moments in the beam-ends. The design optimisation is carried out using the modified stiffness model results and the non-linear analysis with Ruaumoko program. All the elements, beams, columns, floor, and the connections should perform the seismic design requirements. In the design with emulated monolithic R/C as a basic solution and with HFC connection. The design of the columns and the beams end should be followed by the capacity control, while the design with the ductile member connections. And the capacity design requirements are applied through the design requirements and formulation.

11.1 Load calculation

Dead live and wind load may be calculated according to the used material properties. Seismic load is required for the building seismic design. The equivalent shear method is applicable to regular constructions possessing a uniform distribution of mass and stiffness, and without irregular feature that produce a concentration of torsion. The distribution of the lateral forces on the structure elements passes among the system level and the individual elements. The nonlinear behavior of the concrete element in the connections regions where the high influence of the seismic actions can be recognized through the study of the connecting elements properties.

11.1.1 The dead and the live load

The dead and the live load on the building are calculated and the results are summarized in Table 11-3.The calculations are carried out using the following considerations: The transverse beams are the principle loads carrying elements. The transverse elements carry the effect of the seismic and wind loads in the transverse direction and the longitudinal elements caries the seismic and wind load effect in the longitudinal direction. The floor of precast, its weight = 0.20 × 24 = 4.8 kN/m$^{2.}$ The arrangement of the precast units is shown in Figure 11-1. The floor is one-way slab supported on the transverse beams .the longitudinal beams caries strip of 1.2 m width.

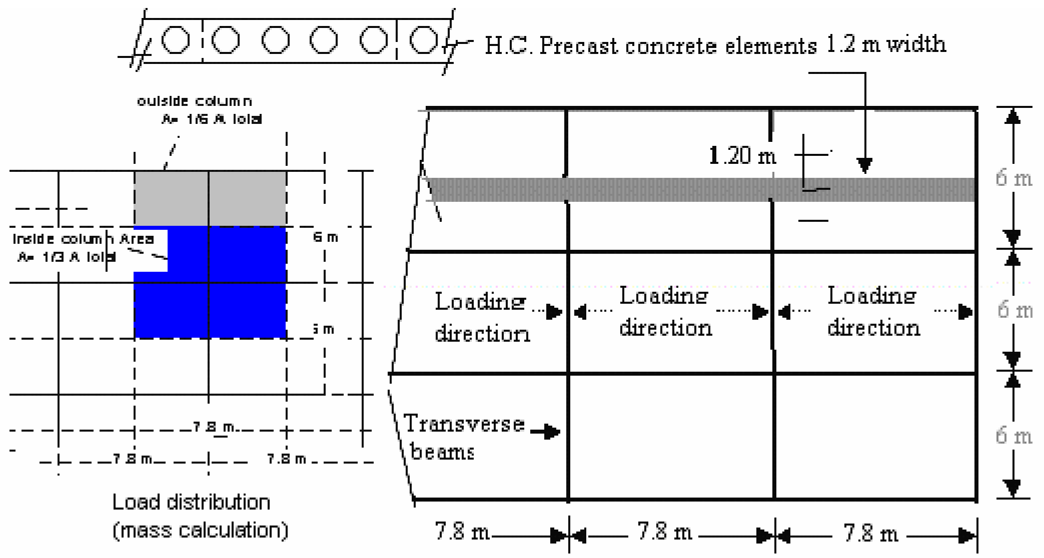


Figure 11-1: Floor of longitudinal, transverse beams and precast units floor

11.1.2 Loads on the beams and the columns

The load on the beams and columns are calculated for the total building in two types. The first is for shear and moment calculation; the loads are used in the elastic analysis. The second type is the mass calculation where they are used for the non-linear analysis. Summary of the load calculation is reported in Table 11-1

11.1.3 Loads and lumped mass for the dynamic analyses

The non-linear analysis of the structure required the information concerns the load and masses on the structure for the use of Ruaumoko program. The loads are distributed according the loading area on each column strip. Figure 11-1 shows the inside and outside column strip areas. In the vertical direction the outside column strip holds 1/6 of the load, while the inside column carries 1/3 of the load. In the horizontal direction x and y the loading is on the columns lines at the intermediate column strip will be; in x and in y direction is calculated using equation (11.1). The Results are summaries in Table 11-2.

$$W_{column} = \frac{\text{floor weigth}}{8\times 4} \tag{11.1}$$

Table 11-1 Summary and description of the load calculation

Location			Dead load [kN/m]	Live load [kN/m]	Description
Longitudinal beams	**Floor**		1.2 × (4.8 + 1.5) + 0.6 × 0.6 × 24 = 16.2	1.2 × (3.6) = 4.32 [kN/m]	0.6 × 0.6 m beams caries 1.2 m width of the floor. The concrete weight: =24 kN/m^3
	Roof		1.2 × (4.8 + 0.5) + 0.6 × 0.6 × (24) =15	1.2 × (1.0) =1.2 [kN/m]	
Transverse beams	Sh.	**Floor**	7.8 × 5/6 × (4.8 + 1.5) + 0.6 × 0.7 × (24) = 51.03	7.8 × 5/6 × (3.6) = 23.4	0.6 × 0.7 m beams caries 7.8 m span of the floor, with ratio of (5/6) of the floor area for shear and assumed uniformly distributed load of (5.5/6) of the floor area for the moment calculations.
		Roof	= 7.8 × 5/6 × (4.8 + 0.5) + 0.6 × 0.7 × (24) = 44.53	7.8 × 5/6 × (1.0) = 6.5	
	Mom.	**Floor**	7.8 × 5.5/6 × (4.8 + 1.5) + 0.6 × 0.7 × (24) = 55.12	7.8 × 5.5/6 × (3.6) = 25.74	
		Roof	7.8 × 5.5/6 × (4.8 + 0.5) + 0.6 × 0.7 × (24) = 47.98	7.8 × 5.5/6 × (1.0) =7.15	
Columns			0.5 × 0.5 × 3.6 × 24 = 21.6 [kN/ column]		Columns dimension: 0.5 × 0.50 × 3.6 m
Load calculation for seismic analysis using Ruaumoko program					
The weight of each floor					
Floor	Mass.	**Floor**	982.8 × 6.3 + 144 × 8.64 + 218.4 × 10.08 + 32 × 33.7 = 10715kN/floor	=982.8 × 3.6 = 3538 kN /floor	The total area of the building: 54.6 × 18= 982.8 m^2 Weight: = 6.3 kN/m^2 floor = 5.3 kN/m^2 roof The total length of the transverse beams: = 8 × 18=144 m Weight = 8.64 kN/m The total length of the longitudinal beams: = 4 × 54.6= 218.4m, Weight =10.08 kN/m The number of floor columns= 32 column, weight = 33.7 kN/floor column
		Roof	982.8 × 5.3 + 144 × 8.64 + 218.4 × 10.08 + 32 × 33.7 = 9732 kN/roof	=983 ×1= 983kN /roof	
The seismic load combination, should take into the account the dead, and the live load. With G is the dead load, and Q is the live, the dead and the live load on the structure are calculated as follows: $G+\psi_E Q$, ψ_E=0.30 for the roof, and ψ_E = 0.15 for the current floors					

For the non-linear analysis, the member end fixation are calculated as follows:

The fixed end moments

1. The current floor

The load combination; E1 = G+0.15 Q+ H (11.2)

Where H is the seismic load, G the dead load and Q is the live load

For shear calculation

DL+0.15LL = 51.03+0.15×23.4 = 54.54 kN/m
Shear force S = 3×54.54 =163.62 k N

For moment calculation

DL + 0.15LL = 55.12+0.15×25.74 = 58.83 kN/m

The fixed end moment; $M_f = -\frac{WL^2}{14}$, M_f = -58.83×36/14 = -151.3 kN.m (11.3)

2. The Roof

Load combination; E1 = G+0.3 Q+ H (11.4)

Shear calculation; DL+0.3LL = 44.53 + 0.3×6.5 = 46.48 kN/m
Shear force S = 3×54.54 =139.5 k N

Moment calculation

DL + 0.3LL = 47.98 + 0.3×7.15 = 50.12 kN/m
Fixed end moment; M_f = -58.83×36/14 = -128.9 kN.m Using equation. (11.3)

Table 11-2: Loads on the exterior and interior columns for lumped mass calculations

Location		Loads [kN]		
		X direction	Y direction	Z direction
Floor	Exterior column.	352	352	268
	Interior column.	352	352	536
Roof	Exterior column.	313	313	239
	Interior column.	313	313	478

11.1.4 The wind load

The effect of the wind load while it is not significant in comparison with the seismic effects, it is calculated based on average distribution of the wind load on the area of the building. With Q_w = 0.8 kN/m^2

(a) The current floor: Each current floor is exposed to a lateral wind force as follows:

In the longitudinal direction: $H_w = 0.8 \times 3.6 \times 18 = 51.8$ kN

In the transverse direction: $H_w = 0.8 \times 3.6 \times 54.6 = 157.25$ kN.

(b) The first floor: The first floor is exposed to lateral wind force as follows:

In the longitudinal direction: H_w = 0.8x(3.6+4.8)/2 18 = 60.5 kN

In the transverse direction: H_w = 0.8(3.6+4.8)/2*54.6 =183.46 kN

(c) The roof: The roof is exposed to lateral force as the follows:

In the longitudinal direction: $H_w = 0.8 \times 3.6/2 \times 18 = 25.9$ kN
In the transverse direction: $H_w = 0.8 \times 3.6/2 \times 54.6 = 78.63$ kN

Summary of the acting forces due to dead, live, and wind loads on the buildings structure are reported in Table 11-3

Table 11-3: The dead live load and wind loads on the buildings structure, in [kN].

Floor level	Story DL	Story LL	Wind load Hw Long.	Trans.	Seismic Load $G+\psi_E Q$	Wind shear force Long.	Tarns.
Roof	9732	983	25.9	78.63	10027	25.9	78.63
5th	10715	3538	51.8	157.25	11246	77.7	235.88
4th	10715	3538.	51.8	157.25	11246	129.5	393.13
3rd	10715	3538	51.8	157.25	11246	181.3	550.38
2nd	10715	3538	51.8	157.25	11246	233.1	707.63
1st	10715	3538	51.8	157.25	11246	284.9	864.88
0			51.8	157.25		336.7	1022.13

11.2 Determination of the seismic actions

The mainframe in the transverse direction is considered for the purpose of the seismic calculations. The seismic response is calculated as follows:

For $T_c \le T \le T_d$; $S_d = a_g . S \frac{2.5}{q} (\frac{T_c}{T})$ (11.5)

For ground group type B; The1elastic response spectrum factor is obtained as follows:

$S = 1.2,\ T_b = 0.15,\ T_c = 0.5,\ T_d = 2$ (11.6)

The behaviour factor q, for frame equivalent system may be obtained as follows:

$q = q_0 . k_w$; With $k_w = 1$ (11.7)

The seismic area is considered in Greece, zone III, according to the Greek Code for earthquake resistant structures.

The base acceleration is obtained as: $a_g = 0.24$ g.

The fundamental period is carried out as follows:

$$T_1 = C_t . H^{\frac{3}{4}} \tag{11.8}$$

For moment resistance concrete frame $C_t = 0.075$, the period T1= 0.783 second and the response of the building structure S_d is obtained as follows:

$S_d = 1.156$ Using equation (11.5)

The base shear force F_b is calculated as follows:

$$F_b = S_d .m.\lambda \tag{11.9}$$

The correction factor λ for $T_1 \leq T_c$ is calculated as follows:

For buildings with more than two storeys $\lambda = 0.85$.

The mass of the building M=W/g , M=66257/g (11.10)

The total base shear force is obtained as follows:

Fb = 6639 kN Using equation (11.9)

11.2.1 The lateral seismic forces

The Lateral seismic forces may be obtained as follows:

The fundamental period is $T_1 = 0.075 h^{\frac{3}{4}}$ using equation (11.8)

The lateral shear force $F_b = S_d \frac{\lambda}{g} \sum_i W_i$ (11.11)

$$Fb = 0.1 \sum_i W_i = 6626 \text{ kN}$$

The distribution factor of the lateral seismic force on the floor is calculated using equation(11.12), and (11.13) as follows:

$$\lambda_i = \frac{z_i W_i}{\sum_i (W_i . z_i)} \tag{11.12}$$

The lateral force acting on each floor is calculated as follows:

$$F_i = F_b . \frac{z_i \cdot W_i}{\sum_i (W_i . z_i)} \tag{11.13}$$

The building consist of 8 frames, each frame resist one eighth of the total seismic shear force The distribution of the lateral seismic force on the building are obtained using computer program. The calculation results are illustrated in 11.2.2

11.2.2 The lateral forces, results

The seismic effect on the building is calculated using a simple computer solve the equations (11.8) through (11.13). The input values of each floor weight are given in Table 11-3.

The fundamental period T1 and the total lateral force F_b, are obtained as follows:

T1 = 0.783 s, Fb = 6.626 10^3 kN

Summary of the calculations results; the lateral seismic and the shear forces are shown in Table 11-4. It is recognized that the lateral seismic shear force are high at the first floor and it decreases at the higher floor.

Table 11-4: The acting forces on the transverse frame

Floor numbe	W	λ	F_ Building	F_mainframe	Shear_mainframe
6	$1.003\cdot10^4$	0.253	$1.677\cdot10^3$	209.594	209.594
5	$1.125\cdot10^4$	0.239	$1.584\cdot10^3$	197.958	407.552
4	$1.125\cdot10^4$	0.194	$1.287\cdot10^3$	160.841	568.393
3	$1.125\cdot10^4$	0.149	989.789	123.724	692.116
2	$1.125\cdot10^4$	0.105	692.852	86.607	778.723
1	$1.125\cdot10^4$	0.06	395.916	49.489	828.212

The total force on the main frame = 828.213 kN

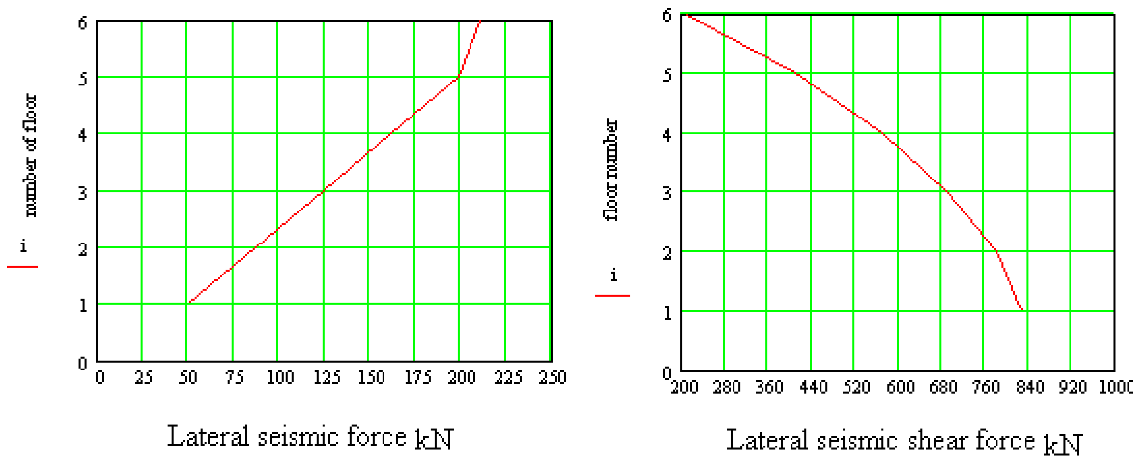


Figure 11-2: the lateral seismic and shear force acting on each floor at the transverse frame

11.3 Design load combinations

The design loads combinations are carried out according to EC8 includes two combinations; Ed1, Ed2 are the first and the second earthquake, as follows:

The service limit state; dead loads and Live loads are given as follows:

$$Ed1 = E\,(1.35\,G + 1.5\,Q) \tag{11.14}$$

The seismic combination; dead loads, live loads, and seismic actions are give as follows:

$$Ed2 = E(G + \sum \psi_E Q + H) \tag{11.15}$$

With $\psi_E = 0.3$; $\psi_E = \phi . \psi_E$, for the last floor $\phi = 1$, and for every other floor $\phi = 0.5$

11.4 The structural analyses

The building structure is modelled as frame as follows:

- The first is the elastic model. In this model we consider two combinations. The first combination is for the serviceability limit state; dead load, the live load and the wind load (Figure 11-4), The second series of combination are the seismic combinations (Figure 11-5 through Figure 11-9).
- The second model uses modified stiffness at the column beam connections. In this model dead and seismic loads are considered using the seismic combinations. The analysis results are reported in Figure 11-11 through Figure 11-12.

The lateral forces, the wind load, and the seismic load are used each of them separately in the load combinations. While the dead and live loads share both types of the load combinations (Figure 11-4). For the numbers of the beams and the column beam connections refer to Figure 11-3.

Table 11-5: Dead and live load

Dead load			Live load		
Location	[kN /m]	[kN]	Location	[kN/m]	[kN]
M25,M30,M47 M50	55.12		M25,M30,M47 M50	25.74	
M40-M42	47.98		M40-M42	7.15	
N1-N28		33.17	N1-N28		15

Table 11-6: Lateral forces, seismic and wind load

Location	Horizontal seismic load [kN]	Horizontal wind load [kN]
N5	50	22.49
N9	87	22.46
N13	124	22.46
N17	161	22.46
N21	198	22.46
N25	209	11.23

Table 11-7: Load combinations (A) includes seismic load

Load case	Description	Load combination.			
		L.C.1	L.C.2	L.C.3	L.C.4
(A) DL+LL+WL					
Load case1	Dead Load	1	1.2	1.2	1.2
Load case2	Live load	1	1.5		1.5
Load case3	Live Load	1		1.5	
Load case4	Wind load			1.5	1.5
(B) DL+LL+S					
Load case1	Dead Load	1.35	1.35	1	
Load case2	Live load	1.5		0.15	
Load case3	Live Load		1.5		
Load case4	Seismic			1.0	

11.5 Analyses results

The structure shows more proper redistribution of the forces on the structural members and especially at the ground columns and the first floor beams. And that is after the consideration of the yield locations at the first floor beams. Different manner of the forces redistribution on the structure can be obtained, considering the member ends behaviours, according to their geometry, material dimensions, and the rotation capacity. The question to be solved is to insure the joint connection behaves within a range suitable for the seismic actions.

The structure analyses model; the geometry, the bending moment diagram, the shear diagram, and detailed bending moment diagram at the first and second floor are illustrated. The analyses results, loads and load combinations are shown in Figure 11-4 through Figure 11-13.

- The elastic model

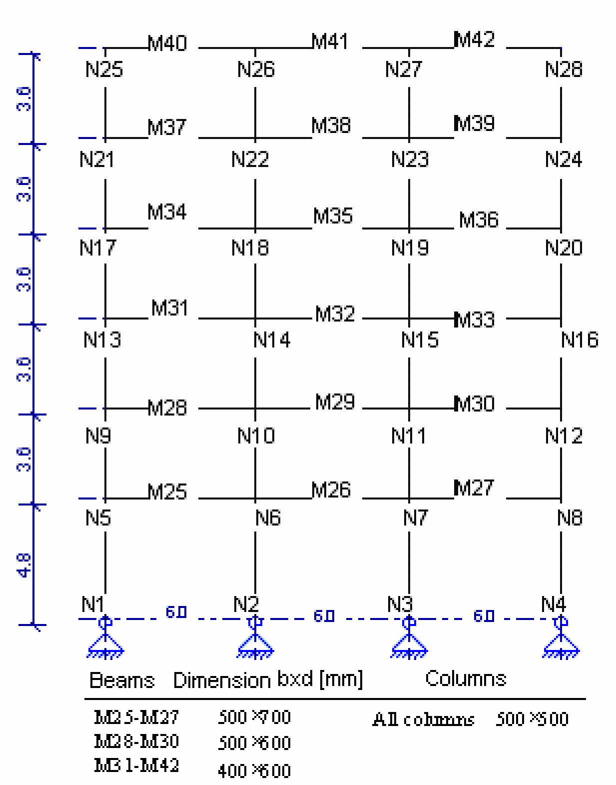
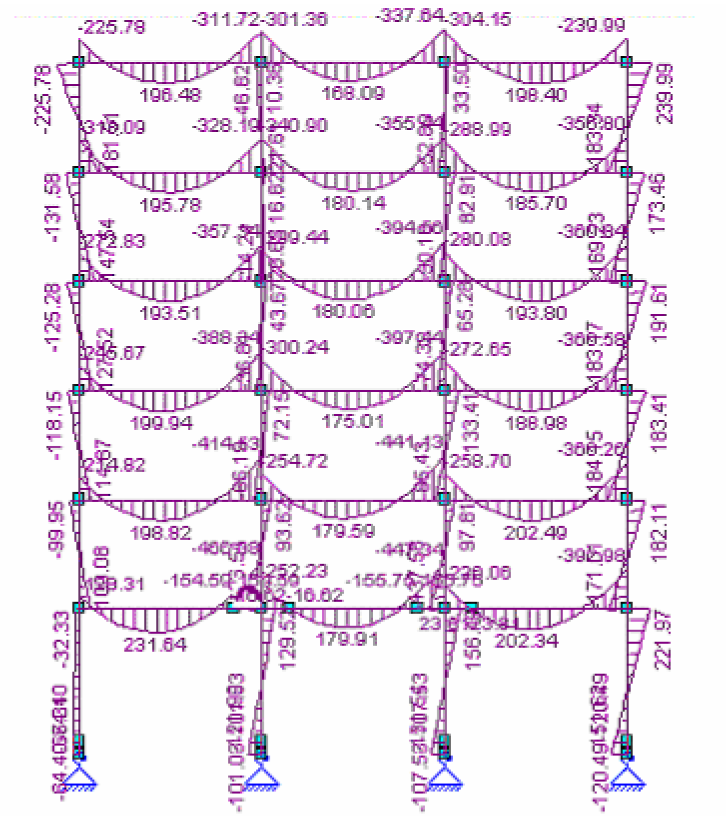
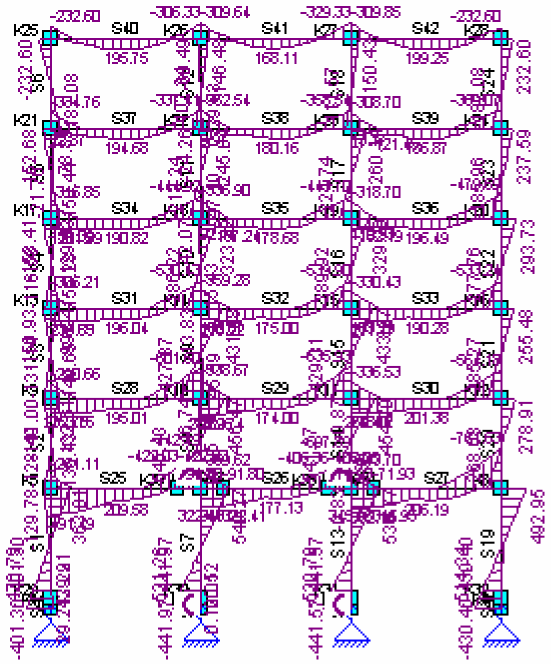


Figure 11-3: The structural model - geometry



Load case	Description	Load combination			
		L.C.1.	L.C.2	L.C.3	L.C.4
L c1	Dead Load	1	1.2	1.2	1.2
L c2	Live load	1	1.5		1.5
L c3	Live Load	1		1.5	
L c4	Wind load			1.5	1.5

Figure 11-4: Bending moment diagram at the building, considering dead load, live load and wind load with elastic stiffness column beams connection model



Load case	Description	Load combination		
		L.c1	L.c2	L.c3
L.c.1	Dead load	1.35	1.35	1
L.c.2	Live load	1.5		0.15
L.c.3	Live load		1.5	
L.c.4	Seismic			1.0

Figure 11-5: Bending moment diagram considering seismic action with elastic stiffness.

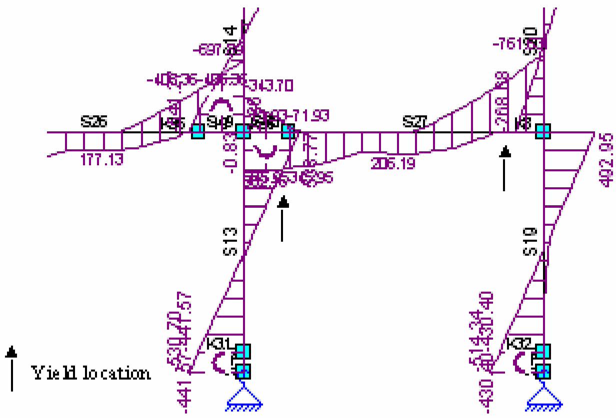


Figure 11-6: Bending moment at the first floor, with elastic stiffness

The yield locations in the beams may be defined after reading the bending moment diagram. They may be located in places where the reversal moment can be occurred. The yield locations indicated in Figure 11-6 are subjected to reversal action; the lower face of the beam is in tension in one seismic direction and it will be in compression as the seismic action works in the reverse direction.

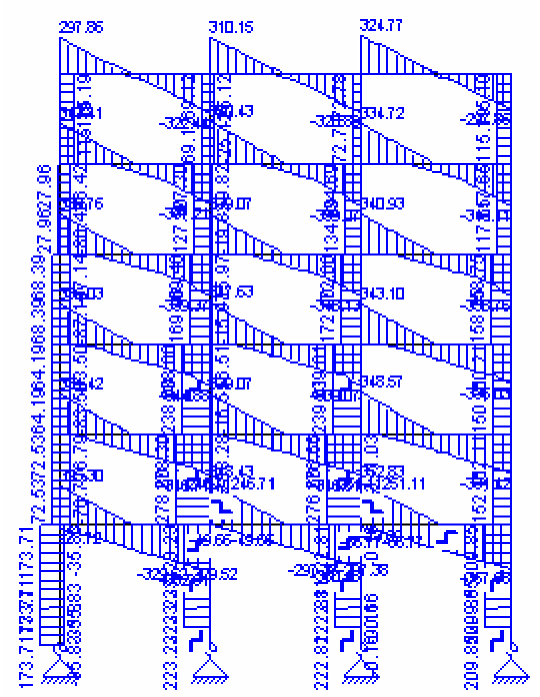


Figure 11-7: Shear diagram, elastic stiffness model

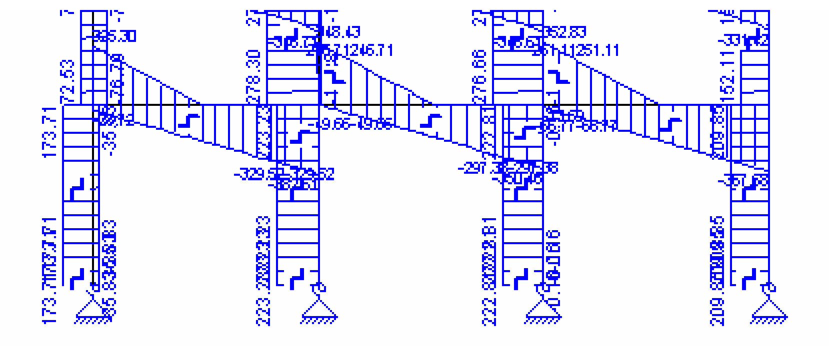
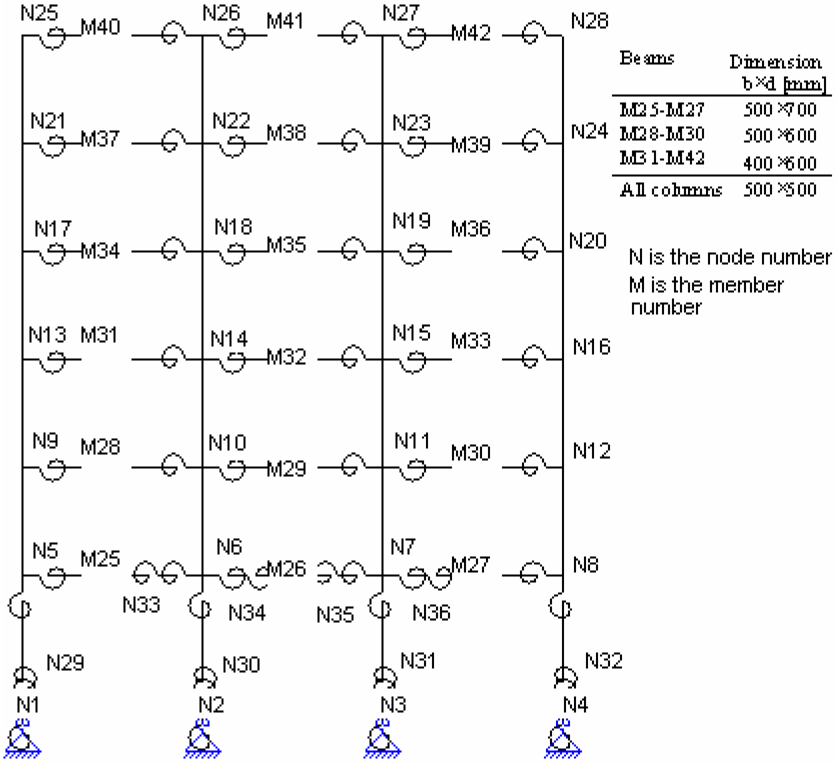
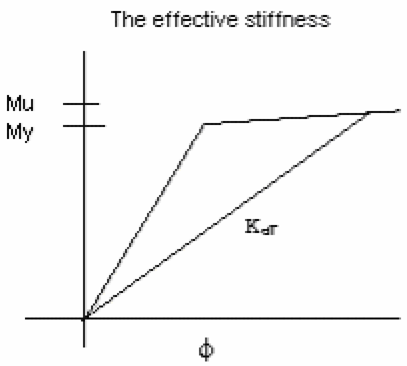


Figure 11-8: Shear diagram at the ground and first floor, elastic model

- The modified stiffness model





Level	K_{eff} kN.m/rad	
1st	60000	
	100000	exteriour connection
2nd	60000	interiour connection
	60000	
3rd	60000	
	60000	
4th	70000	
5th	70000	
	70000	
6th	160000	
	100000	

Load combinations

Load case	Description	Load combination		
		L.C.1	L.C.2	L.C.3
L c1	Dead load	1.35	1.35	1
L c2	Live load	1.5		0.15
L c3	Live load		1.5	
L c4	Seismic			1.0

Figure 11-9: Modified elastic stiffness model, the effective stiffness, and the loads.

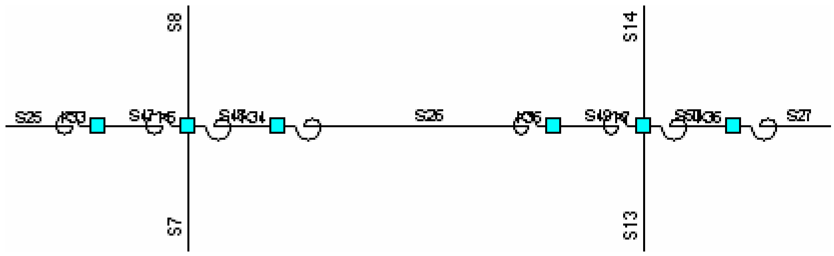


Figure 11-10: Modified elastic stiffness model, the first floor

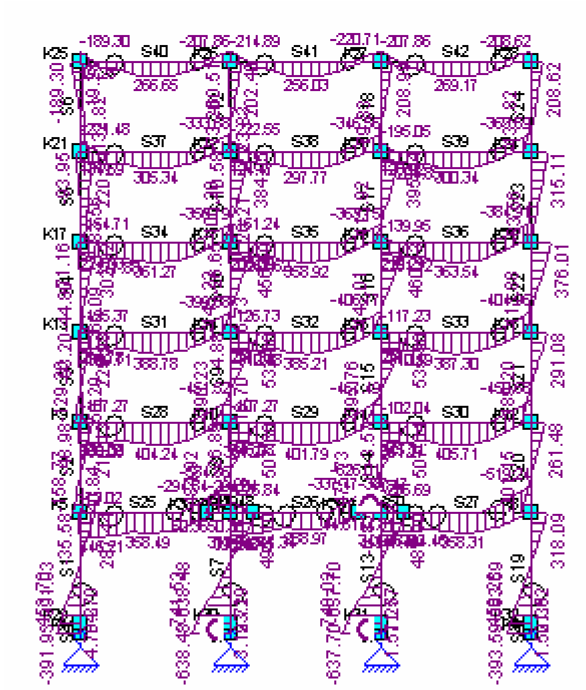


Figure 11-11: Bending moment diagram in the transverse frame, using modified stiffness column beam connections, considering seismic action

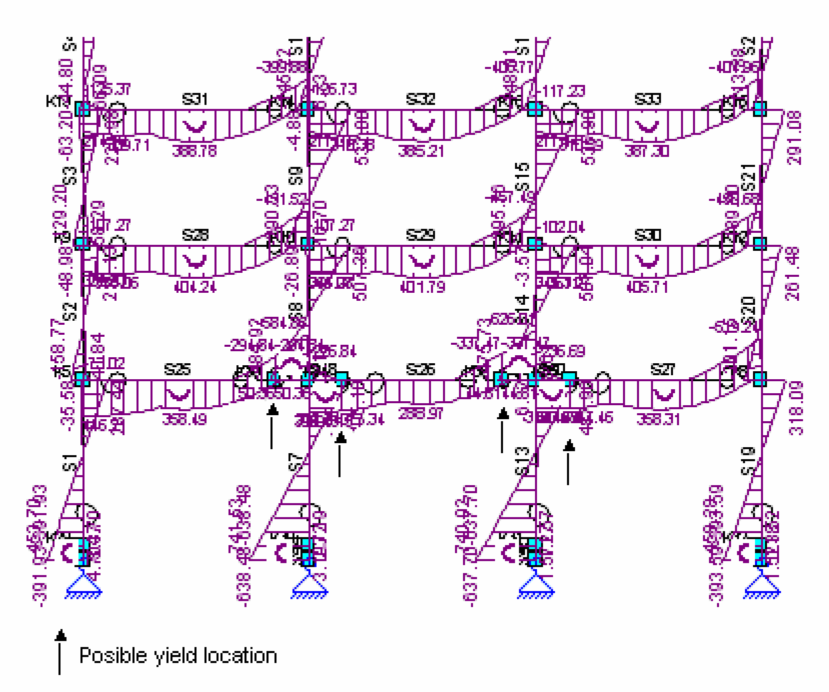


Figure 11-12: Bending moment diagram at the lower floors

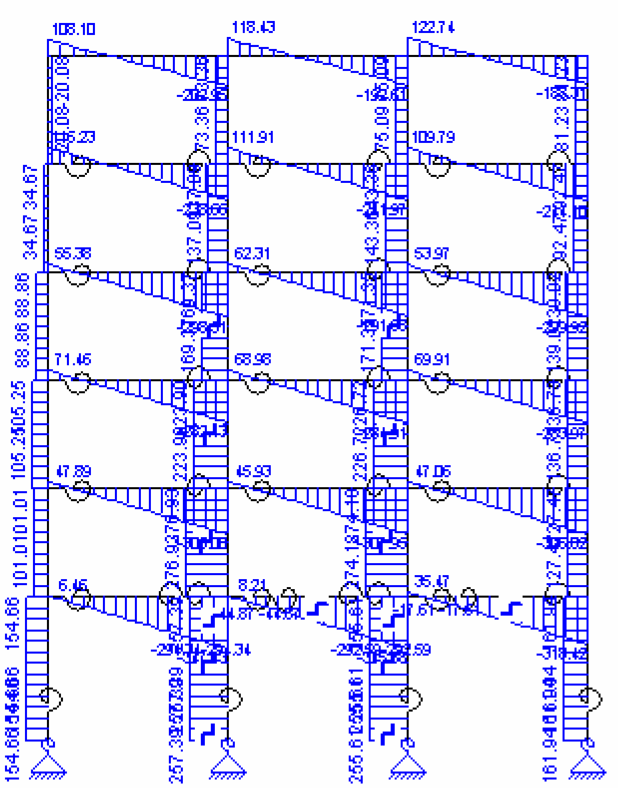


Figure 11-13: Shear diagram in the transverse frame, with modified stiffness columns beams connections, considering seismic actions

12 Design of reinforced concrete structure

The design of the building is carried out for the four connection types: the design with emulated monolithic construction (chapter12), the design with HFC, DDC, and hybrid post-tension assemblage (chapter13). Summary of the design for all the connection types is reported in chapter14.

The design of the emulated reinforced concrete is carried out as follows:

- The premier design of the columns

The columns design is applicable for the different variant and the design optimization required monitoring the change in the columns reinforcement due to the capacity design requirements.

- The design of the beams with emulated monolithic R/C and hybrid-fibre connection.
- The capacity design of the columns

In the designing of ductile moment resisting frame subjected to lateral loads; it is desirable to prevent the formation of flexure higher in columns. Plastic hinges in columns not only resulting in irreparable permanent sway in the structure, but may cause large overturning moment on the structure. The yield locations are assumed at the column bases. In the other floors additional light reinforcement added to the connection core, so that the yield location should be deviated from the columns beam connection faces. The design for the flexural moment in the beams includes the design of the yield location locations, and the design for shear in the beams includes the control of the possible plastic hinge formation at the beam-ends, which is an application for the capacity design requirements. The selection of the reinforcement and should fulfill the details requirements according to EC8. The members are verified to full fill the required strength and the ductility. In the first assessment for the ductility the local curvature ductility in the beams ends are obtained using the q factor obtained from the design spectrum and apply relations given by the CEB for the design period T. In the capacity design application the strength required in the columns for the seismic actions of the beams should be insured, in one direction and in the reversed direction. The capacity design application leads to the conclusion that, in case of the proper design for the beams, as the necessary dimensioning, the reinforcement, and the proportionality of the columns beams inertia. The beams may not exert high moments on the columns, which leads to full fill the capacity requirements without significant modifications in the columns reinforcement. In our building the minimum design reinforcement of the columns in the critical regions is applicable. The most important factors are the sum of the moment ratio factor α, and the moment reversal factor δ, both of them are function of the loading and the section properties. The design of the concrete structure is optimized using the modified elastic stiffness, in which the effective stiffness of the connections is obtained based on the design with the elastic frame model. The resulting bending moment, in the column beam connections with modified stiffness model is about 80% of that which is obtained with the elastic model results, and the design reinforcement in the beams and the columns are identical. Further modification for the columns dimensions within this model leads to a reduction in the ground and first floor columns so that all the column dimensions in the building are 500mm×500mm. The reinforcement of the elastic model is used for the estimation of the modified stiffness to build the modified elastic stiffness model, and the reinforcement in the members is designed according to new model. The design concrete structures of both models are verified using the non-linear analysis program Ruaumoko. The design with the modified stiffness resulted in better distribution of the ductility among the members, and better ductility proportion of the top and the bottom faces in each member end. The top and the flexural reinforcement share the seismic moment resistance. Both design models are verified for degradation, using Takeda degrading and plastic hinges at the columns bases. Both design concrete structure have ductility and strength resistance within the nonlinear analysis results.

This chapter covers the design of the reinforced concrete structure. The columns and the beams are designed with B55 R/C according to EC8. The column beam connections are verified according to the capacity requirements. Two main parts in the design are carried out:

(A) Design the concrete structure using the elastic stiffness model, considering the serviceability and the seismic combinations with a reduction of 20% of the moments at the column beam connections.
(B) Optimise the design using the effective stiffness model. In the design the non-linear behaviour of the concrete is applied at the yield locations. And the effective stiffness at the connections is applied according to the R/C section properties.

The dimensioning of the beams is carried out in proportionality to the seismic action on the structure; at each floor using procedure (A) for the first estimation the dimensions are modified using procedure (B). Finally the design is carried out for the structure using (A) then (B). The application of the elastic model uses a reduction of 20% of the moments at column-beam connections. The application of the effective stiffness model is carried out using the design results of the elastic model. In a building with tall ground floor columns the bending moment at the first floor beams are high. That leads to construct with high beam girders, which is not recommended in precast constructions precast construction. The precast elements and the connection elements are produced in typical limited in dimensions, strength and types of ductile members. The solution for that is to apply a mechanism reducing the seismic bending moments in the girders. This problem is solved by construction with semi rigid beams in the first floor, shortening the columns moment arm. That needs to control that the beams yield occurs in locations away from the column faces.

In the design of the precast concrete element, the decision of reinforcement differs from that in the monolith construction. The reinforcement detail is important. The rise the bending moment diagram point of inflection from the column base may provides a solution to decrease the bending moments on the first floor beams. That costs high and it is difficult to control in the monolith constructions. And it is easy to realize in precast constructions.

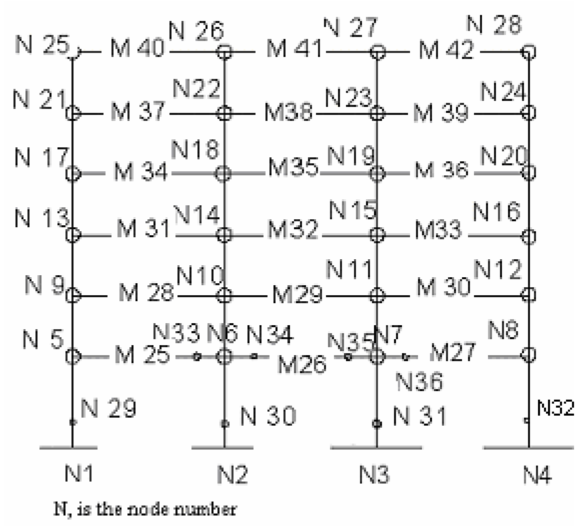


Figure 12-1: Structural model of the building, M is the member numbers and N is the node number

Table 12-1: Column and beam dimension

Column	b×d [mm]	Column	b×d [mm]
M1, M7-9, M13-15, M19, M43-46	500×500	M2-6, M20-24	500×500
M10-12	500×500	M16-18	500×500

Beam	b×d[mm]	Column	b×d [mm]
M25-27	500×700	M28-30	500×600
M31-42	400×600		

12.1 Design with elastic model

The design of the concrete structure is carried out for the elastic model. In this model two types of load combinations are used, the seismic combinations, and the serviceability combinations. The design of the concrete structure includes the premier design of the columns, the design of the beams, and the capacity design.

12.1.1 The strength design of the columns

The columns are designed with emulated monolithic reinforced concrete using the interaction diagram CEB 1982. The critical regions of the columns near the column beam connections are verified for the minimal reinforcement ratio. The capacity design principles are applied for the seismic beam action, where it was applied for the most sever columns actions at the exterior and the interior connections. In the followings the design of the interior and the exterior ground floor columns is introduced. For the structure geometry, and member dimension refer to Figure 12-1 and to Table 12-1.

The interior column beam connection at N6

The flexural reinforcement

The flexural reinforcement is obtained using CEB 1982 coefficients for bending moment and normal forces as follows:

$$\mu = \frac{M_d}{b \cdot h^2 \cdot f_c} \;,\; \nu = \frac{N_d}{b \cdot h \cdot f`_c} \qquad (12.1), (12.2)$$

With moment and normal force on the columns of 500×500 mm the required reinforcement is obtained as follows:

M_d = 540 kN.m, N_d = 2322 kN $\Longrightarrow$ $\mu = 0.131 \cdot 10^{-4}, \nu = 0.201$

The required reinforcement at the critical region according to CEB 1982, is the minimum reinforcement of ω = 0.13, with f_{yd} = 435 N/mm^2. The required reinforcement A_s = 2ϕ16 +2ϕ20 mm each side. The corresponding resistance moment of the column in both directions are obtained as follows:

M_u = 608 kN.m, and M_y = 557 kN. m Using equations (7.9) and (7.10)

The transverse reinforcement

The nominal shear on the column is calculated using equation (12.3) with the followings: $M_{R\_U} = M_{R\_O} = M_u$=608 kN, the column length Lc = 3.0 m, and γ_{Rd}=1.2

$$V_{sd} = \gamma_{Rd} \frac{M_{R\_u} + M_{R\_o}}{L_c} = 1.2 \cdot \frac{608+608}{3} = 486 \text{ kN} \qquad (12.3)$$

The shear reinforcement in the critical length L_c is carried out as follows:

$$V_{cd} = b_w \cdot d \cdot \left\{ \tau_{Rd} \cdot (1.6\text{- } d) \cdot (1.2 + 40 \cdot \rho_1 + 0.15 \cdot \sigma_{cp} \right\}$$

$$\text{with} \quad \sigma_{cp} = \frac{N_v}{b_w h} \quad \text{and} \quad \rho 1 = \frac{A_s}{b_w \cdot h} \qquad (12.4)$$

And with $\tau_{Rd} = 0.5 \dfrac{N}{mm^2}$, b_w =0.5m , h=0.5m , N_v =507kN .

The concrete shear resistance is V_{cd} = 296 kN.

The required transverse reinforcement shear resistance is calculated as follows:

$V_{wd} = V_{sd}\text{-}V_{cd}$ = 486-278 = 208 kN

The transverse reinforcement is calculated as follows:

$$V_{wd} = \frac{A_s \cdot 0.9 \cdot d \cdot f_y}{s} = \frac{(2+\sqrt{2}) \cdot 50.2 \cdot 0.9 \cdot 500 \cdot 435 \cdot 10^{-3}}{150} = 223 \text{ kN} \qquad (12.5)$$

The critical region in the column base is calculated for the possible maximum horizontal shear seismic force occurs in the situation where the building columns yield. The base columns are subjected to maximum shear force due to the building weight accompany with seismic action (Figure 2-1). The design shear force should be calculated as follows:

$$V_{sd} = \gamma_{Rd} \frac{M_{R\_u} + M_{R\_o}}{L_c} + N_c \cdot \frac{U}{h_c} + \frac{F_b}{n_h}$$

with the maximum displacement of the building, column weignt,base shear force as follows:

$$U = \frac{\sum M_{column}}{W_{frame}}$$

$$N_c = \frac{W}{n_v} = \frac{8280}{3} = 2760 \text{ kN}, \quad F_b = 828 \text{ kN}$$

$$U = \frac{4 \cdot 528}{8280} = 0.255 \text{ m}$$

$$V_{sd} = 1.2 \frac{645 + 609}{3} + 2760 \cdot \frac{0.255}{3} + \frac{828}{4} = 943 \text{ kN}$$

The shear resistance of the column section should be greater than the design seismic force:

$V_{Rd} = 2500$ kN > $V_{sd} = 1011$ kN. OK.

The spacing of the hoops in the critical region (L_{cr} = 750mm) should not exceed the following limits:
S_w = min(b/3, 150mm, $7d_{bl}$) = (min 500/3, 150mm, 7×16mm) = 112 mm

Use double hoop transverse reinforcement 2×(2ϕ8@110mm) in the critical region and 2×(2ϕ8@150mm) outside the critical region.

The exterior column beam connection at N5

The flexural reinforcement

Following the same procedure used for N6 the flexural reinforcement may be obtained as follows:

M_d = 428 kN.m, and N_d = 1524 kN, $\Longrightarrow$ $\mu = 0.1$ and ν=0.128

The minimum reinforcement of 3ϕ16mm is required in the critical region at each side of the column. The distance between consecutive restrained longitudinal bars should not exceed 250 mm that required using double hoop transverse reinforcement.

The shear reinforcement

The flexural resistance of the exterior columns are the similar to the interior columns. The shear resistance of the concrete is V_{cd}= 296 kN. Use double hoop 2ϕ8 @150mm. In all columns the minimum flexural reinforcement is applicable in the critical regions of 0.75m lengths, and it may be used outside the critical regions.

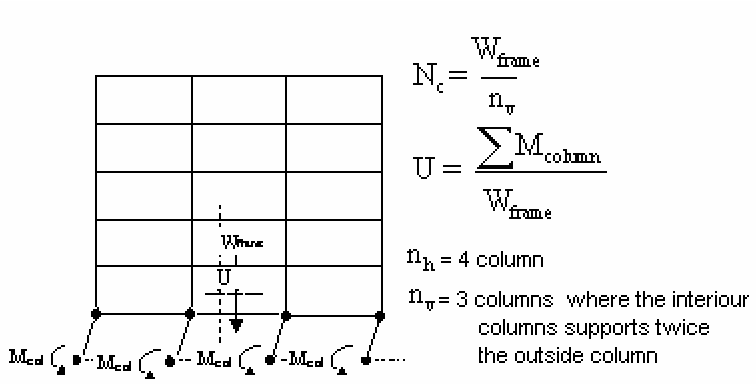


Figure 12-2: The building under collapse

Figure 12-2 shows the situation of the building where the maximum horizontal shear force on the column base occurs due to the building self-weight.

12.1.2 Strength design of the beams

The calculations of the flexure and shear resistance reinforcement have been carried out using equations (12.6) through (12.8). The members are verified for the ductility. In the first assessment for the ductility, we may obtain the local ductility of the structure, which is obtained from the global ductility. The global ductility of the structure in general is for the top sway displacement in the case of a frame, evaluated at the ultimate and the yield conditions. It has been noted that the ductility itself does not represent a sufficient parameter to estimate the structures damage. Therefore a more complete characteristic of the inelastic response of the structure requires the introduction of the damage parameters, such as the cyclic ductility, hysteresis ductility that obtained from the seismic simulations. The local curvature demand ductility has been carried out using equation (12.6) according to EC8 and it is verified finally, using the nonlinear dynamic analysis program Ruaumoko. The beams and the column should be verified according to the capacity requirements in section (12.1.3).

$$\mu_\varphi = 2q - 1 \quad \text{if } T_1 \geq T_c$$
$$\mu_\varphi = 1 + 2 \cdot (q-1) \cdot T_c / T_1 \quad \text{if } T_1 < T_c \tag{12.6}$$

Based on the following relations:

$$\mu_\varphi = 2\mu_\delta - 1 \quad \text{and} \quad \mu_\delta = q \quad \text{if } T_1 \geq T_c$$
$$\mu_\delta = 1 + (q - 1) \cdot T_c / T_1 \quad \text{if } T_1 < T_c$$

Where T_1 is the fundamental period of the structure building; T_1 =0.783 s. using equation (11.7), T_c is the parameter of the elastic response spectrum it is depends on the soil class, and according to the EC8, and for soil class B, where our building is assumed, T_c = 0.5, and q =3.9

The local demand curvature ductility is: $\mu_\phi = 2 \cdot 3.9 - 1 = 6.8$ Using equation(12.6)

The flexure and the shear reinforcement are calculated using equation (12.7)and(12.8)

$$A_s = \frac{M_d}{0.9 \cdot d \cdot f_s}, \ V_{Rd} = V_{cd} + V_{wd} \qquad (12.7) \text{ And } (12.8)$$

Where V_{cd} is the concrete shear resistance and V_{cd} is the shear resistance by the web reinforcement.

The design shear of the beams considered is carried out considering possible plastic hinge formation at the beam-ends using the following relation

$$\text{Shear strength} \geq \gamma \cdot \frac{M_{p1} + M_{p2}}{l_p} + \frac{W}{2} \text{ . With } W = (DL + 0.15LL) \cdot L_{beam} \qquad (5.6)$$

Where $M_p = M_u$, and M_u are given in Table 12-2 for each beam end.

The minimum spacing of the transverse reinforcement in the critical region is calculated according to EC8 as follows:

$S_w = \min(h_w/4, 24d_{bw}, 7d_b, 200\text{mm})$

$S_w = \min(600/4, 24\times8, 7\times16, 200\text{mm})$

The transverse reinforcement is 2 hoop: (12.9)

ϕ8 mm @S_w = 112 mm in the critical region.

ϕ8 mm @S_w = 150 mm outside the critical region.

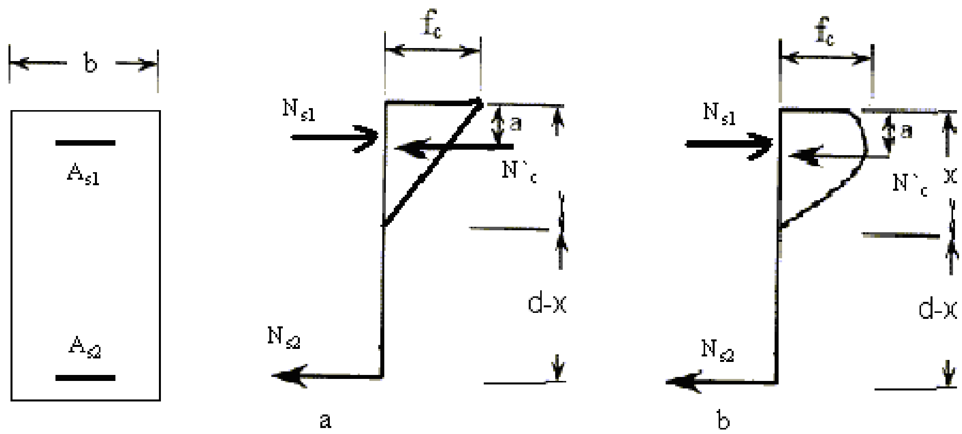


Figure 12-3: Beam section with stress strain diagram at yield and ultimate state

Table 12-2: Summary of the beams reinforcement with emulated monolith B55 R/C elements, using the elastic model.

Type: Member - Node	Loading [kN.m]		Flexure reinforcement		Moment [kN.m]		Ductility
	$-M_d$ $-M_{d\_}$ (1)	80%× (1)	A_{st}	A_{sb}	M_u	M_y	μ_ϕ
Type1: M25-N5 +	-761 +491 +210	-609 +393 + 335	4@20+3ϕ16 3ϕ16	2ϕ20+3ϕ16 6ϕ16+3ϕ10	-596 +396	-460 +329	9.74 9.4
Type1: M47,M48 N33 N6,N7	-342 +422 - 742 +390	-273 +352 -579 +312	5ϕ16 3ϕ16+4ϕ20	6ϕ16 5ϕ16	-316 +387 -596 +318	-262 +315 -460 +262	9.50
Type1: M26-N34 +	-406/-362 +206	-324/-290 +283	4ϕ16+2ϕ12 6ϕ16 3ϕ16	3ϕ16+3ϕ12 6ϕ16	-324 +300	-269 +247	9.3
Type 2: M28-N9 +	-635 +282 +201	-505 +226 +293	2ϕ20+5ϕ16	4ϕ16 6ϕ16	-505 +215	-423 +180	9.32
Type2: M29-N10 +	-635 +282 +174	-508 +226 +266	10ϕ16 3ϕ16	5ϕ16 6ϕ16	-523 +265	-437 +222	9.3 10
Type3: M31-N13 +	-530 +216 +169	-424 +173 +244	2ϕ20+5ϕ16 2ϕ16	4ϕ16 5ϕ16	-423 +208	-355 +175	11.8
Type3: M32-N14 +	-539 +187 +175	-431 +150 +248	2ϕ20+5ϕ16 2ϕ16	4ϕ16 5@16	-423 +220 +280	-355 +180	11.8
Type3: M34-N17 +	-445 +152 +190	-356 +122 +250	2ϕ20+2ϕ16 2ϕ16	3ϕ16 5ϕ16	-351 +165 +280	-293 +135	11.9
Type3:M35-N18 +	-480 +110 +180	-384 +88 +239	3ϕ20+3ϕ16 2ϕ16	4ϕ16 2ϕ16 5ϕ16	-401 +111	-336 +91	11.8
Type3:M37-N21, N22 +	-337 +23 +194	-270 +23 +230	2ϕ20+2ϕ16	2ϕ16 5ϕ16	-280 +112	-227 +90	12
Type3:M38-N22 +	-356 +87 +298	-285 +70 +342	2ϕ20+2ϕ16 3ϕ16	2ϕ16 6ϕ16+3ϕ10	-280 +112 +374	-227 +90	9.4
Type3:M40-N25 +	-232 +200	-186 +235	3ϕ16+1ϕ12 3ϕ16	2ϕ16 4ϕ16+3ϕ12	-196 +246	-159	11.9
Type3: M41-N26 +	-329 +168	-263 +218	2ϕ20+5ϕ16	2ϕ16 4ϕ16	-274 +220	-221	9.3
The transverse reinforcement is 2 hoop: ϕ 8mm @S_w = 112 mm in the critical region ϕ 8mm @S_w = 150 mm outside the critical region							

12.1.3 Capacity design

The column beam connections of the emulated monolith structure are designed and verified using the capacity requirements. The essential of this procedure is to insure the strength required in the columns for the seismic actions of the beams applying weak beam strong column concept. In one direction of the seismic action the top of a beam at the connection will work in tension, and the bottom face will work in compression. As the seismic action is reversed; the seismic force comes from the reverse direction, the same column beam connection functions reverse. The bottom of the beam will work in tension while the top of the beam works in compression.

(1) The exterior column beam connection at node N5 and N8,

The design is carried out using a simple computer program applies equations (5.2 through 5.4) see chapter 5, which based on EC8 regulations. In order to apply these equations the resistance moments of the element at this connection to be calculated using the reinforcement given in Table 12-2. The capacity control of the exterior column beam connection is carried out as follows:

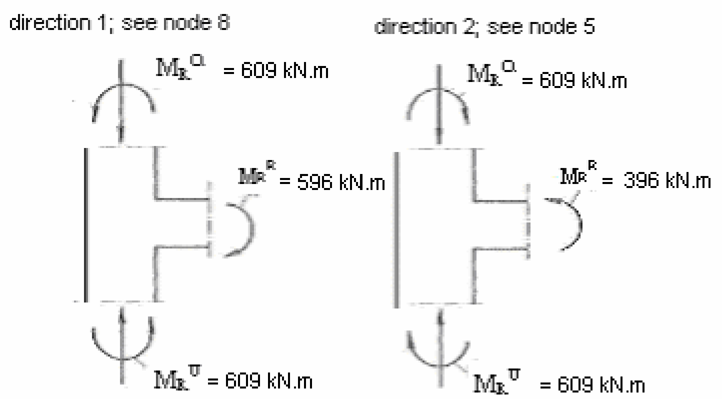


Figure 12-4: Direction of the lading and resistance the column beam connection

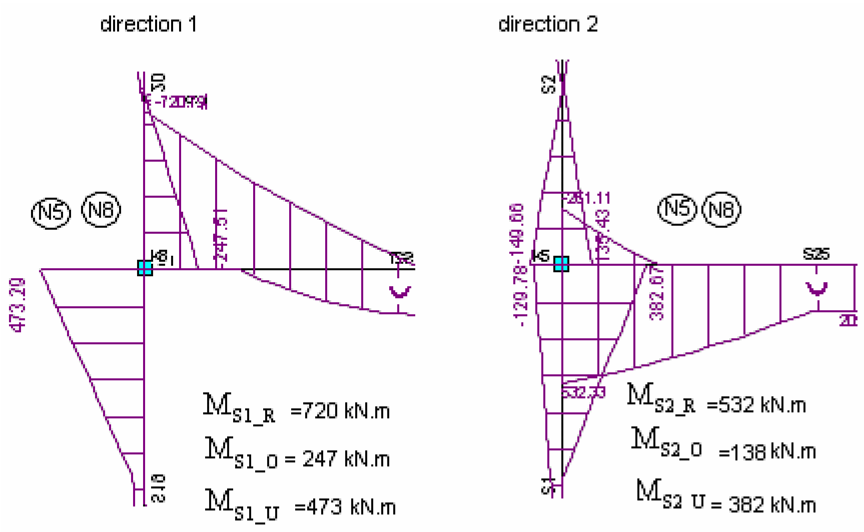


Figure 12-5: Bending moment diagram at node N5 and N8

The column strength multiplication factor and the reversal moment factor may be calculating with moment resistance of the columns and the beam reported in Figure 12-4, and the applied loads given in Figure 12-5.

In direction 1 $\alpha_{cd1} = \gamma_{Rd} \cdot \dfrac{M_{R1\_L} + M_{R1\_R}}{M_{S1\_O} + M_{S1\_U}} = 1.2 \dfrac{0 + 596}{247 + 473} = 0.99$

In direction 2 $\alpha_{cd2} = \gamma_{Rd} \cdot \dfrac{M_{R2\_L} + M_{R2\_R}}{M_{S2\_O} + M_{S2\_U}} = 1.2 \dfrac{0 + 396}{138 + 382} = 0.91$

Using equation (5.2)

The relative importance of the gravitational with respect to seismic load is measured through the so-called moment reversal factor δ as follows:

In direction 1 $\delta_1 = \dfrac{|M_{S1\_R} + M_{S1\_L}|}{|M_{R1\_R}| + |M_{R1\_L}|} = \dfrac{720 + 0}{596 + 0} = 1.21$

In direction 2 $\delta_2 = \dfrac{|M_{S2\_R} + M_{S2\_L}|}{|M_{R2\_R}| + |M_{R2\_L}|} = \dfrac{532 + 0}{396 + 0} = 1.34$

Using equation (5.3)

The required moment resistances of the upper column in direction 1 and in direction 2 for the upper and the lower columns can be obtained using equation (5.4)

For direction 1

$M_{Sd1\_cd} = |1 + (\alpha_{Cd1} - 1)\partial_1| \cdot M_{Sd1} \le qM_{Sd1}$

Over column

$M_{Sd1\_cd} = |1 + (0.99 - 1)1.21| \cdot 274 \le 0.99 \cdot 274 = 271$ kN.m

Under column

$M_{Sd1\_cd} = |1 + (0.99 - 1)1.21| \cdot 473 \le 0.99 \cdot 473 = 468$ kN.m

or direction 2

$M_{Sd2\_cd} = |1 + (\alpha_{Cd2} - 1)\partial_2| \cdot M_{Sd2} \le qM_{Sd2}$

Over column

$M_{Sd2\_cd} = |1 + (0.91 - 1)1.34| \cdot 138 \le 0.88 \cdot 138 = 121$ kN.m

Under column

$M_{Sd2\_cd} = |1 + (0.91 - 1)1.34| \cdot 382 \le 0.88 \cdot 382 = 336$ kN.m

Using equation (5.4)

The required column resistance is

$M_{R2\_O} = M_{R2\_}u = 609$ kN. m < $M_{Sd1\_cd} = 476$ kN.m and that is Ok.

(2) The interior column beam connection at node N6 and N7

The design and the control of the interior column beams connection at the nodes N6 and N7 is carried using the same procedure used in the exterior column beam connection N5 Figure 12-6 Figure 12-7 shows the connected members, and the bending moment diagram at the interior columns connection. This information is used for the capacity as follows:

In our structure because of the symmetry the control is sufficient using one direction of seismic loading on the interior connections, The applied load on the upper and lower column are M_{Sd1}=544 kN.m and M_{Sd1} = 540 kN.m.

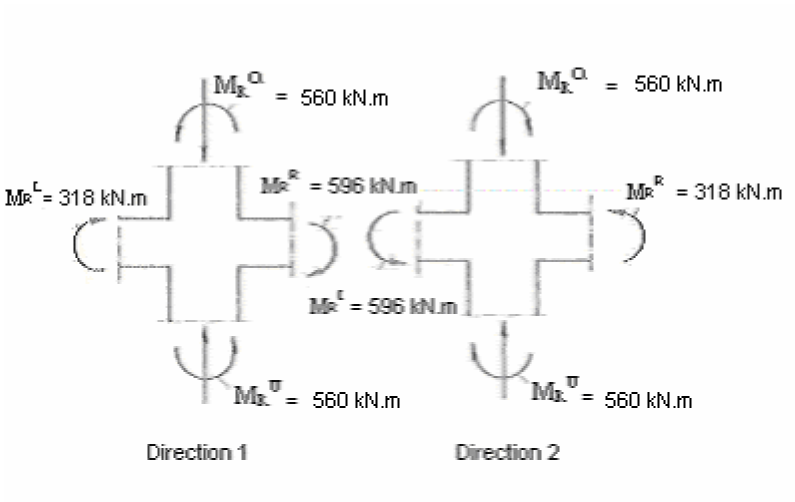


Figure 12-6: Strong column weak beam connection at the interior connection N6 and N7

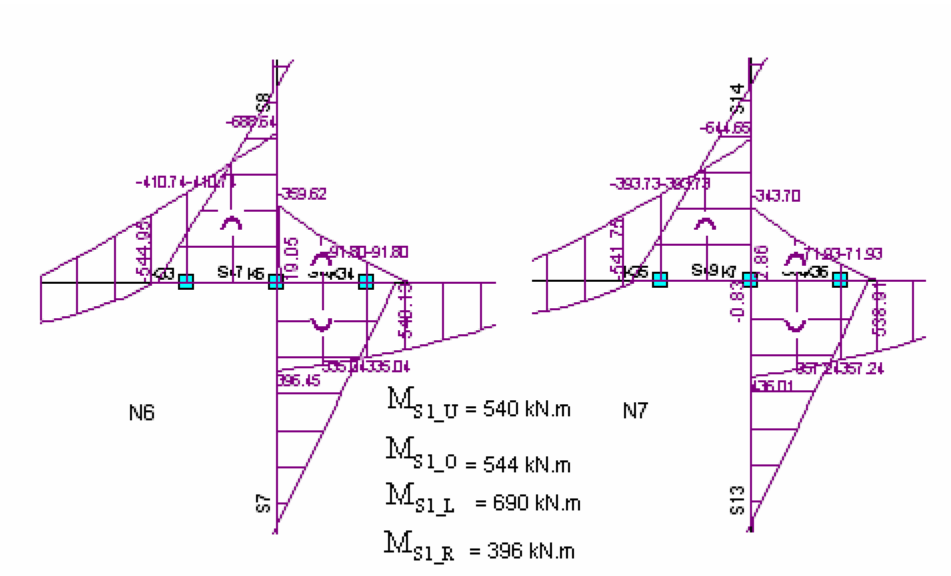


Figure 12-7: Bending moment diagram at node N6 and N7

The column strength multiplication factor is calculated as follows:

In direction 1 $\alpha_{cd1} = \gamma_{Rd} \cdot \dfrac{M_{R1\_L} + M_{R1\_R}}{M_{S1\_O} + M_{S1\_U}} = 1.2 \dfrac{318 + 596}{544 + 540} = 1.01$ Using equation (5.2)

Considering the reversal moment factor using the applied loading as follows:

In direction 1 $\delta_1 = \frac{\left|M_{S1\_R} + M_{S1\_L}\right|}{\left|M_{R1\_R}\right| + \left|M_{R1\_L}\right|} = \frac{\left|396 + 690\right|}{596 + 318} = 1.18$ Using equation (5.3)

The required moment resistance of the upper and the lower column are given as:

For direction 1

Over column: $M_{Sd1\_cd} = \left|1 + (1.01 - 1)1.18\right| \cdot 540 \leq 1.01 \cdot 540 = 546$ kN.m

Under column: $M_{Sd1\_cd} = \left|1 + (1.01 - 1)1.18\right| \cdot 544 \leq 1.01 \cdot 544 = 550$ kN.m

Using equation. (5.4)

The available resistance moments of the interior columns in both directions:

$M_{R2\_O} = M_{R2\_u} = 560$ kN.m.

The resistance moment of the columns is greater than the required design moment. And that is ok.

Conclusion

The columns are designed using reinforcement, according to the requirements at the critical regions. The beams do not exert extra loading which leads to increase demands on column strength. And that is due to the proper dimensioning of the beams.

12.2 Optimize the design

The design of the concrete structure is optimized using the elastic analysis model with modified elastic stiffness. The modified effective stiffness model is built based on the elastic frame model (A) reinforcement. The stiffness at the column beam connections are introduced using the stiffness bending moment relations Figure 12-9 and after the consideration of the yield locations at the first floor.

The resulting bending moment, at the column beam connections with modified stiffness model is about 80% of that which is obtained by the elastic model results (Figure 11-9). The design reinforcement in the beams and the columns are identical to the elastic model reinforcement. Further modification for the columns dimensions within this model leads to a reduction in the ground and first floor columns so that the column dimensions are 500mm×500mm. In the following the design of the concrete structure is introduced

12.2.1 The ground floor design concept

In the most building the ground floor columns are longer than the other floors columns. And the first floor is exerted to higher lateral seismic force than the other floors. That required heavier beams at the first floor. In order to achieve reasonable dimensioning proportionality for the beams and especially the usage of similar ductile members in the joint in the prefabricated construction, it is reasonable to design with redistribution of the forces by introducing flexibility in the first floor connections. And reduces the effective column length. In our project this solution is applied with the yield locations at the first floor and using different degree of fixation for the exterior and the interior columns. The yield locations in the first floor beams are applied in places that allow the reinforcement to yield due to the seismic loading and acts in compression by the reversal loading refer to Figure 12-8 and Figure 11-2. It provides flexibility at the first floors that it obtains reduced depth. The analysis shows better distribution of the loading among the concrete structure.

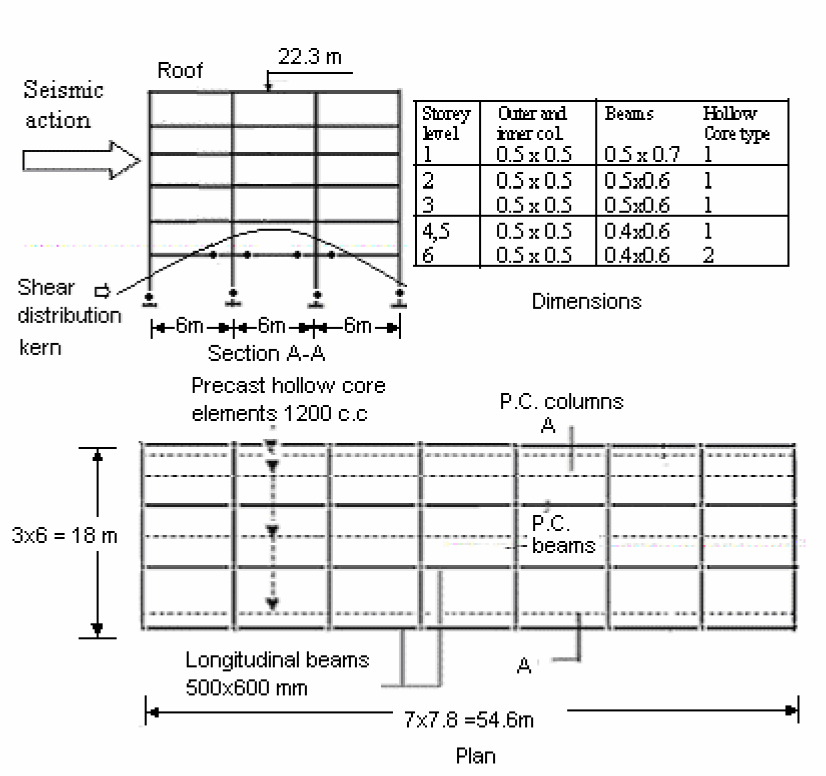


Figure 12-8: The six storey building, geometry, columns and beam dimensions

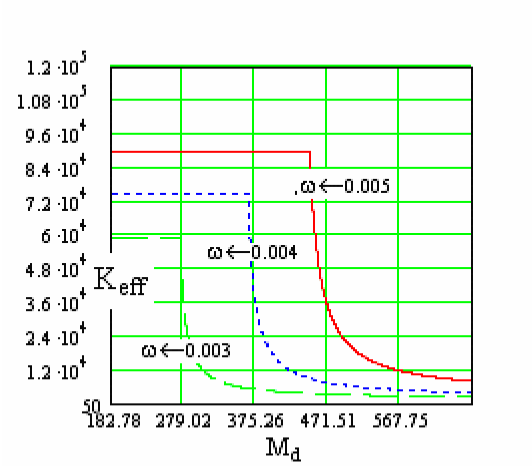


Figure 12-9: The connection and section effective stiffness of the beam end, versus the applied moment, for reinforced concrete section of 700x500mm.

12.2.2 Strength design of the columns

The columns are designed of reinforced concrete B55, using the interaction diagram CEB 1982.The critical regions verified for the minimal reinforcement ratio. The capacity design principles are applied for the seismic beams action. That was applied for the most sever columns actions at the exterior and the interior connections. It have been seen that no big influence rather than the design requirements due to the acting loading was found. That was because of the proper selection of the elements and its dimensions proportions. The redistribution of the loading due to the good estimation of the stiffness at the connections has a positive influence for the reduction of the forces exerted on the connection joints. The moment curvatures of the columns have been verified for the yielding and ultimate limits, and for the available ductility. That gave a positive result without more complexity in the reinforcement details. This detail will be investigated through the Ruaumoko program for the degradation and the influence of the yielding location if it can be eliminated and changed to a fixed connection at the base.

12.2.2.1 Calculate the reinforcement in the columns

The loading on the columns is identical to elastic model loading. The flexural and the transverse reinforcement is verified using same procedure followed in section (12.2.2.1.) In all the columns the minimum reinforcement is applicable at the critical region with a length of 0.75m, and It may be used the same reinforcement outside the critical regions (Figure 12-10).

The Columns yield locations

The yield location of the columns in the building is at the base of the ground floor columns. In this section we control the columns yield locations. This control includes the exterior and the interior columns at the nodes N1, N2, N3 and N4 (Figure 12-1).

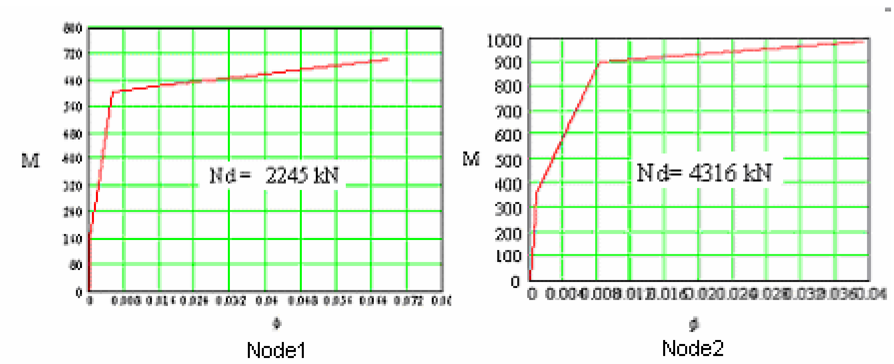


Figure 12-10 Moment curvature for the inside and outside columns at Node N1 and N2

In order to conform the column ductility requirements, the consideration of the confinement concrete is important. Typically spalling of confined concrete initiated at extreme fiber compression strain between 0.006 and 0.01 (Mander et al). The steel strain limits has been determined to insure the residual crack width do not exceed 1.0mm. Consider that the cracks in the plastic hinge region in the columns at 200mm. The cracks width will be 200×0.015 = 3 mm, and the residual cracks is 1/3 and that will be 1mm.Apply the equilibrium equations (7.9) and (7.10), we find the moment curvature of the column M43 at the locations N1 and N29, and that is applicable for the exterior columns at the other side. Figure 12-10, shows the moment verse the curvature at these locations. The applied moment and the reinforcement of the exterior columns are shown in Table 12-3.

Table 12-3: Design and resistance moments [kN.M], reinforcement area [mm^2] for the exterior and interior columns in the critical regions.

<table>
<tr><td colspan="2">Column</td><td>Node</td><td>N_d</td><td>M_d</td><td>A_s</td></tr>
<tr><td rowspan="2">Solution with modification</td><td>M43, M1</td><td>N1
N5</td><td>2274
2274</td><td>418
339</td><td rowspan="2">12ϕ16+4ϕ20=3668 mm^2
(total reinforcement)</td></tr>
<tr><td>M44, M7</td><td>N2
N6</td><td>2290
2290</td><td>594
492</td></tr>
<tr><td colspan="5">All other critical regions of the columns</td><td>8ϕ16+4ϕ20=2856 mm^2
(total reinforcement)</td></tr>
</table>

12.2.3 Design of the beams

The calculations of the flexure and shear resistance reinforcement have been carried out using equations (12.6) through (12.8). The reinforced concrete beams are verified at the yield locations. The control used the nonlinear analysis as it is given in chapter (7), considering the ultimate and the yield conditions of the concrete sections (Figure 12-11 and Table 12-5).

The members are verified to full fill the ductility requirements. In the first assessment for the ductility, we may obtain the global ductility of the structure using equation (12.6), and according to EC8, as estimation of the global ductility. The global ductility of the structure in general is for the top sway displacement in the case of frame, evaluated at the ultimate and the yield conditions. It have been noted that the ductility it self does not represent a sufficient parameter with to estimate the structures damage. There fore a more complete characteristic of the inelastic response of the structure requires the introduction of the damage parameters, such as the cyclic ductility, historian ductility.

The local ductility of the member may be obtained from the structural geometry relations, through solving equations or using a structural program. In our building and due to the symmetry in the beams span lengths and height, the local ductility is assumes approximately equal to the global ductility .The local ductility have been verified finally, using the nonlinear dynamic analysis program Ruaumoko. The beams and the column should be verified according to the capacity requirements in section (12.3).

$\mu_\varphi = 2q - 1 \qquad \text{if } T_1 \leq T_c$

$\mu_\varphi = 1 + 2(q - 1)\cdot T_c/T_1 \quad \text{if } T_c < T_1$ Using equation (12.6)

The fundamental period of the structure building is T1 =0.783 s. using equation (11.7)

The parameter of the elastic response spectrum T_c depends on the soil class. And according to the EC8, and for soil class B, where our building is assumed, Tc = 0.5, and q =3.9 using equation (12.6), the local ductility is calculated as follows:

$$\mu_\phi = 2 \times 3.9 - 1 = 6.8$$

The required flexure reinforcement area is calculated as follows:

$$A_s = \frac{M_d}{0.9 \cdot d \cdot f_s} \qquad \text{Using equation (12.7)}$$

The shear is calculated as follows:

$$V_{Rd} = V_{cd} + V_{wd} \qquad \text{Using equation (12.8)}$$

Where V_{cd} is the concrete shear resistance and V_{cd} is the shear resistance by the web reinforcement

The design shear of the beams calculated considering possible plastic hinge formation at the beam-ends as follows:

$$\text{Shear strength} \geq \gamma \cdot \frac{M_{p1} + M_{p2}}{l_p} + \frac{W}{2}$$

With: $W = (DL + 0.15 \cdot LL) \times L_{beam}$ Using equation (5.6)

Where $M_p = M_u$, and M_u are reported in Table 12-5 for each beam end. The design shear reinforcement of the elastic model is applied, because there is no big difference in the summation of the beam ends resistance moment between the elastic and the modified stiffness model.

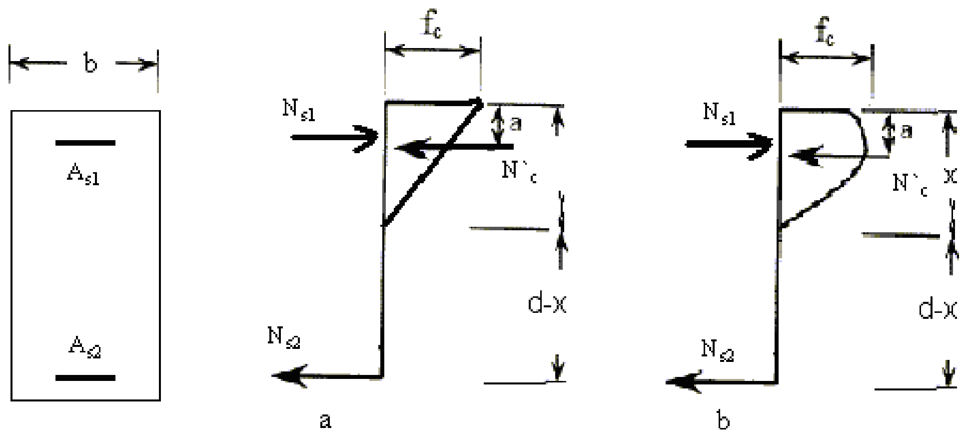


Figure 12-11: Beams section, with stress strain diagram at yield and ultimate state

Design of the column beams connection

In this section we design and control the connection system at the nodes N6 and node N7, refer to Figure 12-1. The column beam connection at N6 is semi rigid connection and the yield location is at the nodes N33 and N34. The required reinforcement is obtained using the equations (7.7) through (7.9). This reinforcement is verified applying the essential principles of the capacity design. That means we chose reinforcement proportions at these nodes in such a manner that leads to the delay of the yield formation at the column beam connection. The yield at the nodes N33 and N34 will dominate. The design process is carried out as follows:

(1) Design the cross sections at these locations and chose the reinforcement according the loading requirements.
(2) Control the yield moments at these locations consider an over strength factor of 1.2 to insure that the yielding occurs in the beams M47 and M48 at nodes N33 and N34 before node N6 (Table 12-4).

This procedure will be applicable for the cluster nodes N7, N35 and N36, due to the structural symmetry.

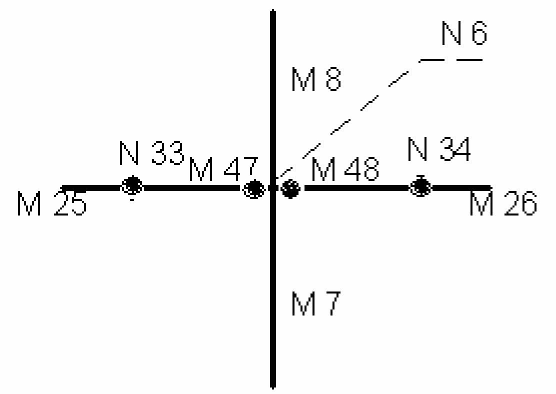


Figure 12-12: The connection at node N6

The non-linear section behaviour solution is carried out using equations (7.1) through (7.10). The reinforcement of the beam M47 and M48 is reported in Table 12-4.

Table 12-4: Loads, reinforcement at the beam column connection N6 and N33.

Member-Node		M_d [kN.m]	Steel area [ϕmm]		M_u [kN.m]	M_y [kN.m]
			A_{st}	A_{sb}		
M47, M48 700×500mm	N6	-626 +390	2ϕ16+5ϕ20	6ϕ16	-632 +383	-474 +315
	N33	-295 +390	3ϕ16+1ϕ12	4ϕ16+1ϕ12	-230 +294	-187 +240

Verify that the yield moment at the yield location N33 is less than the yield moment at the column beam connection N6.considering the over strength factor and the proportion of the applied loads as follows:

$$^{-}M_{y\_location} \leq \frac{295 \cdot \frac{474}{1.15}}{626} = 192 \text{ kN.m} \quad \text{and that is ok}$$

$$^{+}M_{y\_location} \leq \frac{360 \cdot \frac{315}{1.15}}{390} = 252 \text{ kN.m} \quad \text{and that is ok} \qquad \text{Using equation (12.13)}$$

Capacity design, for the optimised solution

The design moments at of the beams are identical to that of the elastic model the dimensions interior columns are designed with 500×500 mm. The capacity design at the column beams connections are carried out with the new columns dimensions and no extra modifications in the reinforcement are needed.

Conclusion

The columns are designed using the minimum reinforcement, according to the requirements at the critical regions. The beams do not exert extra shear loading which leads to increase in the column stiffness. And that is due to the good design of the beams at the connections, and the use of semi rigid column beams connections.

The modified elastic stiffness model provides better estimation for the displacement of the structure, where the rotations of the beams ends can be verified considering the material properties and the reinforcement. For the modified model, a reduction factor is applied to the beam stiffness of the elastic one. The beams reinforcement of the concrete structure is reported for the structural optimisation with the modified stiffness model is reported in Table 12-5.

The ductility control indicates that modifications in the roof beams are required, where they have been applied with 0.3×0.6m instead of 0.4×0.6 m, the building design is modified for the both cases of the elastic model and the modified stiffness model. The resulting reinforcement is identical for the previous design and no further needs for the capacity design. The final design reinforcement of the building beams are reported in Table 12-6.

Table 12-5: The optimized reinforcement with emulated monolith B55 R/C elements. The design loading is the results of the modified elastic model.

Type: Mem. Node	Loading [kN.m]	Flexure reinforcement [ϕmm]		Moment [kN.m]		Ductility
	$-M_d$ $+M_{d\_}$	A_{st}	A_{sb}	M_u	M_y	μ_ϕ
Type1:M25-N5 +	-520 +446 +358	4@20+2ϕ16 3ϕ16	2ϕ20+4ϕ16 6ϕ16+3ϕ10	-531 +448	-448 +373	9.20
1: M47,M8-N33 N6,N7	-295 +360 -626 +390	3ϕ16+1ϕ12 2ϕ16+5ϕ20	4ϕ16+1ϕ12 6ϕ16	-230 +294 -632 +383	-187 +240 -474 +315	9.70
Type1: M26 N34 +	-334 +375 +289	6ϕ16	6ϕ16 5ϕ16	-387 +387 +318	-315 +315	9.44
Type 2: M28-N9 +	-442 +332 +405	6ϕ16+2ϕ20 3ϕ16	3ϕ16+2ϕ20 8ϕ16	-447 +322	-340 +271	9.32
Type2 :M29-N10 +	-457 +346 +402	9ϕ16+2ϕ20 3ϕ16	6ϕ16+ϕ12 8ϕ16	-447 +346	-340 +290	9.3
3 : M31-N13 +	-404 +274 +388	2ϕ20+3ϕ16+ 3ϕ12 3ϕ16	5ϕ16 7ϕ16	-413 +274	-349 +221	11.87
Type3: M32-N14 +	-405 +277 +385	2ϕ20+3ϕ16 +2ϕ12 3ϕ16	5ϕ16 7ϕ16	-379 +274	-317 +221	11.87
Type3:M34- N17 +	-364 +204 +404	4ϕ20+2ϕ16+ 2ϕ12 3ϕ16	6ϕ16 8ϕ16	-486 +316	-409 +265	11.8
Type3:M35-N18 +	-357 +210 +298	4ϕ20+3ϕ16 3ϕ16	4ϕ16+1ϕ12 6ϕ16	-480 +250	-409 +203	11.76
Type3:M37,M38 N21,N22 +	-353 +135 +304	2ϕ20+3ϕ16 3ϕ16	2ϕ16+1ϕ12 7ϕ16	-323 +141	-270 +120	11.9
Type3:M40-N25 +	-206 +30 +202	4ϕ16+1ϕ12 2ϕ16	2ϕ16+1ϕ12 4ϕ16	-250 +141	-203 +120	10
Type3:M41-N26 +	-220 +30 +168	3ϕ16+3ϕ12 2ϕ16	2ϕ16+1ϕ12 4ϕ16	-260 +141 +220	-208 +120	10.5
The transverse reinforcement is 2 hoop: ϕ 8mm @S_w = 112 mm in the critical region ϕ 8mm @S_w = 150 mm outside the critical region						

Table 12-6: The optimized reinforcement with emulated monolith B55 R/C elements, The design loading are the results of the modified elastic model after correction of the roof beams

Type: Member-Node	Loading [kN.m]	Flexure reinforcement [ϕmm]		Moment [kN.m]		Ductility
	$-M_{d\_top}$ $+M_{d\_bottom}$	A_{st}	A_{sb}	M_u	M_y	μ_ϕ
Type1: M25-N5 +	-550 +470 +360	4@20+2ϕ16 + 1ϕ12 3ϕ16	2ϕ20+3ϕ16 7ϕ16	-568 +396	-448 +321	9.20
Type1:M47,M48 -N33 N6,N7	-295 +360 -630 +390	3ϕ16+1ϕ12 2ϕ16+5ϕ20 +1ϕ12	4ϕ16+1ϕ12 6ϕ16	-230 +294 -632 +420	-187 +240 -474 +344	9.70
Type1:M26-N34 +	-334 +375 +289	6ϕ16 3ϕ16	6ϕ16 5ϕ16	-387 +387	-315 +315	9.5
Type 2: M28-N9 +	-447 +375 +405	6ϕ16+2ϕ20 3ϕ16	4ϕ16+2ϕ20 7ϕ16	-447 +377	-340 +314	9.32
Type2: M29-N10 +	-450 +350 +402	9ϕ16+2ϕ20	6ϕ16+ϕ12 2ϕ20+5ϕ16	-447 +346	-340 +290	9.3
Type3: M31-N13 +	-413 +275 +302	2ϕ20+3ϕ16+ 4ϕ12 3ϕ16	5ϕ16+1ϕ12 6ϕ16	-437 +302	-380 +221	11.9
Type 3 / M32-N14 +	-405 +265 +385	2ϕ20+3ϕ16 +3ϕ12 3ϕ16	5ϕ16+15ϕ12 7ϕ16	-407 +302	-342 +240	11.9
Type3:M34 -N17 +	-386 +215 +360	4ϕ20+2ϕ16+ 2ϕ12 3ϕ16	4ϕ16 7ϕ16	-380 +220	-317 +180	11.8
Type3: M35-N18 +	-357 +210 +360	4ϕ20+3ϕ16 3ϕ16	4ϕ16+1ϕ12 7ϕ16	-480 -250	-409 -203	11.8
Type3:M37,M3-N21,N22 +	-353 +135 +305	2ϕ20+3ϕ16 +1ϕ12 3ϕ16	2ϕ16+1ϕ12 6ϕ16	-351 +141	-293 +115	11.9
Type3: M40-N25 +	-213 +30 +256	4ϕ16 2ϕ16	2ϕ16+1ϕ12 5ϕ16	-220 -141	-180 +120	10
Type3:M41-N26 +	-220 +30 +256	3ϕ16+3ϕ12 2ϕ16	3ϕ12 5ϕ16	-260 + 94	-208 + 120	10.5
The transverse reinforcement is 2 hoop: ϕ 8mm @S_w = 112 mm in the critical region ϕ 8mm @S_w = 150 mm outside the critical region						

12.3 Nonlinear control

Nonlinear analysis for the concrete structure of the building is carried out using the Ruaumoko program. The non-linear analysis model is given in (Figure 12-13). The geometry and the columns and beams characteristics are obtained using the design reinforcement for the different solutions. The structure is tested with two different acceleration records; El Centro May 1940 North-South Component, and Bucharest 1977, North-South component (Figure 12-14), scaled to the equivalent base shear force of ground acceleration $A_g = 0.24g$.

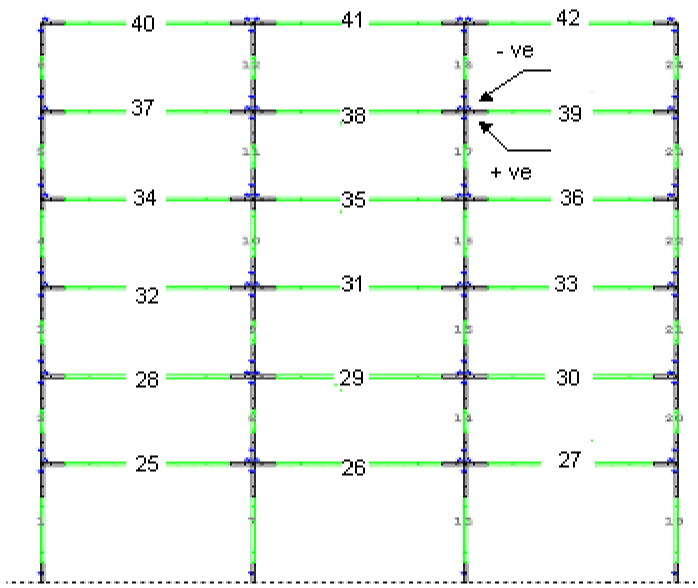


Figure 12-13: Concrete structural model for nonlinear analysis in Ruaumoko program

The six-storey building Ruaumoko model has the following characteristics

- The beams are modelled with Giberson one component beam, consisting of plastic hinge at the end of the beams and an elastic member in between.
- All beams are modelled with Bi-linear –hysteresis for the ductility with controlled analysis.
- The structure is verified for the degradation. All the beams are modelled with Takeda degrading stiffness hysteresis rule. Which is used to model plastic hinges in reinforced concrete beams.
- The ground floor columns are modelled for the capacity design principles and using Takeda degrading rule.
- Raleigh initial stiffness damping with 5% in mode 1 and mode 10
- 15 seconds of excitation with a time-step of 0.01 seconds

The analyses results for the important members are given in Table 12-9. In this table the demand ductility of the members, are for the beams, which vary from 1 to 5.6, while the available ductility in the members varies from a minimum value of 9 to 11.9, which means that the beams fulfils the ductility requirements.

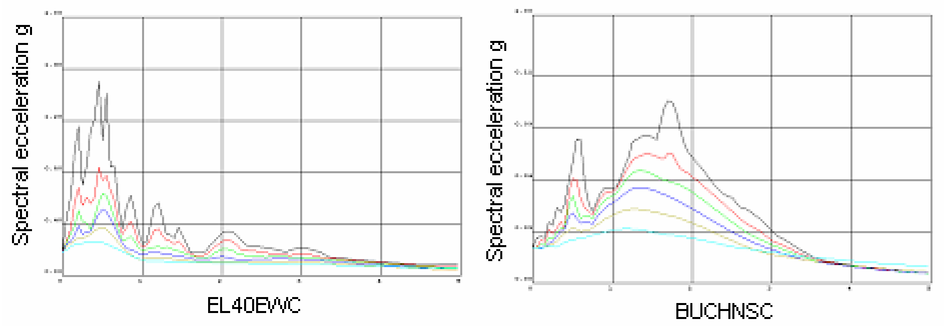


Figure 12-14: Spectral acceleration records, El40elwc and Buchnsc

The positive and the negative ductility envelop at the beam-ends for two different models are reported in Table 12-7, the first group information is for building with columns of 600×600 mm and the second group is for columns of 500×500 mm.

Table 12-7: The ductility of the structure with two different column sizes

600x600 mm ground and first floor columns			500x500 mm columns		
mem.	+ve	-ve	mem.	+ ve	- ve
25	4,57	4,16	25	6,55	4,69
26	5,50	3,52	26	7,84	4,70
27	3,86	5,74	27	6,00	7,57
28	5,97	3,64	28	8,15	4,54
29	5,36	3,79	29	7,34	4,75
30	5,44	3,43	30	7,48	4,30
31	4,79	3,65	31	6,22	4,22
32	4,87	3,71	32	6,25	4,38
33	4,90	3,78	33	6,23	4,53
34	4,44	2,53	34	4,75	2,71
35	4,28	2,63	35	4,74	2,81
36	4,31	2,66	36	4,92	2,84
37	1,55	3,19	37	1,04	2,75
38	1,97	3,37	38	1,48	3,02
39	2,05	3,37	39	1,58	3,10
41	0.000I	2,14	42	0.000E	1,77

Using Bilinear- hysteris

The negative reinforcement at the supports does yield enough; the ductility demand in these members is low when it is compared with the reinforcement at the positive moments, which indicates that the sizes of the columns can be reduced to increase the ductility demand. The displacement should be verified so that the drift ratios of the members are within acceptable limits < 3%.

The structure is modified and designed using 500mm×500mm columns. The obtained members ductility are shown in Table 12-7. The ductility of the structure elements are non-regular distributed, and some of them are greater than 6. The bottom reinforcement participates in yielding more the top reinforcement. The roof beams doses not share the other beams in yielding. For better moment redistribution on in the structure modifications in the beam width is required (Figure 12-15). For the optimisation of the structure building a reduction in the last floor beams width is carried out. The structure is designed with modified stiffness model section (12.2). The design results are verified with the Ruaumoko program. The ductility of the elastic stiffness model and the modified stiffness model is reported in Table 12-8.

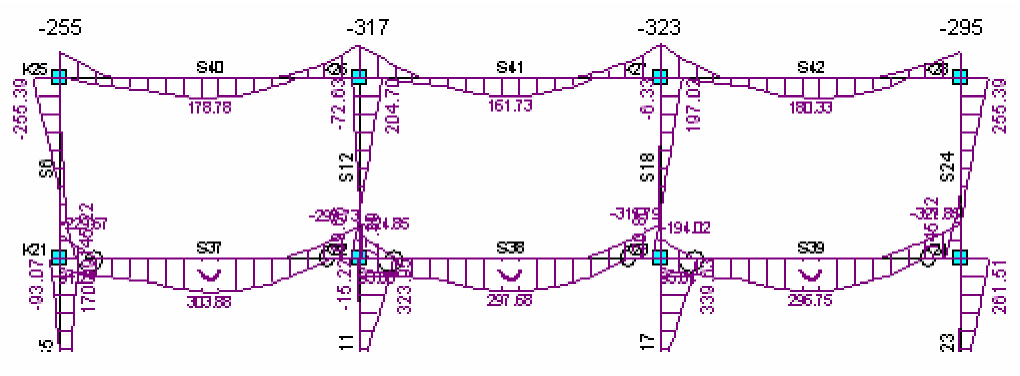


Figure 12-15: B.M. diagram after changing the roof beams dimensions.

Final nonlinear analyses of the concrete structure after the modifications in the roof beams width is carried, the ductility and the ductility proportion are reported in Table 12-8 . The ductility demand in the modified stiffness is reduced in the positive moment and increased in the negative moment.

Table 12-8: Demand ductility of the reinforced concrete end beams, design with elastic mode and design with modified elastic stiffness connections.

A				B1				B2			
mem	+ve	-ve	+/-	mem	+ve	-ve	+/-	mem	+ve	-ve	+/-
25	4.5	4.1	1.1	25	5.3	5.3	1.04	25	4.8	4.7	1.03
26	5.7	4.2	1.36	26	5.8	5.3	1.08	26	5.4	4.7	1.13
27	5.8	4.2	1.38	27	5.9	5.4	1.1	27	5.3	4.9	1.08
28	6.3	3.4	1.82	28	4.2	5.7	0.73	28	3.8	4.4	0.86
29	5.1	3.6	1.43	29	4.3	5.9	0.73	29	4.2	4.5	0.98
30	5.1	3.7	1.39	30	4.7	5.9	0.8	30	4.2	4.3	1.0
31	5.0	4.2	1.2	31	5.1	4.8	1.07	31	4.2	4.5	0.98
32	5.1	4.2	1.2	32	5.1	4.9	1.05	32	4.5	4.5	0.99
33	5.13	4.3	1.2	33	5.0	5.0	1.0	33	4.6	4.6	1.02
34	6.5	3.7	1.74	34	4.3	3.9	1.09	34	2.9	2.9	1.01
35	6.4	3.8	1.69	35	4.4	4.1	1.06	35	3.1	3.0	1.02
36	6.4	3.9	1.64	36	4.5	4.2	1.08	36	3.1	3.0	1.02
37	6.2	5.7	1.1	37	4.9	3.5	1.4	37	2.1	2.0	1.0
38	6.8	5.9	1.17	38	5.4	3.7	1.49	38	1.2	2.2	1.0
39	6.9	5.8	1.19	39	5.5	3.7	1.47	39	1.2	2.4	0.5
40	2.1	2.6	0.82	40	1.2	2.6	0.5	40	0	2.0	0.0
41	2.1	2.7	0.79	41	1.1	3.1	0.36	41	1.7	2.2	0.8
42	1.7	4.0	0.42	42	1.3	3.2	0.41	42	0	1.8	0.0

A design with elastic model
B modified effective stiffness model
B1 substitute the design moments, after steel selection
B2 substitute the analysis results, no steel selection

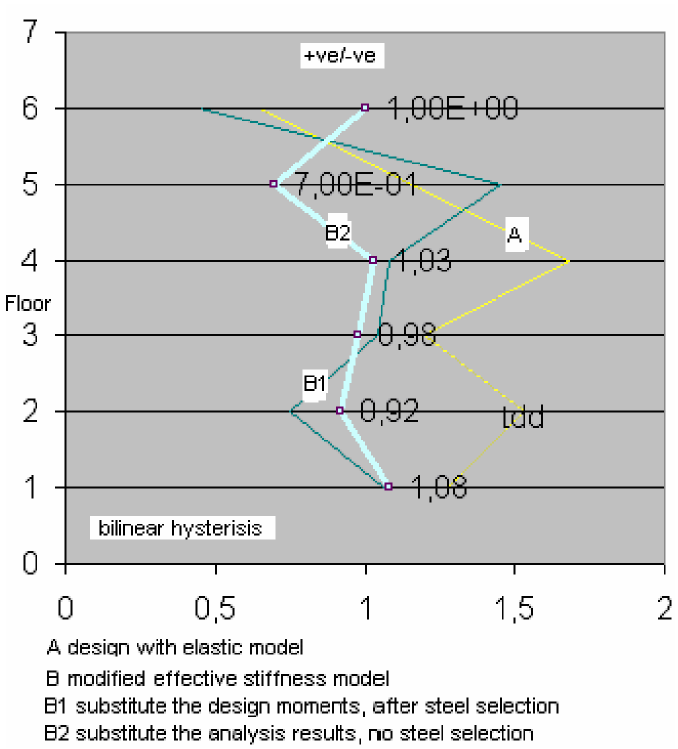


Figure 12-16: Ductility ratio (+ve /-ve) of the beam ends at each floor

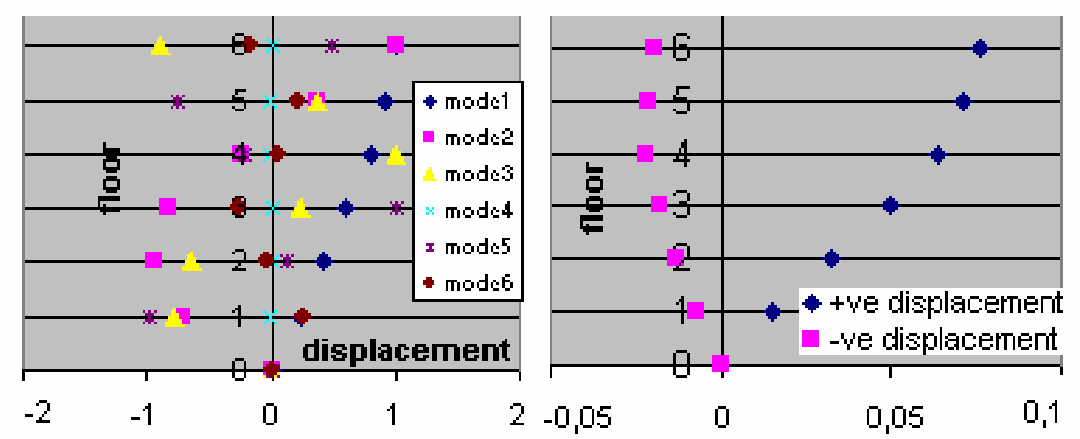


Figure 12-17: Mode shape and maximum relative displacement, for the elastic model, bilinear hysteresis

Table 12-8 may be read with the following description:

- A is related to the elastic model
- B is related to the modified stiffness model as follows:

(1) B1 is for the design with the modified elastic model, after the substitution of the plastic moments in the Ruaumoko program

(2) B2 is for the substitution of moments from the analysis results of the modified elastic model in the Ruaumoko.

It is recognised that the reinforcement results are identical. As an advantage the modified model have regular ductility for the positive and negative moments at the member ends it means that the member's ends shares the ductility demand. In the design with modified elastic stiffness there is more +ve reinforcement and less -ve reinforcement, the positive moments at the members ends participate in the seismic resistance. That leads to decrease the demand ductility in general, due to the balance interaction in the member's ends moments. The selection of the reinforcement plays a role for the decision of the response. Some technical solution to avoid formation shear hinges at the columns face is to invoke the formation of the yield locations at distances from the column face by reduction of the reinforcement. An important notice is that the top floors, and the roof do not yield enough, and that is due to the application of the design according to the equivalent shear method, this method based essentially on the first mode of vibration. The roof is designed with reinforcement enough for its serviceability and the seismic load proportion is lower than the other floors, and that leads to that the roof required les ductility demand.

Final design

The structure is designed with the modified stiffness model. In this final analysis, only yield locations at the columns faces were enabled (Figure 12-18 B with effective stiffness K_{eff}). The analysis results and the reinforcement are given in Table 12-9.
In the final design all the nonlinear analysis aspects are accounted for, including degradation with the Takeda hysteresis model.

The design is controlled for the nonlinear analysis using Ruaumoko. Ductility and the ductility ratio at each floor are given in Figure 12-20 and in Figure 12-21 for the Ruaumoko model with rigid columns connections at the ground floor.

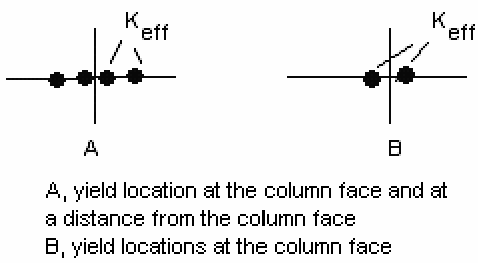


Figure 12-18: Yield locations at the first floor beams

Table 12-9: Analysis results and the beams reinforcement, design with modified effective stiffness according to Figure 12-18

Type: Member-Node	Loading [kN.m]	Flexure reinforcement [ϕmm]		Moment [kN.m]		Ductility
	$-M_{d\_top}$ $+M_{d\_bottom}$	A_{st}	A_{sb}	M_u	M_y	μ_ϕ
Type1: M25-N5 +	-524 +435 +407	4@20+2ϕ16 3ϕ16	2ϕ20+4ϕ16 8ϕ16	-532 +448	-434 +373	11.8
Type1: M4, M48 -N38 N6,N7	-295 +360 -666 +514	3ϕ16+1ϕ12 2ϕ16+5ϕ20 +1ϕ12	4ϕ16+1ϕ12 6ϕ16	-230 +294 -632 +420	-187 +240 -474 +344	11.7
Type1: M26 +	+387	3ϕ16	7ϕ16			
Type2: M28-N9 +	-480 +350 +394	5ϕ16+3ϕ20	4ϕ16+2ϕ20 8ϕ16	-505 +377	-423 +314	11.7
Type2: M29-N10 +	-465 +350 +402	5ϕ16+3ϕ20 3ϕ16	6ϕ16+2ϕ12 8ϕ16	-505 +374	-424 +314	11.7
Type3: M31-N13 +	-434 +282 +302	2ϕ20+3ϕ16+ 4ϕ12 3ϕ16	5ϕ16+1ϕ12 6ϕ16	-437 +302	-380 +221	11.87
Type3: M32-N14 +	-412 +273 +372	2ϕ20+4ϕ16 +2ϕ12 3ϕ16	5ϕ16+15ϕ1 2 7ϕ16	-429 +302	-360 +240	11.87
Type3: M34-N17 +	-385 +217 +365	4ϕ20+2ϕ16 +1ϕ12 3ϕ16	4ϕ16 7ϕ16	-401 +220	-336 +180	11.8
Type3: M35-N18 +	-354 +217 +360	2ϕ20+4ϕ16+ 1ϕ12 3ϕ16	4ϕ16+1ϕ12 7ϕ16	-401 +250	-336 +203	11.8
Type3: M37,M38- N21,N22 +	-383 +110 +362	2ϕ20+4ϕ16 +1ϕ12 3ϕ16	2ϕ16+1ϕ12 7ϕ16	-401 +141	-336 +115	11.9
Type3: M40-N25 +	-249/+50 +281	4ϕ16 2ϕ16	2ϕ16 5ϕ16	-220 +112	-180 +91	10
Type3:M41-N26 +	-203 +50 +285	3ϕ16+2ϕ12	2ϕ16 5ϕ16	-226 + 112 +274	-185 + 91	9.2
The transverse reinforcement is 2 hoop: ϕ 8mm @S_w = 112 mm in the critical region ϕ 8mm @S_w = 150 mm outside the critical region						

Non-linear analysis, with rigid connection at the ground floor columns

The demand ductility is lower for the design with the modified stiffness model than the designed with elastic model, and the ductility proportion is more regular. The positive and negative reinforcement at the columns beams connections participates, in the seismic resistance. The results in (Figure 12-22) indicate that the lower parts of the structure require more ductility demand. That comes from the considered distribution of the lateral forces according to the equivalent shear method, which solves the problem in principle according to the first mode action. It may conclude that the design with modified stiffness model results in a structure resist able to resist higher seismic actions with identical reinforcement used in the elastic mode.

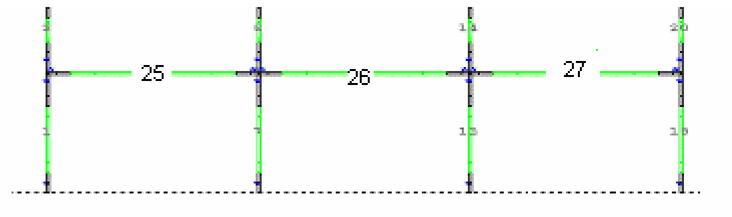


Figure 12-19: Ruaumoko model, ground floor columns with connection

Table 12-10: Ductility demand of the structural concrete members, designed with elastic stiffness (A), with modified stiffness (B) with modified stiffness model (C) with member strength identical to the analysis results, and apply the degradation. All models have the same geometry and member

	(A)			(B)			(C)		
mem.	+ve	-ve	+/-	+ve	-ve	+/-	+ve	-ve	+/-
25	6,54	6,45	1,01	7,36	7,39	1,00	6,81	7,1	0,96
26	8,69	6,47	1,34	8,76	7,36	1,19	7,45	7,1	1,04
27	9,09	6,28	1,45	9,21	7,51	1,23	7,29	8,4	0,86
28	9,80	5,21	1,88	4,76	7,62	0,63	4,51	5,4	0,83
29	8,11	5,38	1,51	6,82	8,59	0,79	5,43	6,1	0,89
30	8,07	5,57	1,45	6,46	8,38	0,77	5,35	6,1	0,88
31	7,43	6,51	1,14	3,63	5,16	0,70	5,67	5,7	1
32	7,39	6,74	1,10	3,64	5,50	0,66	5,48	5,9	0,92
33	7,40	6,79	1,09	3,62	5,79	0,63	5,65	6,1	0,92
34	5,95	4,10	1,45	0,27	1,74	0,16	1,59	3,3	0,48
35	5,47	4,42	1,24	0,23	1,98	0,12	2,14	3,6	0,6
36	5,60	4,45	1,26	0,18	2,06	0,09	2,37	3,5	0,67
37	1,85	6,01	0,31	0,03	2,61	0,01	1,85	1,9	0,98
38	3,88	2,73	1,42	0,83	3,50	0,24	2,1	2,4	0,87
39	3,48	6,57	0,53	0,87	3,80	0,23	2,1	2,6	0,81
40	0,05	1,99	0,02	0,00	1,52	0,00	0	2,6	0
41	0,00	2,03	0,00	0,00	1,80	0,00	0	2,7	0
42	0,00	3,73	0,00	0,00	1,91	0,00	0	2,7	0

A) Design with elastic model
B) Design with modified model
C) Design with member strength identical to the analysis results
Takeda hysterisis

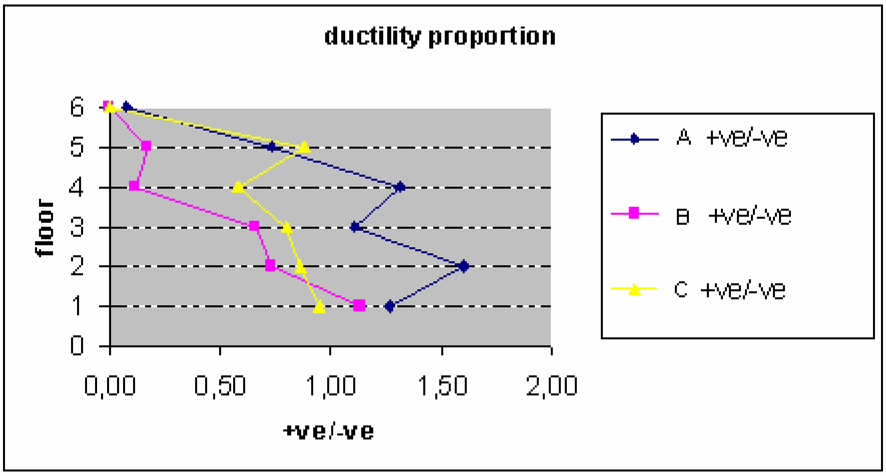


Figure 12-20: Ductility proportion

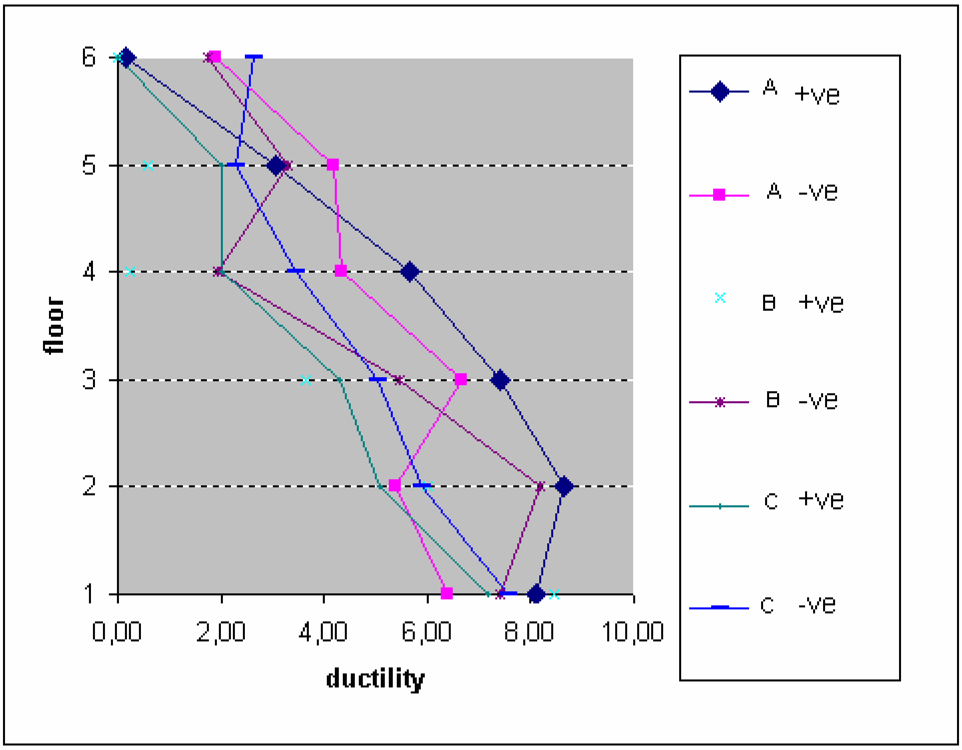


Figure 12-21: The ductility of the structural member at each floor, designed with elastic model, modified model and modified stiffness model.

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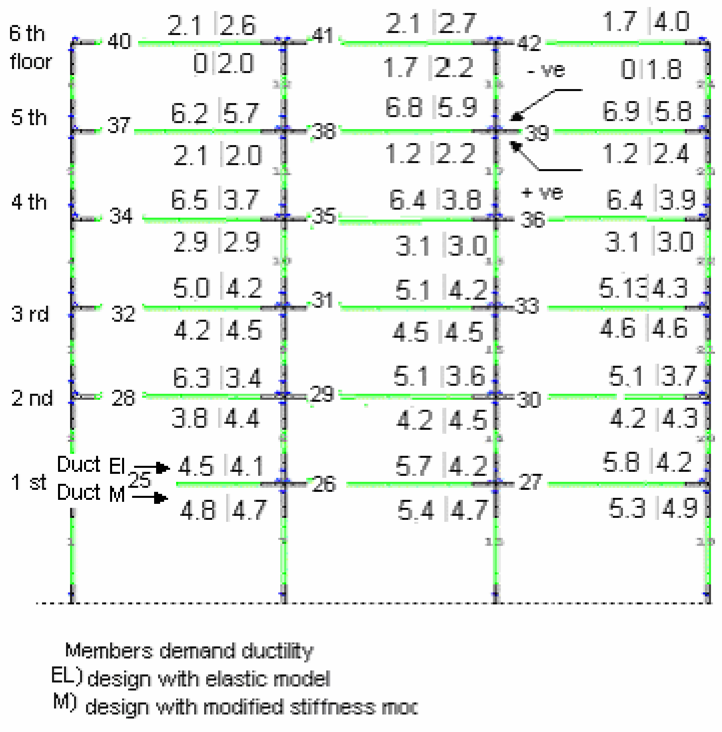


mem	EL +/-	M +/-
25	1.1	1.03
26	1.36	1.13
27	1.38	1.08
28	1.82	0.86
29	1.43	0.98
30	1.39	1.0
31	1.2	0.98
32	1.2	0.99
33	1.2	1.02
34	1.74	1.01
35	1.69	1.02
36	1.64	1.02
37	1.1	1.0
38	1.17	1.0
39	1.19	0.5
40	0.82	0.0
41	0.79	0.8
42	0.42	0.0

Figure 12-22: Ductility and ductility proportion at the structure members.

Final non-linear analysis, with plastic hinges at the ground floor columns

The nonlinear analysis of the structure is carried out for the elastic model and the modified stiffness model with Ruaumoko. In this analysis all the nonlinear aspects are accounted for, including the plastic hinges for the ground floor columns in the Ruaumoko program (Figure 12-23). The analysis results are given in Table 12-11 through Table 12-12 and (Figure 12-24) through (Figure 12-28).

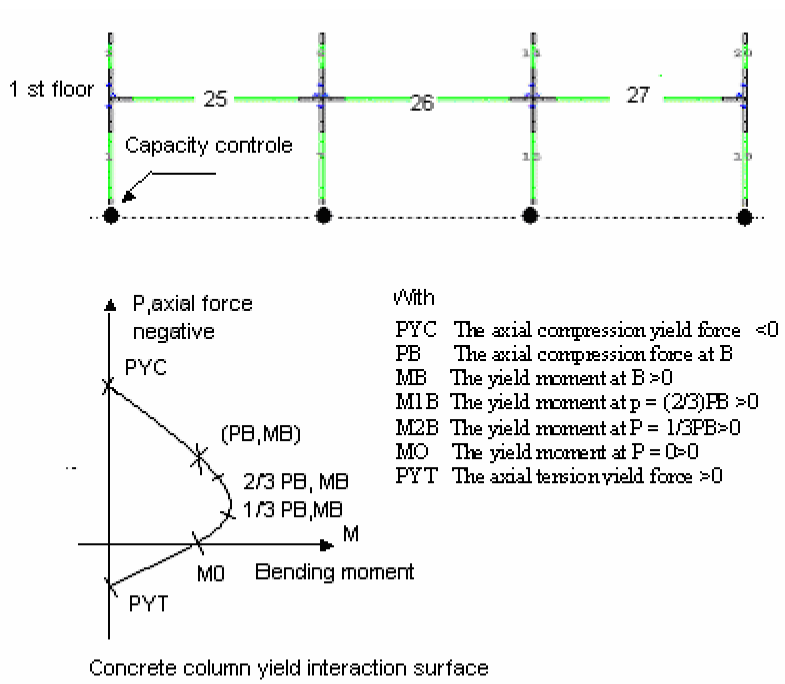


Figure 12-23: Ruaumoko model, ground floor columns with plastic hinges

Table 12-11: The ductility of the members with elastic and modified stiffness model considering the degradation and the capacity requirements.

	Elastic model			modified stiffness model		
mem	+ve	-ve	+ve/-ve	+ve	-ve	+ve/-ve
25	7,76	7,16	1,08	6,94	6,67	1,04
26	9,70	7,00	1,39	7,17	7,02	1,02
27	9,90	6,88	1,44	7,15	7,83	0,91
28	10,30	5,21	1,98	4,18	5,14	0,81
29	8,75	5,45	1,60	5,00	5,73	0,87
30	8,52	5,81	1,47	4,91	5,54	0,89
31	8,07	6,58	1,23	5,11	5,44	0,94
32	7,98	6,64	1,20	5,12	5,61	0,91
33	8,00	6,70	1,19	5,10	5,68	0,90
34	5,45	4,18	1,30	2,24	3,35	0,67
35	5,54	4,39	1,26	2,31	3,63	0,64
36	5,93	4,32	1,37	1,89	3,74	0,51
37	1,66	5,85	0,28	0,11	2,04	0,05
38	3,60	6,16	0,59	1,82	2,39	0,76
39	2,89	6,44	0,45	1,68	2,48	0,68
40	0,00	1,76	0,00	0,00	2,45	0,00
41	0,00	1,89	0,00	0,00	2,69	0,00
42	0,00	3,49	0,00	0,00	2,60	0,00

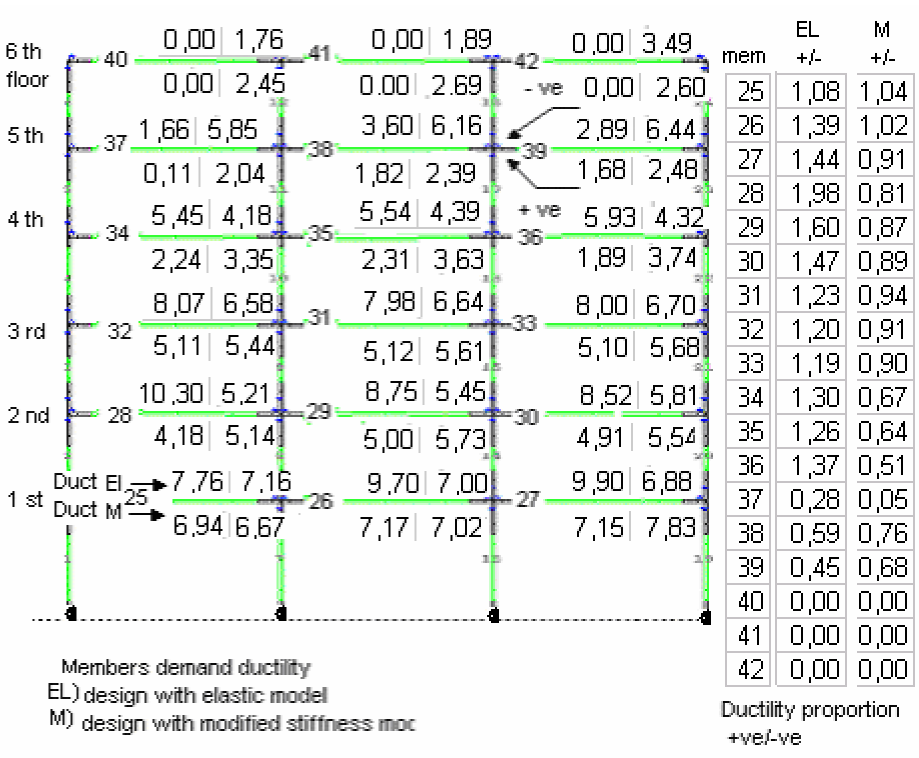


Figure 12-24: Ductility and ductility proportion at the structure members, with plastic hinges at the supports

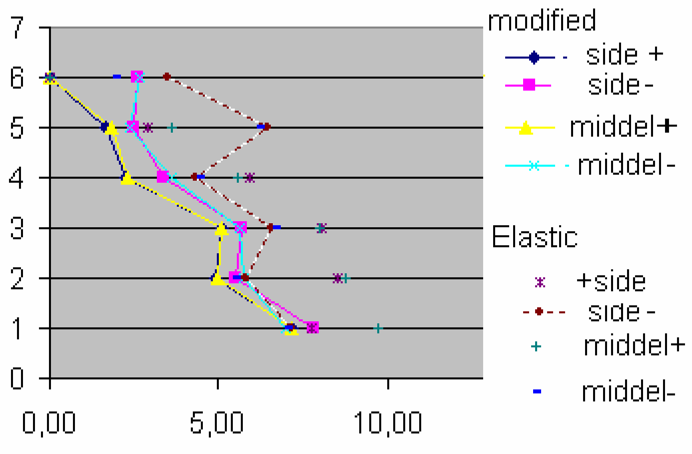


Figure12-25: The ductility of the structural member at each floor, designed with elastic model, modified model and modified stiffness model considering the capacity and the degradation hysteresis.

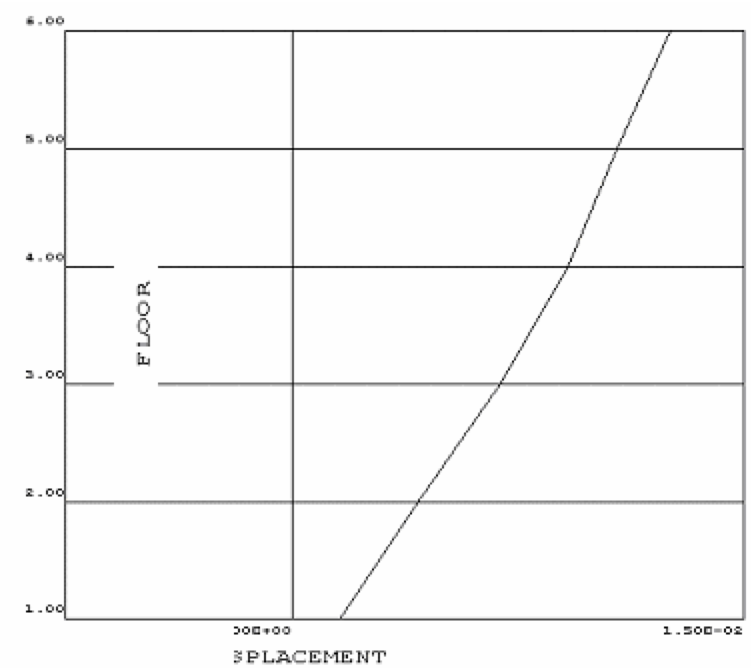


Figure 12-26: Relative displacement in the concrete structure

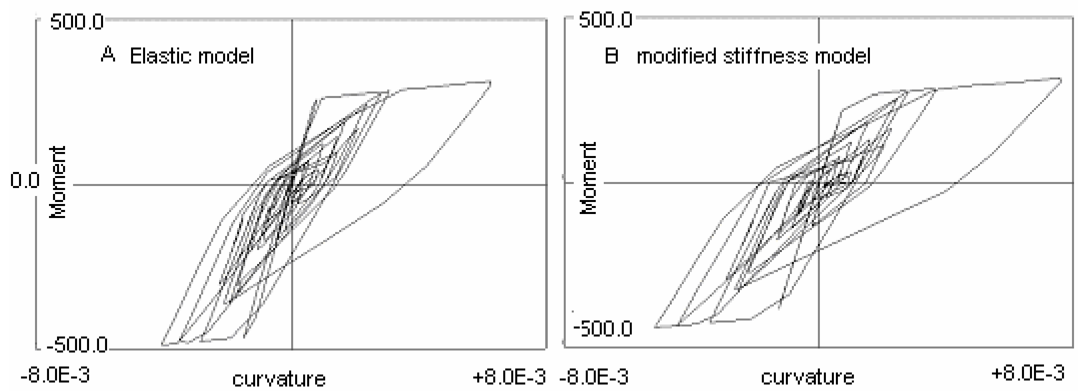


Figure 12-27: Moment curvature diagram for member 26 Node 5 with A) Elastic model, B) Modified effective stiffness model effective stiffness model

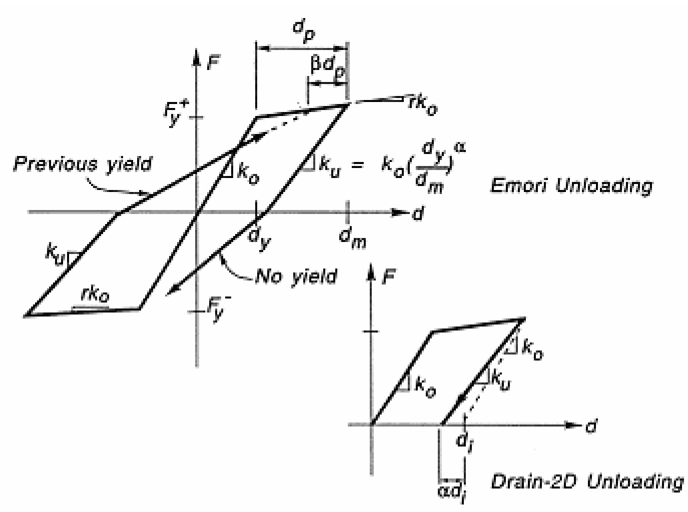


Figure 12-28: Takeda bilinear degrading stiffness

Table 12-12: Takeda bilinear degrading parameters

Location	Alpha	Beta (reloading)	N (power)	Unloading (1=Drain, 2=EMORI)
Column basis Sec1, 2,	0.4	0.6	1	1
Beam ends Sec 3,4…13	0.5	0.0	1	2
N power; reload stiffness power.				

Table 12-13: The ductility and the bending moment and its proportions of the beams, with EL4scaled to 0.75 and 1.2

Member	scale =0.75		scale =1.2		Proportion	
	μ1	F1	μ2	F2	μ1/μ2	F1/F2
25	4,08	-470	8,52	-522	2,09	1,1
26	3,77	-510	7,89	-562	2,09	1,1
27	3,64	-508	7,42	-556	2,04	1,1
28	2,9	-445	5,86	-479	2,02	1,1
29	3,1	-448	6,74	-490	2,17	1,1
30	3,01	-447	6,18	-483	2,05	1,1
31	3,44	-405	6,98	-441	2,03	1,1
32	3,41	-405	7,28	-444	2,13	1,1
33	3,3	-404	6,78	-439	2,05	1,1
34	1,56	-341	3,76	-361	2,41	1,1
35	1,52	-341	3,9	-362	2,56	1,1
36	1,47	-340	3,62	-360	2,46	1,1
37	2,04	-345	1,78	-343	0,88	1
38	2,39	-349	1,95	-345	0,82	1

μ1 F1 The ductility and the bending moment on the member seismic record EL40 scale 0.75

μ2 F2 The ductility and the bending moment on the member seismic record EL40 scale 1.2

The analysis for the modified effective stiffness model is carried out with the Ruaumoko non-linear program, using the degradation with Takeda degrading rule. The resulting demand ductility is high while the response bending moment in the beams did not record high quantity. The reinforcement in the beam end has good proportion to resist and homogeny system works with the required seismic actions. The demand ductility in the beams is still within the available ductility given by the design. While the displacement increased considerably.

Conclusion

The ductility can be used as a measure for the strain and the participation of the reinforcement in the seismic resistance. In case of the controlled reinforcement ratio in both sides of the beams under seismic loading the response of the connected elements can be improve to resist higher seismic actions. The hysteresis behaviour of the connection designing with modified effective stiffness is improved and the ductility of the beams indicates the capability to exert higher seismic action. And the resulting bending moment do not increases considerably. The control of the ductility is carried out using the effective stiffness at the beam ends, its use leads to better redistribution of the bending moment and its proportionality in the top and the under faces. The reinforcement proportionality in the connected element is a factor to be considered, which influences the reaction of the elements against the seismic action.

13 Design with ductile, pretension elements, and hybrid-fibre concrete.

Hybrid-fibre concrete has high–dissipation capacity, which is of interest when dynamic loads are involved, moreover, because of its high tensile strength, cracking and structural integrity can control even in case of relatively strong impact. For known Ultra-High performance, the tensile strength f_t increases similar to that for conventional concretes. It means that the cracks at the columns face can be verified with the same level of confidence as that of the conventional concrete, using the similar over-strength factors .The steel reinforcement should yield before that the concrete cracked. In the reversal loading the drop of the stiffness and strength of the concrete due to reversal load should be limited in order to protect the column core from sudden shocks. It is important to insure that the resistance in the column beam intersection core, remains in any case above the transmitted loads, moments, shear, from the columns, and beams. In the design with HFC its is required to protect its high resistance strength in the column beam connection from the crack, and use its high ductility and its ability to resist different moment levels according to the used reinforcement in the beam ends. And it is ductile with high shear resistance that enables relocate the yielding in places, away from the column, that can be mobilizes, the member connection with the beam of B55 conform the shear requirement easily without heavy vertical reinforcement. The design and the design recommendations with hybrid-fibre concrete are reported in chapter 14. In the design of the HFC, the structural analysis results of the elastic stiffness model with 20% reduction of the moments in the beams ends. Hybrid-fibre concrete is used in the column beam connections, as a precast unite supported on the column and connected with the inside beams of B55 by cast in place concrete joints. The column reinforcement passes inside sleeve through the precast unit. The design of the flexure reinforcement has been carried out solving the Hybrid-fibre concrete equilibrium equations of the cross section (chapter7). In the columns beams connections extra flexural reinforcement is added to increase the moment resistance of the connection and prevent crack and splits failure at the core. The control of the yield locations carried out considering an over-strength factor. The design shear of the beams considered is carried out considering possible plastic hinge formation at the beam-ends apply the capacity design principles, and because of the high shear resistance of the Hybrid-fibre concrete, the required shear reinforcement is the minimum. The columns used in the solution are the same of the emulated monolithic R/C B55. The most important different is the forces exerted by the beams on the columns, due to the different strength and stiffness of the two materials. That has been verified through the application capacity design requirements. The increase of the hybrid-fibre concrete reinforcement increases considerably its moment resistance. The design reinforcement should be carefully calculated, extra reinforcement results in high resistance moment in the beam ends leads to higher reversed seismic action on the columns.

The important adaptation principles in the design with DDC connection is the relocating the causative actions by; Relocate the yielding element to within the column where the confinement of the concrete protect it verse the lateral support action. In the design of the ductile connectors, the capacity design principles are applied trough considering the strength reduction factor and the over strength factors within the design criteria’s. The design is carried out using the elastic model with 20% reduction of the moments in the beams ends, and using the modified effective stiffness model. The important factors in the design is the nominal moment capacity of the ductile member group, which decides the transfer load on the column, and the nominal shear capacity in the connectors. The applied force on the ductile connector is unaffected by the preloading level, and the connecting members should be logically verified for the uncertainties associated with each of the considered transfer mechanisms. The load and the over strength factors should be used in the ultimate moment calculation. The design results indicate that the average reinforcements ratio, and the pretension of the bolts are reasonable values. The roof may be built with emulated monolithic simple connection, since the shear and the bending moments is not significant.

Hybrid post-tension beam system consists of concentric post-tensioned cables anchored at both ends of the frame The clamping force creates a friction force between the beams and columns, which transfers the shear demands. Mild reinforcing provides energy dissipation during a seismic event is placed at the top and bottom of the beam through the joint and is grouted in place. By limiting post-yield rotations to the joint, damage to the system is minimized. The restoring force in the hybrid beam system is provided by the elastic post tensioned and the flexure strength is provided by combination of unbounded post tensioning strand and bonded mild steel. The design with this type of connection is carried out using the elastic model with 20% reduction of the bending moments in the beams ends, and the modified effective stiffness model. Factors should be considered and verified in the design are the strand stress and its nominal moment, and the flexural moment provided by the mild steel. The friction due to the compression force of the tensioned tendons provides the connections shear resistance. The usage of the hybrid post-tension connection provides better solution for the shear where it is needed.

The design and the calculations of the beam column connection with different alternatives such as the post tension assemblage PT, the ductile rod connection DDC, and a new connection system, the connection with hybrid-fibre concrete HFC is illustrated in this chapter (Figure 13-1). The structural analysis has been carried out using the elastic model. The design loading has been used after a reduction of 20% of the bending moments in the beam-ends (Figure 11-3 through Figure 11-6). Possible yield locations at the first floor may be recognised in Figure 13-2.

Analyses models for these solutions have been made. These models include the capacity design requirements. The solution was applied for all the building connections, and the design results are listed in tables for each connection type.

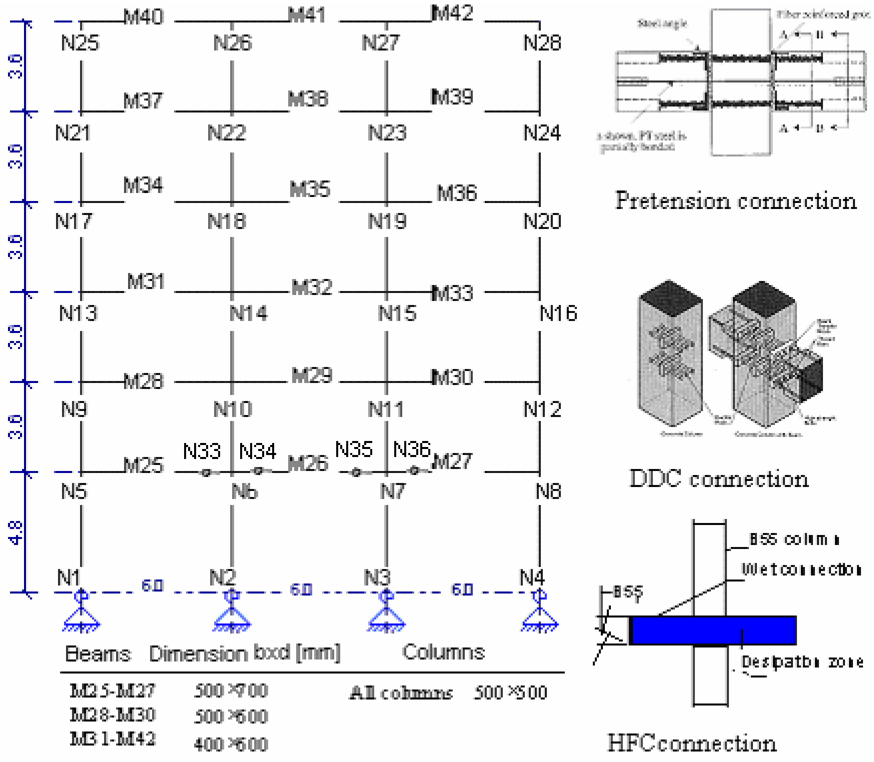


Figure 13-1: The building structure with beams columns, of DDC ductile Connectors, post tension assemblage, and HFC connection, dimension in [mm]

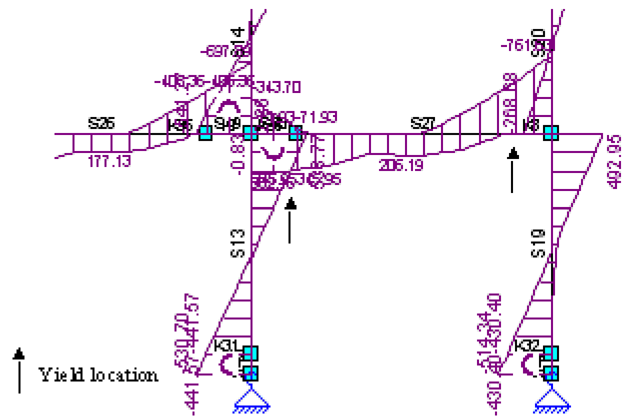


Figure 13-2: Bending moment diagram and yield locations at the first floor.

13.1 Bolted Assemblages Dywidag ductile connector DDC

The Bolted assemblage Dywidag ductile connector DDC was motivated to improve the post-yield behaviour of the concrete ductile frame (Figure 13-4). The main aspect in the design is the introduction of a ductile road or fuse into the path away from the toe of the beam to allow post-yield deformation where the members are jointed. The desired behaviour of the structure achieved through merging the steel technology with the basic objective of seismic loading limitations.

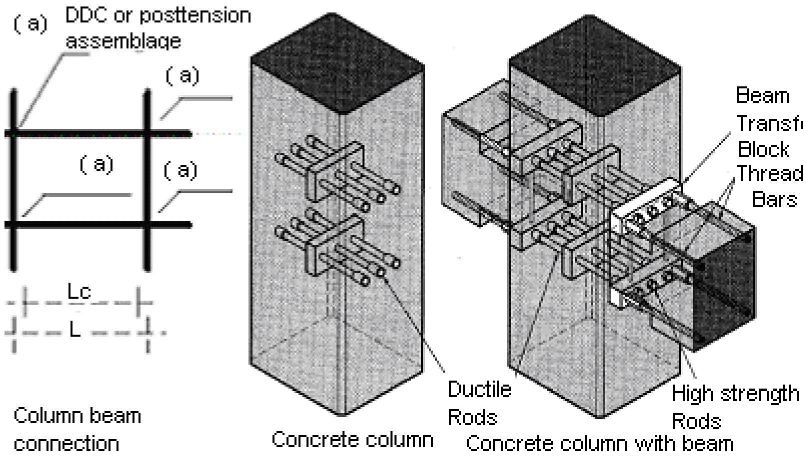


Figure 13-3: Bolted assemblage, Dywidag ductile connectors (DDC).

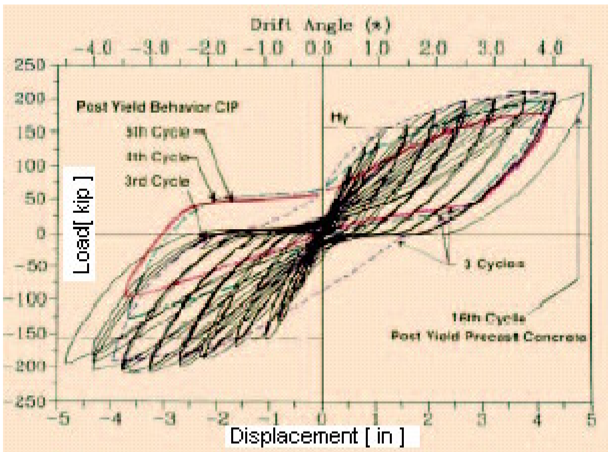


Figure 13-4: Experimental hysteresis loop using DDC connection.

The important principles for the adaptation of this connection type for the seismic action is the relocating the causative actions by; Relocate the yielding away from the beam toe. In the design of the ductile connectors the capacity design principles are applied trough considering the strength reduction factor and the over strength factors.

13.1.1 The design with Bolted Assemblages Dywidag ductile connector DDC

The relations, formulas and the limitation cover the design with bolted assemblage ductile connectors DDC are introduced. System of column beam connection using DDC connection is shown in (Figure 13-1). The relations govern the design with DDC may be given as follows:

The nominal tensile strength of one ductile rod is calculated as follows:

$$M_n = Num \cdot T_y \cdot (d\text{-}d`)$$

The bolt cover $d` = 2c$ (13.1)

where $T_y = f_{ybolt} \cdot A_{bolt}$ and $d` = 2c$

The transfer load on the column is calculated as follows:

$$T_{bn} = \lambda_0 \cdot \frac{M_n}{(d\ \text{-}d`)} \tag{13.2}$$

The shear load induced by the ductile rod at mechanism on the beam-column interface is calculated as follows:

$$V_{nE} = 2 \cdot \frac{\lambda_0 \cdot M_n}{L_c} \tag{13.3}$$

The nominal shear capacity required of the connectors is calculated as follows:

$$V_{cn} = V_{nE} + V_D + V_L \tag{13.4}$$

The shear transfer mechanism between the column and the beam is the friction.

The ability of pretension is 2NTp. (13.5)

The applied moment in some instance is zero. At this instance both the upper and the lower bolts participate in the shear transfer. The equilibrium condition is carried out as follows:

$$V_D + V_L < 2 \cdot Num \cdot T_p \cdot f \tag{13.6}$$

Observe that the force applied to the ductile rod is unaffected by the level of preload The nominal capacity of the shear transfer mechanism is the maximum value of V_1 and V_2 and it is calculated as follows:

The shear due to the moment action is given as follows: $V_1 = \frac{M}{d\text{-}\ d`} \cdot f$ (13.7)

Or

The shear due to the pretension is given as follows: $V_2 = Num \cdot T_p \cdot f$ (13.8)

The members should be logically verified for the uncertainties associated with each of the considered transfer mechanisms. The factor ϕ/λ_0 should be used for the account of the moment. It means that:

$$M_u \le M_n \cdot \frac{\phi}{\lambda_0} \tag{13.9}$$

The required area of the flexural reinforcement is given as follows:

$$A_s = \frac{\lambda \cdot Num \cdot T_y}{\phi \cdot f_y} \tag{13.10}$$

The development length needed for the DDC is given as follows:

$$L_d = \frac{T_y}{5 \cdot \pi \cdot d_b \cdot \sqrt{f'_c}} \tag{13.11}$$

13.1.2 The building connections design with DDC connection

The design of the building connection with bolted assemblage was carried out using a simple computer program to solve the relations given in section (13.1.1.) The structural analyses results of bending moment and shear forces (Figure 11-11 and Figure 11-13) are used as input for this solution. The design results are summarised in Table 13-3. The design is carried out as follows:

(1) Design of the column beam connection using DDC.

In this paragraph we apply the design of DDC connectors according to the relations given in 13.1.1, using equation (13.1), (13.12) and (13.2), the loading and the strength of the ductile members are reported in Table 13-2.

The applied moment and shear forces are given as follows:

M_u = 451 kN.m, V_u = 309 kN, and P = 338 kN

Where the ultimate shear is the ultimate shear associated with the seismic loading, and P is the shear force due to the dead and live load.
Assume that $V_d = 0.7 \cdot P$ and $V_L = 0.3 \cdot P$. The nominal tensile strength of one ductile rod T_y = 321.699 kN. The connection characteristics are reported in Table 13-1.

Table 13-1: Characteristic of the DDC, connection members

The beam depth d The clear span length L_c	0.6 m 5.4 m
The yield stress f_y The bolt yield stress, f_{ybolt}	400 N/mm^2 400 N/ mm^2
Bolt diameter ϕ	32 mm
The maximum number of The rods in one set, N_{bolt}	5
The parameters: λ_0 ϕ	1.2 0.9

The nominal area of the bolts in one set is calculated as follows:

$$A_{ns} = N_{bolt} \cdot A_{bolt} \tag{13.12}$$

The nominal moment capacity is calculated as follows:

M_n = 732.823 kN.m Using equation(13.1)

The transfer load on the column is calculated as follows:

T_{bn} = 1930 kN Using equation(13.2)

The shear load induced by ductile rod at mechanism on the beam column interface is calculated as follows:

V_{nE} = 321.699 kN Using equation(13.4)

The bolt pretension T_p = 130 kN. At each face the number of bolts is Num $_p$= 4. The friction factor allowed by the LRFD specifications is f = 0.33

The control of the shear and the moment equilibrium is carried out as follows:

$V_D + V_L < 2 \cdot N_{bolt} \cdot T_p \cdot f$, Ok Using equation(13.6)

$M_u \leq M_n \cdot \frac{\phi}{\lambda_0}$, Ok Using equation (13.9)

The shear due to the moment action and the shear due to the pretension are calculated as follows:

V_1= 330.733 kN and V_2 = 171.6 kN Using equation, (13.7)and (13.8)

V_n= 330.733 kN, Where: V_n is the maximum of V_1 and V_2

The required area of the flexural reinforcement is calculated as follows:
A_s = 3990 mm^2 Using equation (13.10)

Table 13-2: Loading and stress on the bolted assemblage DDC.

The nominal tensile strength, T_y	321.699 kN
The nominal moment capacity, M_n	723.823 kN.m
The transfer load on the columns, T_{bn}	1.93 10^3 kN

The shear load induced by the ductile rod at mechanism on the beam-column interface

V_{nE} = 321.699 kN Using equation (13.3)

The nominal shear capacity required of the connectors is

V_{nc} = 659.699 kN Using equation (13.4)

And the nominal steel area A_{sn} = 4.021 10^3 mm^2 Using equation(13.12)

The design information of the column beam connection with DDC connectors is illustrated in Table 13-3. The dimensions of the connection are classified in Section types 1: 700×500 mm, Section types 2: 600×500 mm, Section types 3: 600×400 mm, for the member numbers referee to the geometry of the structure (Figure 13-1).

Table 13-3: Connection design details with bolted assemblages DD, with elastic model and modified stiffness model

Type: member number*	Loading			$A_{ns}[mm^2]$		T_p [kN]
	M_{ucon}[kN.m]		M_{ucon}[kN.m]			
	80%El.	Mod.		El	Mod	
1/25	609	446	295	4021	3217	130
1/26	579	337	294	4021	3217	130
1/43	580	453	154	4021	3217	150
1/44	580	741	256	4021	4825	150
2/28	508	451	309	4021	4021	130
2/27	508	519	319	4021	4021	130
3/31	434	400	289	3217	3217	130
3/32	434	406	290	3217	3217	130
3/34	356	456	273	3217	3217	130
3/35	356	363	271	3217	3217	130
3/36	384	384	276	3217	3217	130
3/37	270	331	254	3217	3217	130
3/38	285	335	254	3217	3217	130
3/ 40	186	194	195	3217	3217	150
3/41	285	204	190	3217	3217	150
Ductile rod area of one set A_{ns}, Bolt pretension; T_p . The loading is considered as follows: 80%El; 80% of the beams ends bending moments resulting from elastic model, Mod; The end beams bending moments resulting from the modified elastic model.						

Notice that the reinforcement ratio; A_s /bd = 3200/(600×400) = 0.0133 per set (Table 13-3). And that is an acceptable reinforcement ratio. The pretension of the bolts varies from 130 to 150 kN, the section at the roof members 40 and 41 can be executed with emulated monolithic simple connection, while the shear and the bending moments is not significant. There is no wide difference between the reinforcement with modified and elastic model.

13.2 Hybrid Post-Tensioned Assemblage

The Hybrid beam system consists of concentric post-tensioned cables anchored at both ends of the frame (Figure 13-5). The clamping force creates a friction force between the beams and columns, which transfers the shear demands. Mild reinforcing steel provides the straining of which provides the necessary energy dissipation during a seismic event is placed at the top and bottom of the beam through the joint and is grouted in place. These bars are also wrapped, rebounded in a region adjacent to the column to reduce inelastic strain and to force all post-yield rotation to occur at the beam-column interface. By limiting post-yield rotations to the joint, damage to the system is minimized. An additional benefit of the Hybrid Beam system is the restoring force provided by the elastic post tensioned. Flexure strength in the hybrid beam is provided by combination of unbounded post tensioning strand and bonded mild steel, if the strand stressed up to T_{nps} then the moment of M_{nps} is developed. The mild reinforcement grade 60, where placed in the top and bottom of the beam in tube that where subsequently grouted with high-strength grout. Figure 13-6 shows experimental hysteresis loop for post tension assemblage, where the drift versus the loading appears a wide range envelop.

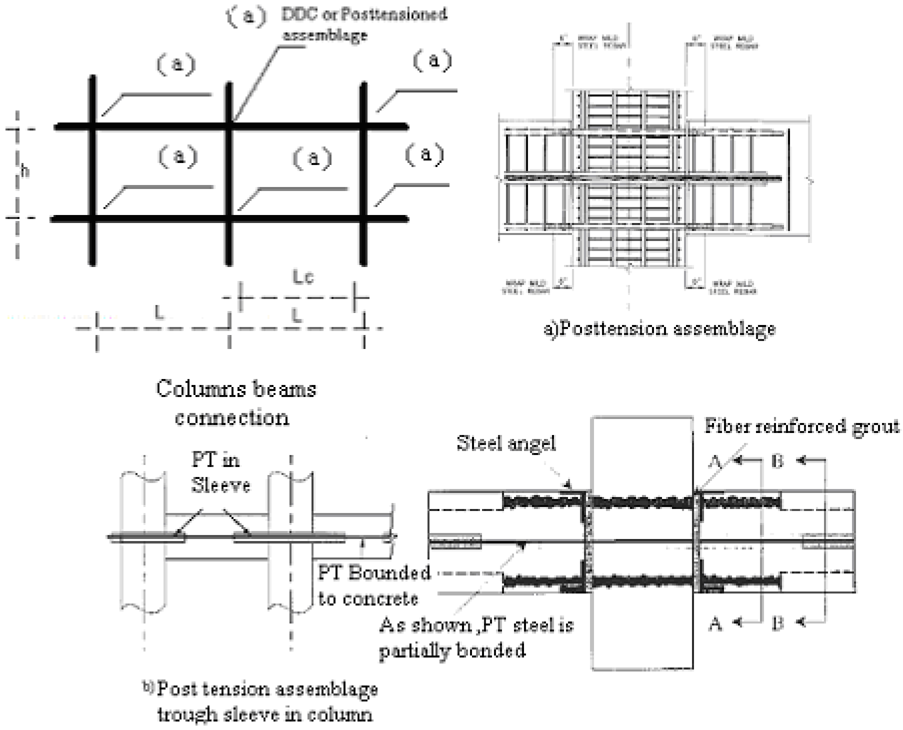


Figure 13-5: Details of post-tensioned connection, and alternative post tensioned assemblage through sleeve in column.

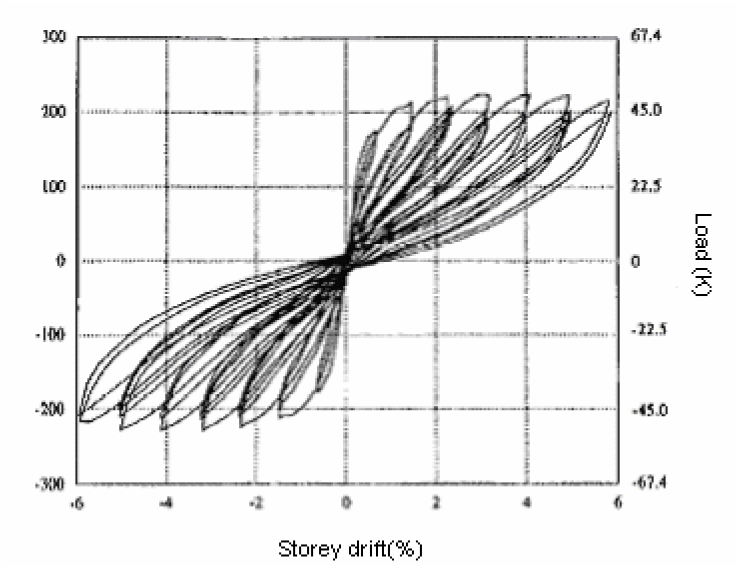


Figure 13-6: Experimental hysteresis loop using post tension assemblage.

13.2.1 Design with hybrid post tensioned assemblage

The relations and the limitation for the design with post-tensioned assemblage are illustrated in the following pages. System of beam columns connection with post-tensioned assemblage is shown in Figure 13-1. The detail of the connection and the arrangement of the ductile bars and post tensioning are illustrated in Figure 13-5. The post tension is partially bounded. An alternative for the post-tensioned assemblage with the post tension cables passes inside a sleeve through the column is also shown.

The flexural strength provided by the mild steel is calculated as follows:

$$M_{ns} = T_{ns} \cdot (d\text{-}d`) \tag{13.13}$$

The flexural strength provided by the unbounded post-tensioning M_{ns} is developed as follows:

$$T_{nsp} = T_{nsp} f_{pse} \tag{13.14}$$

Te compression depth zone and the nominal moment are given as:

$$a = \frac{T_{nsp}}{0.85 \cdot f`_c b_b} \quad \text{and} \quad M_{nsp} = T_{nsp} \cdot (\frac{h_b}{2} - \frac{a}{2}) \tag{13.15}$$

The nominal moment capacity of the hybrid frame beam is calculated as follows:

$$M_n = M_{ns} + M_{nsp} \tag{13.16}$$

The corresponding shear, and the associated column shear or applied test frame force F_{col} are calculated as follows:

$$V_{nb} = \frac{M_n}{L_b} \quad \text{and} \quad V_c = F_{col} = V_{nb} \cdot (\frac{L_b}{h_x}) \tag{13.17}$$

The rotation capacity of the connection is defined as follows:

$$\theta = \frac{\Delta_s}{d\text{-}a} \tag{13.18}$$

The stiffness factor is calculated as follows:

$$K_{connection} = \frac{M_{nps}}{\theta} \tag{13.19}$$

Solution

In the assemblage assume that the stress in the mild steel, the compression reinforcement reaches the yield stress. The flexural strength provided by the unbounded post tensioning; M_{nps} is developed as follows:

$$\begin{gathered} M_{ns} = T_{ns} \cdot (d - d`) \\ T_{nps} = A_{nps} \cdot f_{pse} \\ a = \frac{T_{nps}}{0.85 \cdot f`_c b_b} \quad \text{and} \quad M_{nps} = T_{nps} (\frac{h_b}{2} - \frac{a}{2}) \end{gathered} \tag{13.20}$$

The nominal moment capacity of the hybrid frame M_n is calculated as follows:

$$M_n = M_{ns} + M_{nps} \tag{13.21}$$

This is correspond to beam load or shear V_{nb} as follows:

$$V_{nb} = \frac{M_n}{L_b} \tag{13.22}$$

The associated column shear is :

$$V_c = V_{nb} \cdot \left(\frac{L_b}{h_x}\right) \tag{13.23}$$

With L_b is the beam length and h_x is the column height.

The design is given in the following steps:

Step (1)

Determine trial reinforcement as follows:

$$M_{us} = C_m M_u \tag{13.24}$$

$$A_s = \frac{C_m \cdot M_u}{\phi \cdot f_y \cdot (d - d`)} \tag{13.25}$$

With the assumption of using reasonable level of mild steel

Determine the amount of post tensioning steel required as follows:

$$M_{ups} = M_u - 0.9 \cdot (d - d`) \cdot A_s \tag{13.26}$$

$$A_{ps} = \frac{M_{ups}}{\phi \cdot f_{ps} \cdot \left(\frac{h_b}{2} - \frac{a}{2}\right)} \tag{13.27}$$

Step (2)

Determine the minimum size of the beam column connection joint as follows:

$$A_j = 62 \cdot (A_s + A_s`) + 210 \cdot A_{ps} \tag{13.28}$$

Control the beam size so that A_j □ $b_b \times h_b$

The hybrid beam will become integral part of the floor system. Large stress differential may cause undesirable cracking in the unstressed floor. Accordingly it is preferred to limit the prestressed to 6.89 N/mm^2. The shear in the beam is calculated as follows:

$$V_b = \frac{\lambda_0 \cdot (M_{ns} + M_{nps})}{\frac{L_c}{2}} + \frac{P_D + P_L}{2} \tag{13.29}$$

Control that: $V_b < b_b h_b \tau_b$

Sufficient accuracy in the design may be obtained using over-strength factor λ_0 = 1.25

Step (3)

Check the column shear if the beam is deep and the column is short.
The elongation of the mild steel is assumed to be equal over the unbounded length.
It is calculated as follows:

$$\Delta_s = \varepsilon_u \cdot (L_u + 5.5 \cdot d_b) \qquad (13.30)$$

The angel of rotation and the stiffness factor are calculated as follows:

$$\theta = \frac{\Delta_s}{d\text{-}a}$$

$$K_{connection} = \frac{M_{nps}}{\theta} \qquad (13.31)$$

13.2.2 The building design with hybrid post tension connection

The design of the building connection was carried out. In the solution we used simple computer program to solve the relation in (13.2.1). And the structural analyses results of bending moment and shear forces (Figure 11.4 and Figure 11.5) are used as input for the calculation. The design results are summarised in Table 13-4.

Table 13-4 Connection design details with post tension assemblage, for the elastic model and the modified stiffness model

Type: Member, number	Loading			A_{nps}[mm^2]		A_{ns}[mm^2]	
	M_{ucon} [kN.m]		V_{ucon} [kN.m]				
	80% El.	Mod.		El.	Mod.	El	Mod
1/25	609	446	295	808	598	1090	805
1/26	579	337	294	808	450	1090	605
1/43	580	453	154	808	598	1090	805
1/44	580	741	256	808	598	1090	805
2/28	508	451	309	808	720	1090	1000
2/27	508	519	319	890	690	1090	1390
3/31	434	400	289	690	640	930	860
3/32	434	406	290	690	640	930	860
3/34	356	456	273	567	726	764	980
3/35	356	363	271	567	580	764	780
3/36	384	384	276	612	580	764	780
3/37	270	331	254	430	580	579	780
3/38	285	335	254	430	534	579	720
3/ 40	186	194	195	400	325	302	438
3/41	285	204	190	430	325	579	438
80% El; 0. 80% of the beams end bending moment of the elastic model. Mod; The beam-ends bending moment of the modified elastic stiffness model. $V_{ucon;}$ The applied shear force. A_{ns}[mm^2]; The nominal area of the mild steel. A_{nps}[mm^2]; The area of the post tension steel							

13.2.3 Design with hybrid-fibre concrete

The studies indicate that Hybrid-fibre concrete (HFC) perform well in seismic design. The main reason is the confinement for the concrete provided by the steel fibre, and hence improve its toughness and ductility, which is one of the important concepts in the seismic design. In our study we illustrate tests results, over the steel fibre concrete. Fiber reinforced concrete is a concrete mix that contains short discrete fibers that are uniformly distributed and randomly oriented. Fiber material can be steel, cellulose, carbon, polypropylene, glass, nylon, and polyester [2]. The amount of fibers added to a concrete mix is measured as a percentage of the total volume of the composite (concrete and fibers) termed V_f, V_c, typically ranges from 0.1 to 3%. Aspect ratio (L/d) is calculated by dividing fiber length (L) by its diameter (d). Fibers with a non-circular cross section use an equivalent diameter for the calculation of aspect ratio. The effects of steel fibers on mechanical properties of concrete are depicted in (Figure 13-7). The addition of steel fibers does not significantly increase the compressive strength, but it increases the tensile toughness, and ductility. It also increases the ability to withstand stresses after significant cracking (damage tolerance) and shear resistance. Reinforced HFC possess higher ultimate and yield moment as it compared with the conventional reinforced concrete, with les reinforcement ratio (Figure 13-8).

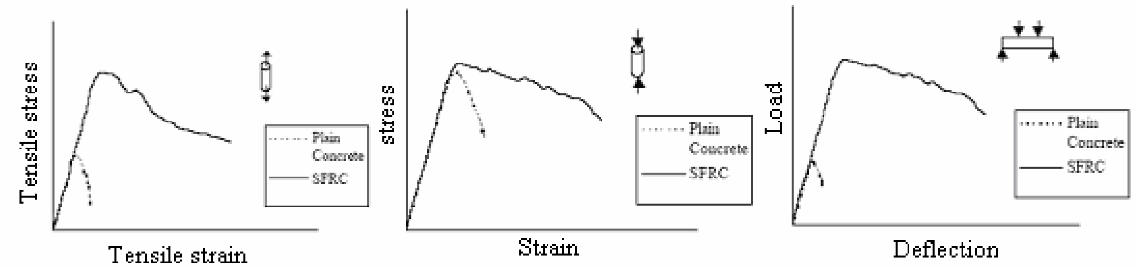


Figure 13-7: Properties of reinforced steel fiber concrete

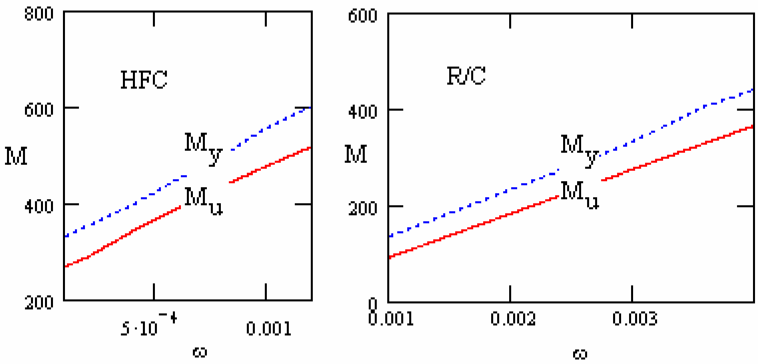


Figure 13-8: Ultimate and yield moment [kN.m] for HFC, and R/C rectangular section b =500mm, h=700mm

The stress strain characteristics of the HFC, used for the building in our study, are given in the appendix. And the stress strain tension and compression parameters are calculated in chapter (7).

13.2.3.1 Toughness.

Toughness enhancement is among the most important contributions of steel fibers to concrete. Toughness or energy absorption capacity is the area under a load-deflection, moment-rotation, or stress-strain curve (Figure 13-9). This is especially important for structures subjected to large energy inputs such as earthquakes, blast loads, impact loads, and other dynamic loads.

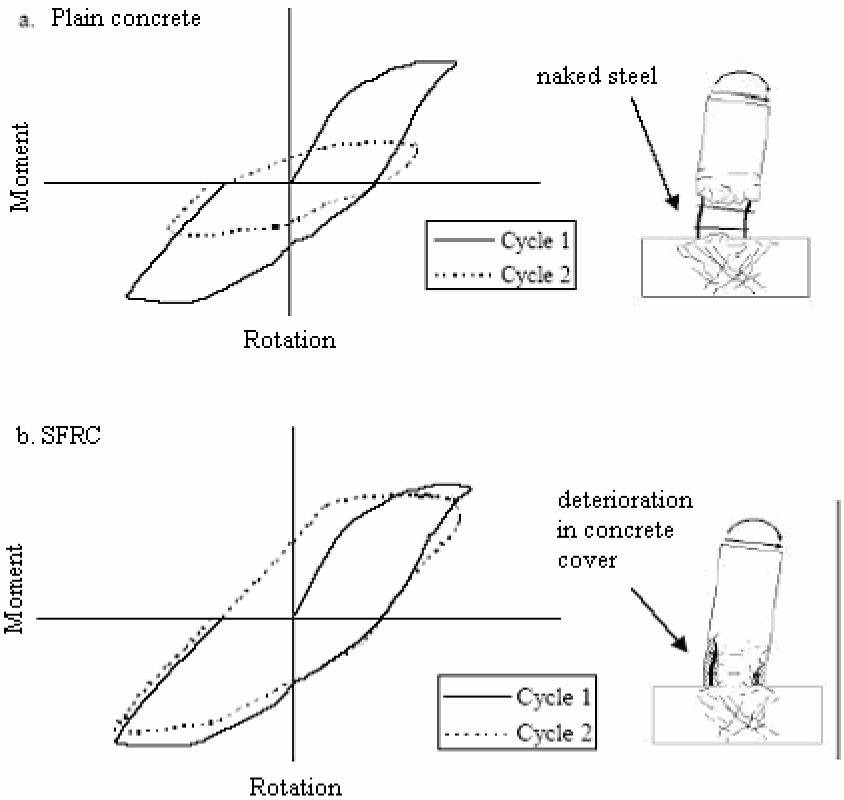


Figure 13-9: Improvement of the joint behavior resulting from SFRC

13.2.3.2 Seismic strength

It is well known that the use of steel fibers raises the ductility of concrete and the fracture energy. This phenomenon is transferable to the concrete shear strength. Researchers reported about an increasing shear capacity of steel fiber reinforced concrete beams. The experimental results clarify the enormous influence of steel fibers on the shear capacity of slender beams (Figure 13-12). Experiments show that the transverse shear reinforcement spacing influences the shear resistance of the SFRC, and that the SFRC has better shear resistance than the plain concrete joint. Experiment with different the hoop spacing shows that the joint with (15.2 cm) hoop spacing provides better seismic resistance than joint with (20.3 cm) hoop spacing which itself improved seismic resistance over the plain concrete joint with (10.2 cm) hoop spacing. Hysteresis loops, hysteresis envelope curve, observations, and test show that. These exterior SFRC joints would prevent structural collapses of a building unlike the code-designed plain concrete The behaviour of the joints with SFRC perform better than plain reinforced under cyclic loading (Figure 13-10).

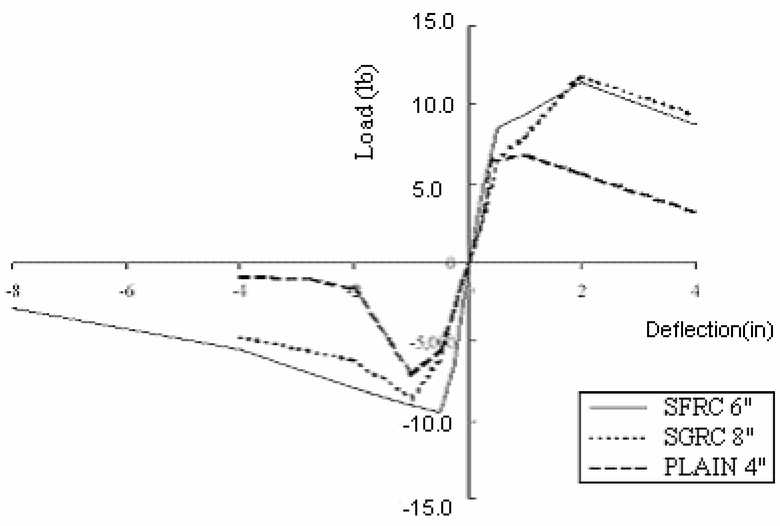


Figure 13-10: Hysteresis envelop curve, with transverse reinforcement at 6”,8” for HFR, and at 4” for plain concrete

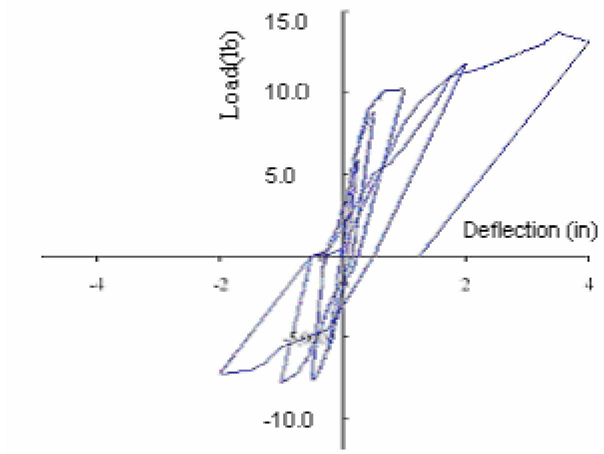


Figure 13-11: Hysteresis loop for SFRC beam-column joint with (15.2 cm) spacing

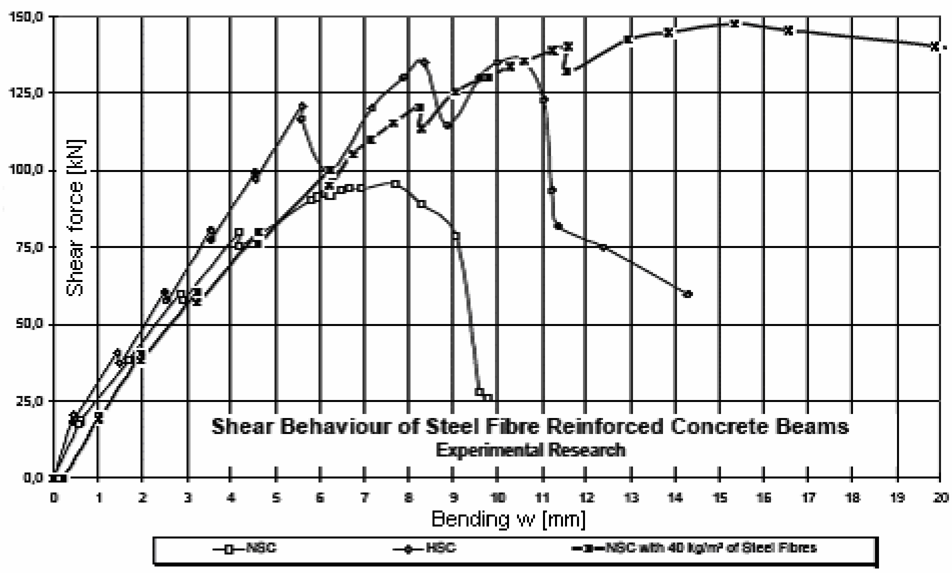


Figure 13-12: Shear capacity of concrete with and without steel fiber

13.2.3.3 Design recommendations

FRC, like most fiber-reinforced concrete, has a high dissipation capacity, which is of interest when dynamic loads are involved, moreover, because of its high tensile strength, cracking and structural integrity can be verified even in case of relatively strong impact. On the other hand no data on the use of the HFC for penetration problems is currently easily available.

The empirical simplified description which applies to concretes with compressive strength ranging from 30 to 120 Mpa, and which can be transposed to FRC quite precisely so that it can be put to full effect in special applications. For known UHPFRC, the tensile strength f_t increases by about 0.8 Mpa/log10 unite, compared to o.70 Mpa/ log10 unit for conventional concretes. It means that the cracks at the columns face can be verified with the same level of confidence as that of the conventional concrete and using the similar overstrength factors factor. The cracks of the concrete are not faster than that in the steel. The steel reinforcement should yield before that the concrete cracked. In the reversal loading the drop of the stiffness and strength of the concrete due to reversal load should be limited.

In order to protect the column core from the sudden shock it is important to insure that the resistance in the column beam intersection core, remains in any case above the transmitted loads, moments, shear, from the columns, and beams.

13.2.3.4 Design of the beams

The calculations of the flexure and shear resistance reinforcement have been carried out solving the equilibrium equations (7.16) and (7.17) for the HFC, and apply the stress strain relations given in Figure 13-13. The reinforced concrete beams are verified at the yield locations for the first level beams. And the capacity design requirements are applied at the columns beams connections.
The design procedure is introduced, and the design results are given in Table 13-3.The member ductility should full fill the ductility requirements according to equation (12.7) with T_1 = 0.783 s, and T_c = 0.5 s.

The demand ductility of the members is calculated as follows:

$$\mu_\phi = 2\times 3.9 - 1 = 6.8 \qquad \text{Using equation (12.7)}$$

The shear is calculated according to the equation (13.32). The ultimate shear strength is calculated as follows:

$$V_u = V_{Rb} + V_a + V_f \qquad (13.32)$$

Where:

V_{Rd}; is in term of participation of the concrete.

V_a; is the traditional term for the participation of the reinforcement.

V_f; is the term for the participation of the fibres.

In the case of using reinforced concrete the shear force is calculated as follows:

$$V_{Rb} = \frac{1}{\gamma_E}\frac{0.21}{\gamma_b}.k\sqrt{f_{cj}}b_0 d \qquad (13.33)$$

with k as follows:

$$\text{in compression} \quad k = 1 + \frac{3\cdot\sigma_{tm}}{f_{tj}} \qquad (13.34)$$

$$\text{and in tension} \quad k = 1 - \frac{0.7\cdot\sigma_{tm}}{f_{tj}}$$

The design shear of the beams considered is carried out considering possible plastic hinge formation at the beam-ends using the following relation

$$\text{Shear strength} \geq \gamma\cdot\frac{M_{p1}+M_{p2}}{l_p} + \frac{W}{2}$$

With $W = (DL + 0.15LL)\cdot L_{beam}$ Using equation (5.6)

Where $M_p = M_u$, and M_u are given in Table 13-6 for each beam end. The minimum transverse reinforcement is required, which is sufficient to prevent bar buckling. As follows

$$\begin{aligned} &S_w = \min(h_w/4,\ 24d_{bw},\ 7d_b,\ 200\text{mm}) \\ &S_w = \min(600/4,\ 24\times 6,\ 7\times 12,\ 200\text{mm}) \\ &\phi 6\text{ mm } @S_w = 85\text{ mm in the critical region} \\ &\phi 6\text{ mm } @S_w = 150\text{ mm outside the critical region} \end{aligned} \qquad (13.35)$$

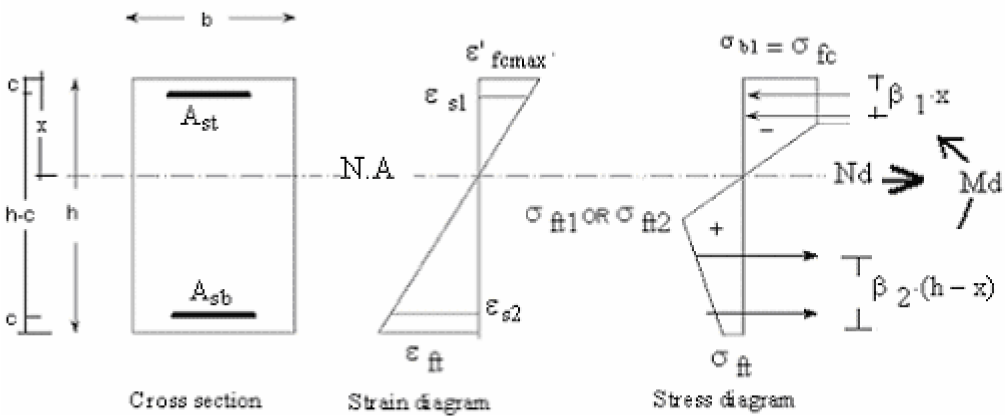


Figure 13-13: Beams section, with stress strain diagram at yield and ultimate state

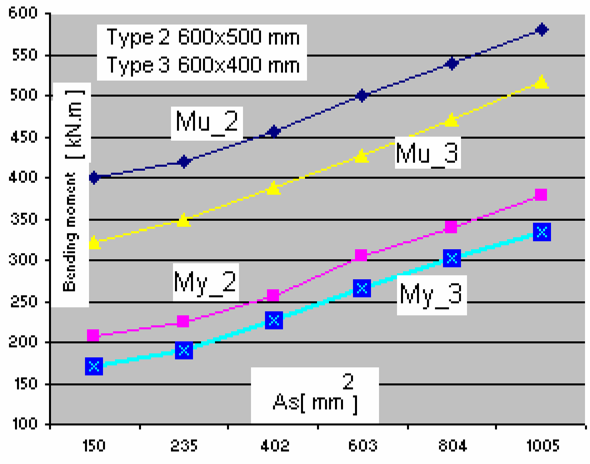


Figure 13-14: The ultimate and the yield moment rectangular HFC sections.

13.2.3.5 Columns beams connection design.

In this section we design and control the connection system at the nodes N6 and node N7 refer to Figure 12-11. The column beam connection at N6 is semi rigid connection and the yield location is at the nodes N33 and N34. The required reinforcement is obtained using the non-linear analysis. This reinforcement is verified applying the essential principles of the capacity design. That means we chose reinforcement proportions at these nodes in such a manner that leads to the delay of the yield formation at the column beam connection.

The design proceeds is carried out with the following steps.

(1) Design the cross sections at these locations and chose the reinforcement according the loading requirements.
(2) Control the yield moments at these locations consider an overstrength factor of 1.2 to insure that the yielding occurs in the beams M47 and M48 at nodes N33 and N34 before node N6.

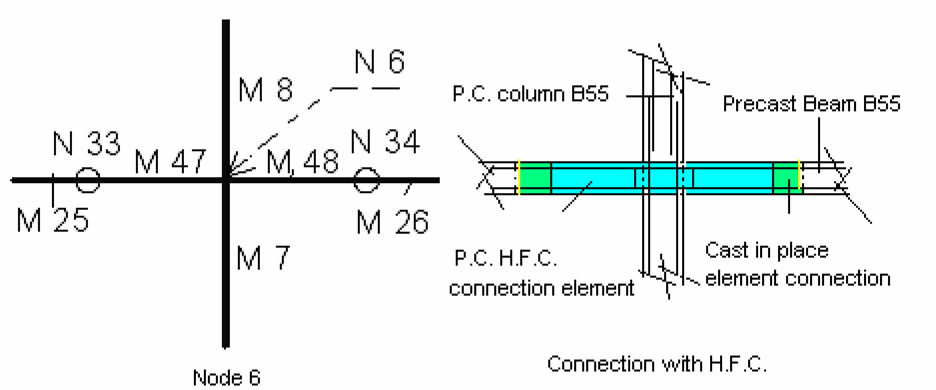


Figure 13-15: The connection with HFC, node N6

The solution of the non-linear equations (7.15 to 7.16) is carried out with the following considerations:

The ultimate strain of the tension reinforcement, $\varepsilon_{su} = 2.75\%$
The yield strain of the tension reinforcement, $\varepsilon_{sy} = 0.2175\ \%$
The ultimate strain at the columns beam connection is 0.33 %. This is applied to insure that the yielding do not leads to exceed the tension resistance of the HFC in the columns beam connection. The beams M47 and M48 dimensions and loading are given in Table 13-5.

Table 13-5: Loads, reinforcement at the beam column connection N6 and

Member /Node		M_d [kN.m]	Steel reinforcement A_{st} / A_{sb}	M_u [kN.m]	M_y [kN.m]
M47, M48 700×500mm	N6	500	5ϕ16 / 2ϕ16	520	280
	N33, N34	293	2ϕ10	450	*242

Verify that the yield moment at the yield location N33 is less than the yield moment at the column beam connection N6.considering the over-strength factor and the proportion of the applied loads as follows:

$$M_{y\_location} \leq \frac{280 \cdot \frac{552}{1.15}}{500} = 269 \text{ kN.m, and that is ok} \qquad \text{Using equation (12.5)}$$

Where: $M_{y\_location}$ = *242 kN.m

It is recognized that the moment at the section of HFC varies rapidly with the reinforcement change. And at the columns beams connections we can add extra flexural reinforcement, due to the low percentage of the steel reinforcement in the core. This reinforcement increases the moment resistance of the core and prevent cracks and splits failure at the core.

13.2.3.6 Capacity design and control with HFC

The column beam connections of the emulated monolith structure is designed and verified using the capacity requirements Figure 13-16. And as it have been discussed earlier in chapter 8. The essential of this procedure is to insure the stiffness required in the columns for the seismic actions of the beams. In one direction of the seismic action the top of a beam at the connection will work in tension, and the bottom face will work in compression. As the seismic action is reversed; the seismic force comes from the inverse direction, the same column beam connection functions inversely. The bottom of the beam will work in tension while the top of the beam works in compression. This point should be considered in the design and the control. In our solution it takes place when we choose the values of resistance of the beams in as it is shown in Figure 13-17. The loading of the seismic force acts in the same manner. As the seismic action acts as the same mariner as, and due to the geometrical symmetry of the structure we obtain the analyses results for the exterior joints N5 and N8, as it is seen in Figure 13-18. At the node N5 the bending moment exerted tension at the outside column M2 is –151 kN.m, and the bending moment from the reversed seismic action exerted tension at inside of the column M2, can be considered equal to 194 kN.m, as it is given on member M20. By this method we can use the analysis result considering one direction of the seismic action considering the symmetry conditions to obtain the loading on the elements at the columns beams connections.

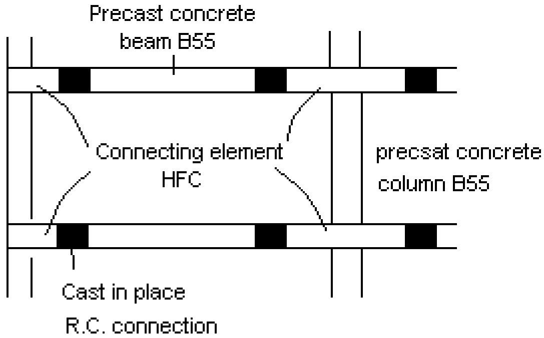


Figure 13-16: Design with hybrid-fiber concrete

(1) The exterior column beam connection at node N5 and N8

The design have bean done using simple computer program apply the equations (8.2 to 8.4) chapter 8, which based on EC8 regulations. In order to apply these equations the resistance moments of the element at this connection are to be calculated. Using the reinforcement given in Table 13-6. The capacity control of the exterior column beam connection is carried out as follows:

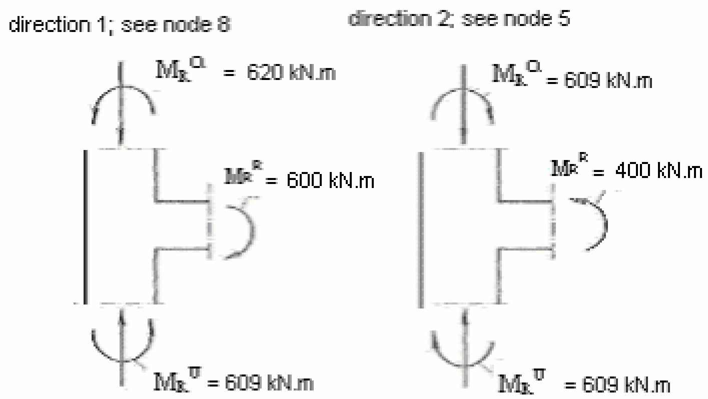


Figure 13-17 Direction of the loading and resistance at beam column connection

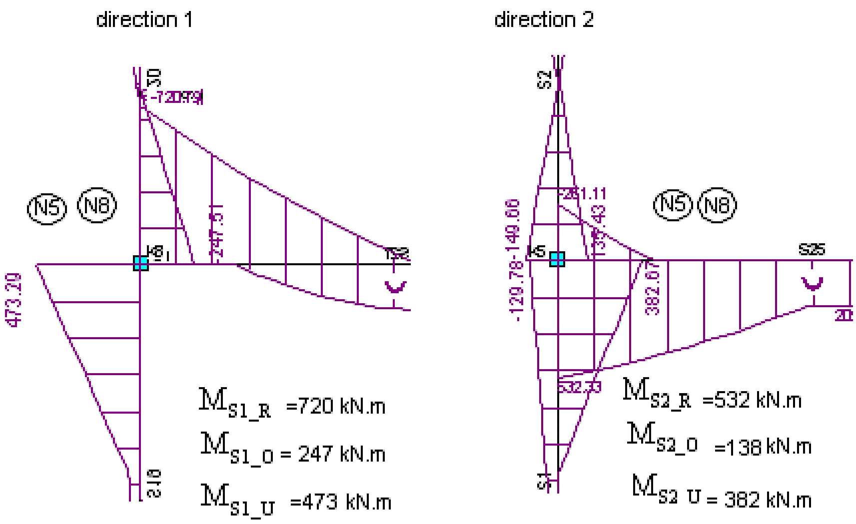


Figure 13-18 Bending moment diagram at node N5 and N8

The column strength multiplication factor and the reversal moment factor may be calculating with moment resistance of the columns and the beam (Figure 13-17), and the applied loads (Figure 13-18).

In direction 1 $\alpha_{cd1} = \gamma_{Rd} \cdot \dfrac{M_{R1\_L} + M_{R1\_R}}{M_{S1\_O} + M_{S1\_U}} = 1.2 \dfrac{0 + 600}{247 + 473} = 0.83$

Using equation (5.2)

In direction 2 $\alpha_{cd2} = \gamma_{Rd} \cdot \dfrac{M_{R2\_L} + M_{R2\_R}}{M_{S2\_O} + M_{S2\_U}} = 1.2 \dfrac{0 + 400}{138 + 382} = 0.77$

The relative importance of the gravitational with respect to seismic load is measured through the so-called moment reversal factor δ as follows:

In direction 1 $\delta_1 = \dfrac{\left|M_{S1\_R} + M_{S1\_L}\right|}{\left|M_{R1\_R}\right| + \left|M_{R1\_L}\right|} = \dfrac{720 + 0}{600 + 0} = 1.2$

Using equation (5.3)

In direction 2 $\delta_2 = \dfrac{\left|M_{S2\_R} + M_{S2\_L}\right|}{\left|M_{R2\_R}\right| + \left|M_{R2\_L}\right|} = \dfrac{532 + 0}{400 + 0} = 1.33$

The required moment resistances of the upper column in direction 1 and in direction 2 for the upper and the lower columns can be obtained using equation (5.4)

For direction 1

$$M_{Sd1\_cd} = \left|1 + (\alpha_{Cd1} - 1)\partial_1\right| \cdot M_{Sd1} \le q \cdot M_{Sd1}$$

Over column

$$M_{Sd1\_cd} = \left|1 + (0.83 - 1)1.2\right| \cdot 274 \le 0.8 \cdot 274 = 218 \text{ kN.m}$$

Under column

$$M_{Sd1\_cd} = \left|1 + (0.83 - 1)1.2\right| \cdot 473 \le 0.8 \cdot 473 = 378 \text{ kN.m}$$

Using equation (5.4)

or direction 2

$$M_{Sd2\_cd} = \left|1 + (\alpha_{Cd2} - 1)\partial_2\right| \cdot M_{Sd2} \le q \cdot M_{Sd2}$$

Over column

$$M_{Sd2\_cd} = \left|1 + (0.77 - 1)1.33\right| \cdot 138 \le 0.69 \cdot 138 = 96 \text{ kN.m}$$

Under column

$$M_{Sd2\_cd} = \left|1 + (0.77 - 1)1.33\right| \cdot 382 \le 0.69 \cdot 382 = 265 \text{ kN.m}$$

The required column resistance is M_{R2_O} = $M_{R2_}$u = 609 kN.m < M_{Sd1_cd} = 378 kN.m and that is Ok.

(2) The interior column beam connection at node N6 and N7

The design and control of the column beams connection at the nodes N6 and N7 is carried out using the same procedure used in the exterior column beam connection N5. In our structure because of the symmetry the control is sufficient using one direction of seismic loading on the interior connections. The bending moment in the column beam connection information (Figure 13-20 and Figure 13-19) are used in the capacity control as follows:

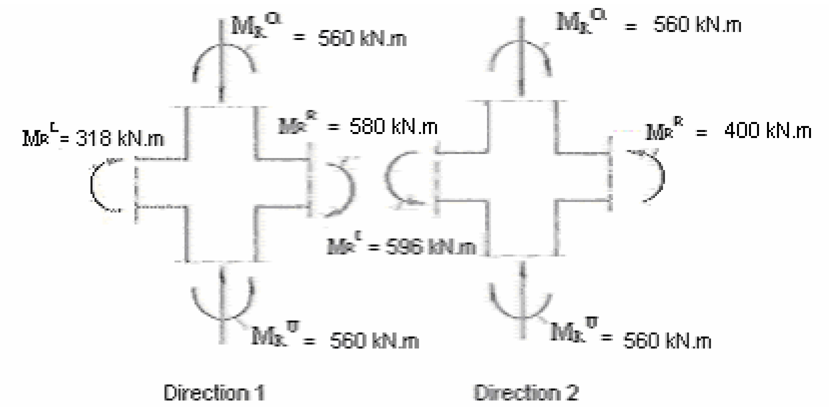


Figure 13-19 Strong column weak beam connection at the interior connection N6 and N7

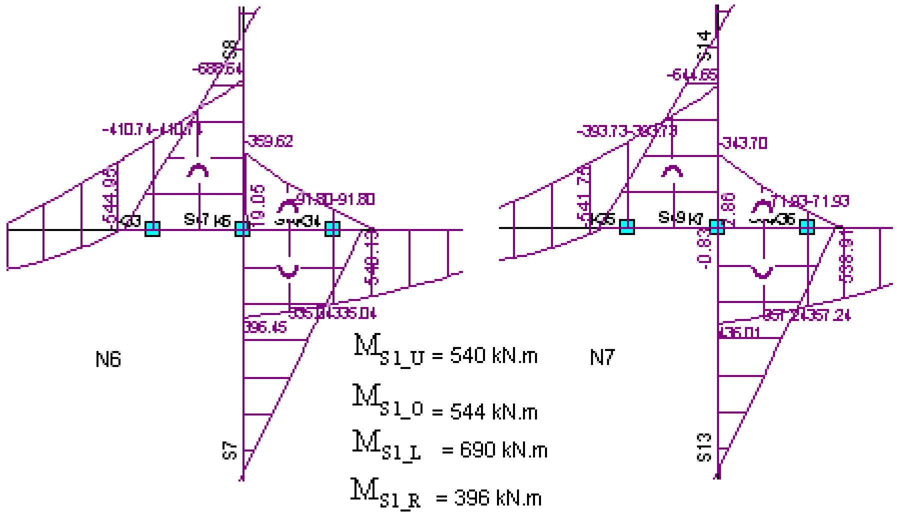


Figure 13-20 Bending moment diagram at node N6 and N7

The column strength multiplication factor is calculated as follows:

In direction 1 $\alpha_{cd1} = \gamma_{Rd} \cdot \dfrac{M_{R1\_L} + M_{R1\_R}}{M_{S1\_O} + M_{S1\_U}} = 1.2 \cdot \dfrac{400 + 580}{544 + 540} = 1.08$ Using equation (5.2)

Considering the reversal moment factor is calculated as follows:

In direction 1 $\delta_1 = \dfrac{\left| M_{S1\_R} + M_{S1\_L} \right|}{\left|M_{R1\_R}\right| + \left|M_{R1\_L}\right|} = \dfrac{\left| 396 + 690 \right|}{580 + 400} = 1.11$ Using equation (5.3)

The required moment resistance of the upper and the lower columns are calculated as follows:

For direction 1

Over column

$M_{Sd1\_cd} = \left|1 + (1.08 - 1)1.11\right| \cdot 540 \leq 1.09 \cdot 540 = 588 \text{ kN.m}$ Using equation (5.4)

Under column

$M_{Sd1\_cd} = \left|1 + (1.08 - 1)1.11\right| \cdot 544 \leq 1.09 \cdot 544 = 593 \text{ kN.m}$

The available resistance moments of the interior columns in both directions are given as follows:

$M_{R2\_O} = M_{R2\_}u$ = 608 kN.m.

The resistance moment of the columns is greater than the required design moment. And that is ok.

Table 13-6: the reinforcement with HFC at the beam column connection, and with B55 at the middle span beam.

Type: Member-Node	Loading [kN.m]	Flexure reinforcement [ϕmm]		Moment [kN.m]		Ductility
	$-M_{d\_top}$ $+M_{d\_bottom}$	A_{st}	A_{sb}	M_u	M_y	μ_ϕ
Type1: M25-N5 +	-524 +435 +407	4@20+2ϕ16 3ϕ16	2ϕ20+4ϕ16 8ϕ16	-532 +448	-434 +373	11.8
Type1: M4, M48 -N38 N6,N7	-295 +360 -666 +514	3ϕ16+1ϕ12 2ϕ16+5ϕ20 +1ϕ12	4ϕ16+1ϕ12 6ϕ16	-230 +294 -632 +420	-187 +240 -474 +344	11.7
Type1: M26 +	+387	3ϕ16	7ϕ16			
Type2: M28-N9 +	-480 +350 +394	5ϕ16+3ϕ20	4ϕ16+2ϕ20 8ϕ16	-505 +377	-423 +314	11.7
Type2: M29-N10 +	-465 +350 +402	5ϕ16+3ϕ20 3ϕ16	6ϕ16+2ϕ12 8ϕ16	-505 +374	-424 +314	11.7
Type3: M31-N13 +	-434 +282 +302	2ϕ20+3ϕ16+ 4ϕ12 3ϕ16	5ϕ16+1ϕ12 6ϕ16	-437 +302	-380 +221	11.87
Type3: M32-N14 +	-412 +273 +372	2ϕ20+4ϕ16 +2ϕ12 3ϕ16	5ϕ16+15ϕ1 2 7ϕ16	-429 +302	-360 +240	11.87
Type3: M34-N17 +	-385 +217 +365	4ϕ20+2ϕ16 +1ϕ12 3ϕ16	4ϕ16 7ϕ16	-401 +220	-336 +180	11.8
Type3: M35-N18 +	-354 +217 +360	2ϕ20+4ϕ16+ 1ϕ12 3ϕ16	4ϕ16+1ϕ12 7ϕ16	-401 +250	-336 +203	11.8
Type3: M37,M38- N21,N22 +	-383 +110 +362	2ϕ20+4ϕ16 +1ϕ12 3ϕ16	2ϕ16+1ϕ12 7ϕ16	-401 +141	-336 +115	11.9
Type3: M40-N25 +	-249 +50 +281	4ϕ16 2ϕ16	2ϕ16 5ϕ16	-220 +112	-180 +91	10
Type3:M41-N26 +	-203 +50 +285	3ϕ16+2ϕ12	2ϕ16 5ϕ16	-226 + 112 +274	-185 + 91	9.2
The transverse reinforcement is 2 hoop: ϕ 8mm @S_w = 112 mm in the critical region ϕ 8mm @S_w = 150 mm outside the critical region						

Conclusion

The columns are designed with minimum reinforcement, according to the requirements at the critical regions. The beams in the first floor are modified into section type 2, of 600×500 mm. The design with HFC provides high moment resistance with less reinforcement. The modification of the first floor beam dimension leads to decrease the exerted moment in the beam-ends on the column. And that is important to conform the capacity requirements.

14 Evaluation, building design summary and conclusion

Usage of precast concrete elements for building construction speeds up the construction process, with high quality. Non-seismic precast concrete structural system has been widely used in the world. They may cause catastrophic disaster if they are used for structure in high seismic zones. The development of seismic connections is essential. Extensive tests of members and sub assemblages with precast concrete connections were carried out in Japan since the 1980's by research institutes of construction industry. Based on those tests, the construction of moment resisting frame reinforced concrete building using precast concrete columns, beams, walls, or floor slabs have increased in the 80's and 90's. Most of those constructions adopt connection details emulating cast-in-place concrete action such as to assure that they should have equivalent seismic performance as monolithic concrete members. Those connections have almost the same detailing as ordinary R/C members, while minor modifications are made to eliminate difficulties in construction and achieve high productivity

14.1 Evaluation

The study discusses and applies seismic performance of connections in precast construction, and investigates different related subjects. The main subjects covered by the study can be summarised as follows:

- The precast constructions, its different types, system of connections, and the types of connections. It illustrates the developments of the solution for the ductility by using the ductile elements.
- The study used the different types of the connections in a 6-storey building. Each solution carried out for the whole building. The performance criteria have been applied using the general solutions of structural analysis. The criteria been applied for the specific solution according to its requirement for each type of connection. Comparison between the different connection types and the possibility its utilisation, within another solution is investigated.
- The earthquake phenomenon and its effects on the structure have been explained. The discussion includes the dynamic analysis methods used to predict the seismic actions on the building structures. Many factors affecting the seismic resistance of the concrete structures have been studied such as the cyclic loading, the degradation, and the damping.
- For the analyses a building under seismic loading, it is required approximate solutions for the structure. This study leads to a new method for the analyses of a precast concrete structure under the extreme loading as the earthquake. It is as an approach using the static analysis of the structure-equivalent shear method using effective stiffness- in elastic frame to approximate the dynamic non-linear analyses results. This method has been discussed and applied on a precast building structure, the dynamic analysis for the building designed for equivalent shear load using elastic with effective stiffness is carried out, the demand ductility, maximum bending moment resulting from the dynamic analysis lays within the design with equivalent shear method The study explains the future use and the extension of this method in the monolithic constructions.

- The capacity design method has been applied in the different solutions of the 6-storey building. This application is carried out for different types of connections, the monolithic with B55, the HFC concrete and the connections with ductile members. Each solution has its special capacity design application.
- In the design of the beams at the yield location non-linear behaviour of reinforced concrete sections is applied with a new method. This method has been applied for reinforced concrete and hybrid-fibre concrete. It uses new parameters (α_{non} ,β_{non},) describes the compression stress block intensity and its centre from the compressed face in a rectangular R/C section, and (α_{1},,β_1,α_2,β_2) describes the tension and the compression stress block intensity and its centre from the compression and the tension faces in a rectangular HFC section.
- Model for the precast concrete beam have been illustrated. The stiffness of the beam is an explicit function of the reinforcement ratio and the beams dimensions.

14.2 Structural analysis, design and seismic performance

For seismic actions, the design of reinforced concrete construction requires the distribution of the seismic shear force over the structure units and elements, the determination of local effects, and the performance of the elements for the seismic effect. The distribution of the shear force may be carried out through the choice of a structural system as follows:

- Frame system where both the vertical and lateral loads are mainly resisted by the space frame.
- Wall system where the loads are resisted by wall system coupled or uncoupled with high shear resistance.
- Dual wall frame system.
- Core system of wall or frame system without satisfactory torsion rigidity.
- Inverted pendulum system where 50% of its mass is located in the upper height of the structure.

The subsystem supports a proportion of the seismic force, which may be verified through the stiffness and the strength of its elements. And that requires a local ductility demand at specific places and especially near the connections where damage can be permitted.

The seismic lateral shear force is about inversely proportion to the global displacement ductility μ_δ of the structure building. It is economical to increase the global available ductility of the R/C structure, through a proper application of the capacity design procedure and the detailing of the members connected members. There is another argument in the seismic design lower ductility and higher strength. The higher is the strength of the structure, the smaller is the structural damage during an earthquake. It is obvious the design with the low ductility and high strength where damage preferred not to occur and use ductile properties in places where damage can be allowed. That may be applied as follows:

- Strengthen the columns beams connections, to prevent shear failure and ensure the flexural strength and the stiffness of the system connection.

- Bring the yield locations of the beams at a distance from the columns face, to decrease the degradation, the bar slip effect on the connection. The determination of the yield locations should be in places where the special details and requirements of the plastic hinge, reduction in the flexural reinforcement and additional transverse reinforcement can be applied.

The performance of the reinforced concrete for the seismic design should consider the non-linear behaviour through the update of the stiffness with an iteration process when using a non-linear analysis program, or by modifying the column beam connection stiffness when using a linear structural analysis program. The analysis schema is illustrated in Figure 14-1. The stiffness modification is given in section (14.2.1). The equivalent base shear method, which is based mainly on the first vibration period of the structure the seismic load, is computed as follows:

(a) Find the total lateral seismic force on the structure. This force is the sum of all seismic forces and can be considered as the shear force at the base.

$$F_b = S_d \cdot \frac{\lambda}{g} \cdot \sum_i W_i \qquad \text{Using equation (11.10)}$$

(b) Distribute the force on each floor of the building.

$$F_i = F_b \cdot \frac{z_i W_i}{\sum_i (W_i . z_i)} \qquad \text{Using equation (11.12)}$$

Two types of analysis are used. Elastic stiffness frame for ultimate and serviceability limit state. And elastic frame with modified linear effective stiffness, using seismic combinations (Figure 14-1) as follows:

- Model the structure as elastic stiffness frame; apply the ultimate and serviceability limit state combinations (model A).
- Model the structure as linear stiffness frame (model B), and apply the seismic loading combinations. The connection stiffness may be introduced in Model B to build a new model (Model C) with modified effective stiffness at the expected post yield locations using the seismic combinations.

The first model (A) describes the serviceability of the building through a specific code regulations, this model account for the dead, live and wind load (DL+LL+W) with elastic stiffness. The second model (Model C) describes the behavior of the structure under seismic loading; it accounts for the dead, live and seismic load (DL+LL+S), and uses the modified elastic stiffness. Model C is built in two steps as follows:

(1) Building Model B, which represents the structure in elastic stiffness as in (A), it used for the form modification and to obtain the modified elastic stiffness. It is recommended by many codes and literatures that the redistribution of the moment do not exceeds 35% of the beam end moments resulting from the elastic analysis. The reduction factor of 20% of bending moment in the beam-ends is used, where it represent different between the estimated ultimate and the yield beams strength in the beams, the beams start to yield at 80% from the design loads the demand ductility is assured in a range within its increase provided by of plastic hinge formation.

(2) Building model C, which define the behavior of the structure under the seismic action in its optimum yield condition for the seismic force, which is obtained by the equivalent shear method. It uses the modified elastic stiffness, which is obtained using the reinforcement and the analysis results of model A, and [K_{eff}, M] diagram.

The analysis process is shown in Figure 14-1. Description the modified elastic stiffness calculation is given in section (14.2.1.). Regardless of mechanism formation, a structure was assumed collapsed if the inter-storey drift at any locations exceeds a limiting value of 3%. It is founded that this criteria although it is rough make feasible for the time.

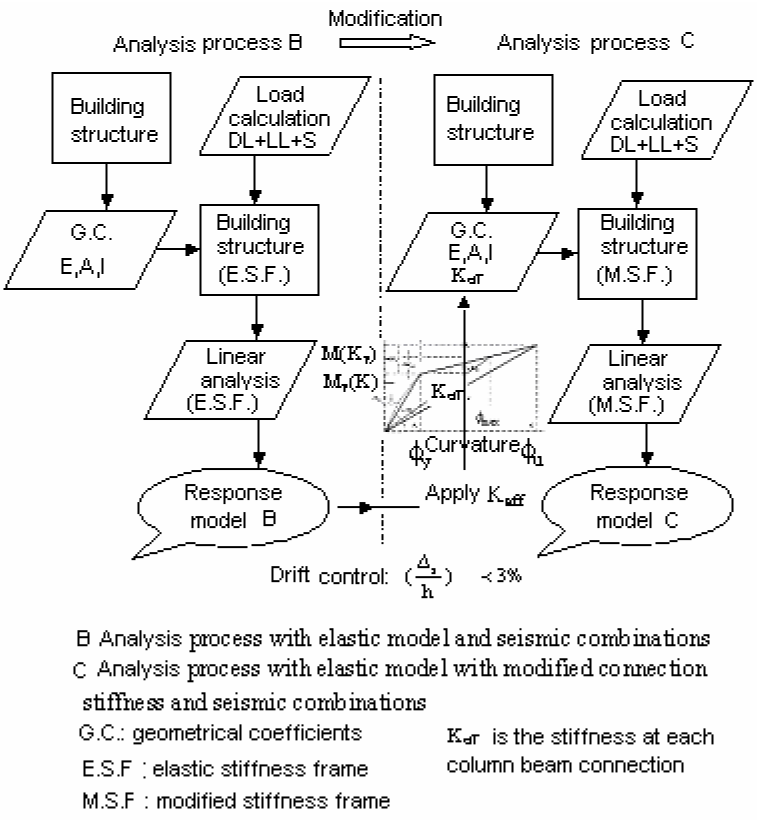


Figure 14-1: Analysis process with seismic loading

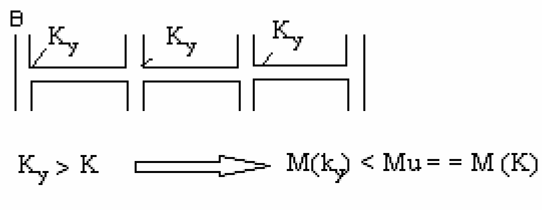
14.2.1 The effective stiffness

The effective stiffness for the modified elastic stiffness frame (Figure 14-1) can be obtained as follows:

- Step 1: Use the resultant moment from the combinations (D + L + S) from the rigid connection model with a reduction of 20 %.

  $M`(K) = 0.8 \times M\ (_{D+L+S})$

- Step 2: M` may be used to estimate a reinforcement level for the member and it will be the highest bending moment can be obtained in the postyield situation $M_u(k_y)$. It is expected that the moment will drop down due to decrease of the stiffness according to the loading. To obtain the bending moment in the beam-ends. The maximum moment at the beam-ends can be obtained design using the yield stiffness K_y, for the specific reinforcement ratio (Figure 14-1). That may be explained as follows: When substitute for the yield stiffness K_y in a structure results in a bending moment M (k_y) les than the bending moment when substitute in elastic model resulting moment M_u because $K_y < K$ and it is in the same time greater than the ultimate bending moment when apply the elastic stiffness.



And in the same time it is greater than the bending moment resulting from the substitute of the $K_{u.}$

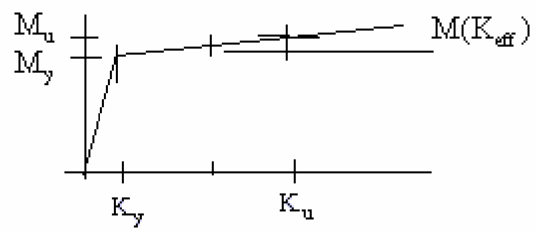
- Step 3: The new modified stiffness may be applied for the model obtaining semi rigid connection with the seismic combinations. These values can be modified according to the resulting moment and the section reinforcement. Obtaining $K_{eff.}$ Notice that for a given section reinforcement the increase of the stiffness in a member leads to more loading on the member and that leads to the decrease of the effective stiffness obtained using M-$K_{effective}$ diagram Figure 14-2. By this the stiffness converges at the given section and connection. In a design of a ductile location in a beam that can be considered acting simultaneously ductile connection and to assure that it reach the postyield behavior under the influence of the seismic action, the applied bending moment should be les than the ultimate moment considering damage factor so that no damage occurs and greater than the yield moment considering overstrength factor so that the section behave in yielding. That can be expressed as follows:

$$\frac{M_u}{D} \succ M(K_{eff}) > C.M_y$$

where:

C is the overstrength factor.

D is the damage factor.



- Step 4: The effective stiffness (K_{eff}) may be given explicitly as a function of the reinforcement ratio and the loading level (Figure 14-2) for specific R/C section. Substitute K_{eff} in a structural analysis model for certain bending moment and reinforcement ratio will result in new response, due to the redistribution of the load in the structure. And that need the choice of new $K_{eff,}$ accordingly the new K_{eff} is required to be determined for the new response, and that leads to structural, and material iteration problem (usage the graph figure - substitute and analysis with the structural model). In order to solve this problem. The behavior of the structure can be described through the relation of (K_{eff} –M). The description of the structure behavior can be carried out for a connection by introducing K_{eff} in the structure and monitoring the resultant bending moment for several times. The effective stiffness and the moment relations are shown in Figure 14-2, and how to obtain K_{eff} for the column beam connection N5. The bending moment resulting from the structure analysis at node 5.

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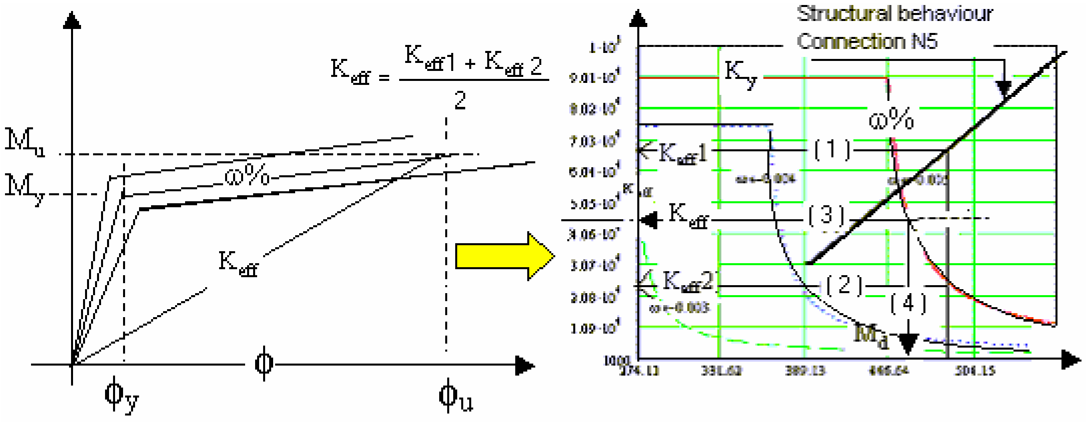

Figure 14-2: The effective stiffness in a local column beam connection, and the behaviour of the R/C section registered at connection N5

The effective stiffness may be obtained using Figure 14-2 as follows:

(1) For a certain bending moment M_d
   (a) Follow line (1), intersect with the structure behaviour line and obtain $K_{eff}1$.
   (b) Follow line (2), intersect with the section property line for certain reinforcement ratio and obtain $K_{eff}2$
(2) The effective stiffness is the average of the $K_{eff}1$ and $K_{eff}2$ obtained on line $K_{eff} = (K_{eff}1 + K_{eff}2) / 2$
(3) The expected bending moment M_d can be read on line 4

- In the prefabricated constructions design parts of the dead loads on the supporting elements may be considered as initial loading, where the beams are erected first and it is designed as simply supported member caries its own weight or a part of the floor dead loads. The reduction of the bending moment in the beam ends in the building designed in this thesis may be allowed up to $0.4M_0$ when considering apart of the dead load of the beams and part the roof is carried by the simples supported beam. (Table 14-1).

Table 14-1: Possible for reduction, loads at the current floor in the building structure, chapter 9, with M_0 is the dead load of the floor and the beam

Construction type	Dead load [kN/m]	M_0 [kN.m]	Fixed connection ~0.66 M_0 [kN.m]	Partially-Fixed ~0.25M_0 [kN.m}	Reduction ~ 0.4M_0 [kN.m]
Precast concrete	51	229	151	57	91
$0.4M_0$ is calculated for beams carries its weight and a part of the floor as simply supported beam.					

- Step 6: Designing with rigid and semi rigid connections may be carried out using the effective stiffness as follows:

The construction with beams, where yielding is considered in the beam-ends first and the floor reinforcement participate in the seismic resistance later.

$$K_{eff} = \frac{6\,E \cdot I}{L} \text{ Or the } K_{eff} = K_y \tag{14.1}$$

For non-rigid connections, non-resistant members according to the most codes are designed with stiffness as follows:

$$K_{eff} = (0.4 \text{ to } 0.5) \cdot \frac{6\,E \cdot I}{L} \tag{14.2}$$

In a required ductile member the available ductility of the member should be greater than the demand ductility given by the analysis. That can be verified using dynamic nonlinear analysis program and in this study the nonlinear Ruaumoko program is used. Premier estimation for the demand local ductility may be obtained using the global ductility of the structure, and the geometrical relations, for the displacement and the rotation. It can be obtained using linear structural analysis program, after the application of the maximum ultimate and the yield displacement on the structure and find the local ultimate and yield rotation for the concerns members. The local ductility of the structure may be estimated using codes of practice. The local curvature ductility of the building structure may be carried out using equation (12.6) according to EC8. The Local ductility requires high plastic rotation capacities in potential plastic hinge region: Sufficient curvature ductility (Post failure 85% moment resistance level) in all critical regions of primary elements. The local ductility should be mobilized for the demand ductility, where the beams span is long in comparison with the columns length. In cases that the section reinforcement is critical for the demand ductility, it can be ensured using additional confinement reinforcement.

$$\mu_\varphi = 2q - 1 \quad \text{if} \quad T_1 \leq T_c$$
$$\mu_\varphi = 1 + 2(q - 1) \cdot T_c/T_1 \quad \text{if} \quad T_c < T_1 \qquad \text{Using equation ((12.6))}$$

Based on the following relations:

$$\mu_\varphi = 2\mu_\delta - 1 \quad \text{and} \quad \mu_\delta = q \quad \text{if} \quad T_1 \geq T_c$$
$$\mu_\delta = 1 + (q - 1) \cdot T_c/ T_1 \qquad \text{if} \quad T_1 < T_c$$

The determination of the stiffness in a connected member depends on the structure geometry, the manner of connection, the performance of the section. It is the designer's decision also to decide the appropriate distribution and continuity of the forces in a specific location. Regarding the building geometry, the enclosed spaces. The stiffness can vary according to the connection types and their ability to transmit loading and availability for ductile behavior. In general the effective stiffness for the connecting concrete members designed as seismic resistance or non-seismic resistance members used in the construction is given as follows:

$$\frac{4E \cdot I}{L} \cdot (0.1 \text{ to } 1.4) \tag{14.3}$$

14.3 Summary of the building design

The precast concrete of 6-storey building is designed with reinforced concrete type B55. The column beam connections are designed as: emulated monolithic R/C B55, Hybrid-fibre concrete HFC, Dywidag ductile connectors DDC, and Post-Tension assemblage. In the structural analysis a modified elastic model is used as an approach for the non-linear behaviour of the reinforced concrete structure. In the analysis a model describing the non-linear behaviour of the R/C sections is used to control the yield locations at the beams. The design of the connection fulfils the seismic requirements, the different material and connection types. Capacity design control is carried out for the connecting elements according to EC8.

14.3.1 The building shape and dimension

The building structure consists of (3×6m span) frames at 7.8m. The frames support a floor of (350mm) thick. The structural system is based on the concept that:

- The precast concrete units should be as long as it is possible for the transport and the erection. Hollow core p.c. of (7.8m) span is used in the floor design.
- The bending moment in beams (1) of 6m spans in the transverse direction is smaller than that in the beams (2) of 7.8m in the longitudinal direction. It means designing the structure so that the (6 m) span frame in the transverse direction supports the floor dead and live (+ seismic loading), and the frame of (7.8m) span in the longitudinal direction supports partially (1.2m of the floor width) the dead and live of the floor (+ seismic loading) (Figure 14-3).

The structure is designed to function under the service loading, and possess a reserve capacity for the extreme loading, due to the seismic actions. The dimensioning of the building have been decided for, the dead, live, and wind load; these dimensions have been modified to resist the seismic action. Redistribution with 20% reduction of the bending moment in the connections is used. The material used of high strength concrete B55, which allows designing using different materials and connection, types without essential modifications in the structure. Loads on the building and the material are illustrated in Table 14-2.

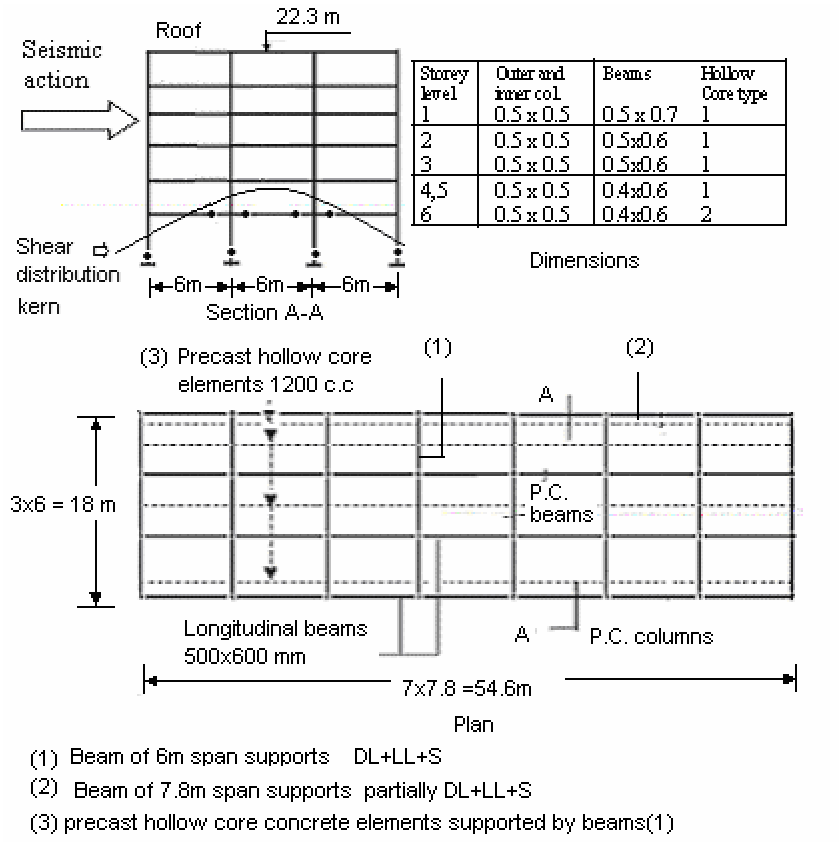

Figure 14-3: Six-storey building, cross section and details

Table 14-2: Load and material properties

<table>
<tr><td colspan="6">(A) Load</td></tr>
<tr><td colspan="2">Location</td><td>DL [kN/m]</td><td>LL [kN/m]</td><td>Seismic load</td><td>Wind load</td></tr>
<tr><td colspan="2">Basic</td><td></td><td>2.4</td><td rowspan="6">Considered region of subsoil class B, zone III in Greece, $A_g = 0.24$ g.</td><td rowspan="6">0.8 [kN/m^2]</td></tr>
<tr><td colspan="2">Allow for corridor</td><td></td><td>1.2</td></tr>
<tr><td colspan="2">Superimposed dead load</td><td>1</td><td></td></tr>
<tr><td colspan="2">Ceiling</td><td>0.5</td><td></td></tr>
<tr><td rowspan="2">Total</td><td>Current floor</td><td>1.5</td><td>3.6</td></tr>
<tr><td>Roof</td><td>0.5</td><td>1</td></tr>
<tr><td colspan="6">(B) Material</td></tr>
<tr><td>Concrete</td><td>f_{ck} [N/mm^2]</td><td>$f_ç$ [N/mm^2]</td><td>Steel</td><td>$f`_y$ [N/mm$^2$]</td><td>$E`_s$ [N/mm^2]</td></tr>
<tr><td>B55</td><td>55</td><td>33</td><td>FeB500</td><td>435</td><td>200000</td></tr>
<tr><td>HFC</td><td>75</td><td>Stress strain diag.</td><td></td><td></td><td></td></tr>
</table>

14.3.2 Loads

The lateral seismic force distribution is generally based on the static equivalent lateral forces specified in building codes. EC8 determine the seismic effect on the basis of the response spectrum method using an elastic model of structure and by introducing a behaviour factor. The building location is considered in Greece, zone III, and considering the location and building characteristics, the seismic acceleration A_g is 0.24g. In the calculation and distribution of the seismic force on the building the equivalent shear method is used, the force is distributed on each floor of the building depending on the building mass and height according to (11.13). The dead, and live load of the building are calculated considering the transverse frame of 6m spans is the main support.

$$F_i = F_b \cdot \frac{z_i \cdot W_i}{\sum_i (W_i . z_i)}, \text{ With } F_b = S_d \frac{\lambda}{g} \sum_i W_i, \ \lambda_i = \frac{z_i \cdot W_i}{\sum_i (W_i . z_i)}$$

And $S_d = a_g . S \frac{2.5}{q} (\frac{T_c}{T})$ For $T_c \leq T \leq T_d$; Using Equation. (11.8) through (11.13)

The seismic and wind loads are distributed on the transverse frames where the period $T=0.783>T_c=0.5$, and the frequency is higher in the transverse direction higher response (S_d) in the transverse direction and the transverse frames beams carries the higher percentage of the dead and live loads. The floor system serve as rigid diaphragm (18m deep) between the vertical elements of the lateral force resisting system, thus the columns undergo the same drift, and because of the equal column inertia and length at each floor, these frames are considered to resist equal seismic forces. The seismic, dead and live loads are illustrated in Table 14-3. The seismic shear forces on the first floor is high, the effect of the ground floor on the first floor also more than in other floors. That should be considered in the dimensioning and the premier design.

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Table 14-3: Seismic force on the main frame of 3x6m spans, dead and live load on the beams

Seismic load [kN]					
Floor number	Story DL	Story LL	Seismic Load $G+\psi_E Q$	Seismic Force Force	Shear force
Roof	9732	983	10027	210	210
5th	10715	3538	11246	198	408
4th	10715	3538.	11246	161	568
3rd	10715	3538	11246	124	692
2nd	10715	3538	11246	87	779
1st	10715	3538	11246	50	828
The total seismic shear force on the main frame=828 kN					

Dead and live load on the beams		
Location	DL.[kN/m]	LL.[kN/m]
M25, M30, M45, M50	55.12	25.74
M40, M42	47.98	7.15

The non-linear program (Ruaumoko) is used two with two different acceleration records; El Centro May 1940 North-South Component (0.32g), and Bucharest 1977, North-South component scaled to the design shear force (scale = 0.24g/0.32 = 0.75). The two records have different peak acceleration time accelerations to cover the stochastic vibration of the building.

14.3.3 Structural analysis

Two models are used for the structural analysis of the building. The first is the linear analysis model with elastic stiffness. The second is linear analysis with a modified effective stiffness model. In the analysis two types of combinations are used. The first combinations covers the ultimate and serviceability limit state (DL+LL+W). The second accounts for seismic action (DL+LL+S). As the moment on the connections increased, decrease the effective stiffness of the beam end considerably, for that modified effective stiffness at each connection is used, its value is a function of the reinforcement at the section and the estimated applied loading. The modified effective stiffness is obtained using the effective stiffness and moment relations and its variation on the structure through the substitution in a conventional linear program. According to the design requirements, the drift of the current and the drift of the top floor is les than the permitted drift $\Delta/h \leq 3\%$. The shear force on the building is distributed on the interior and the exterior columns in a manner lead to decrease the moments at the exterior columns. Where using rigid connection at base of the interior column and flexible connection at the base of the exterior columns Figure 14-4. The yield location at the first floor beam and ground floor column provides flexibility in the structure is verified applying flexibility at the first floor beams and different degree of fixation for the ground floor columns). The modified stiffness model is used to optimise the design in combination with the non-linear program Ruaumoko.

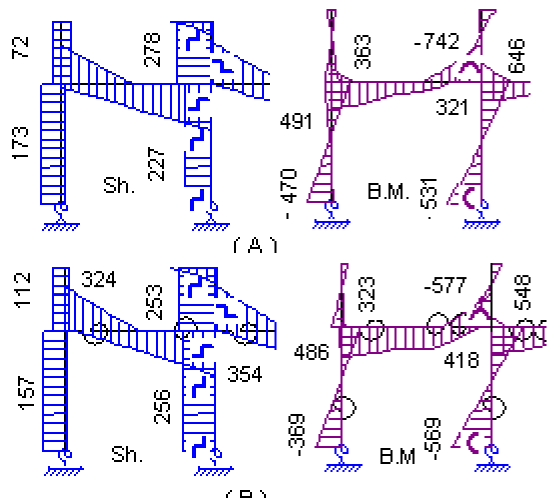


Figure 14-4: Shear and moment diagram at the ground and first floor columns

14.3.4 Reinforced concrete elements design

The design of the concrete elements is carried out for the four connection types (Figure 14-5) using the analysis results of the elastic frame model. The design is based on the monolithic connection as the principle choice. The alternative designs are the connection with ductile elements -the DDC connection, the post-tension assemblage-, and the connection with HFC. The design of the column and the beams has been carried out using the structural analysis results, and the reinforced concrete element properties. All the elements, beams columns, floor, and the connections should perform the seismic design requirements. The design process of the R/c elements used in our study is illustrated in Figure 14-6. The columns beams connections shares the beams and the columns properties, the design process take its way through the analysis of the structure as a system and the design of the connecting elements, the column and the beams, the connection properties follows the decisions in these elements. The specific configuration for each type of connection has been considered. The choice of the ductile members and the post tension members in the connections may be followed with the modifications at the column and beam-ends, and apply the capacity requirements within the connection design. In the emulated concrete and the HFC connection, the design of the columns and beams end, should be followed by the capacity control.

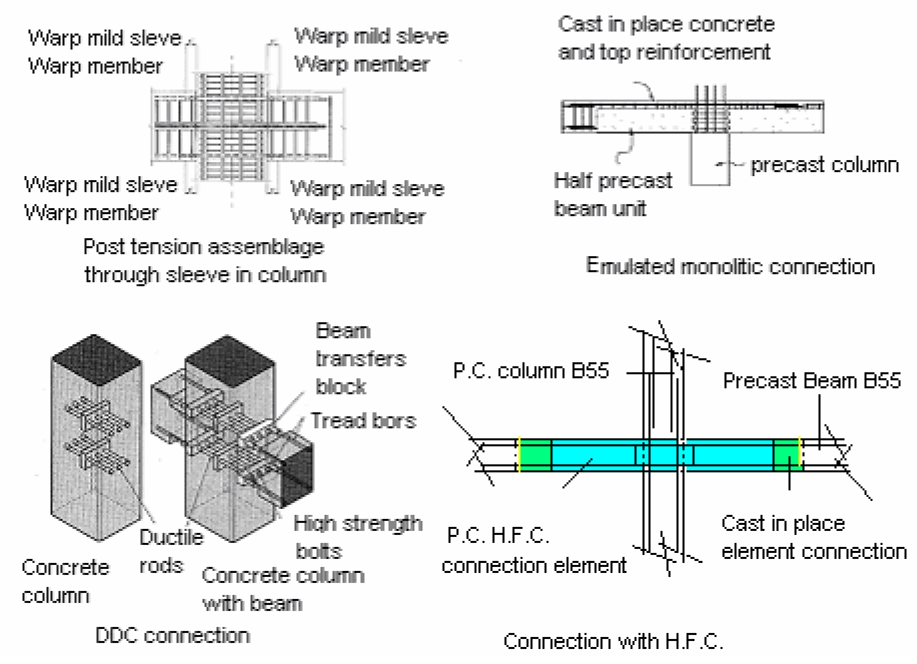


Figure 14-5: The building structure with different column beam connections types

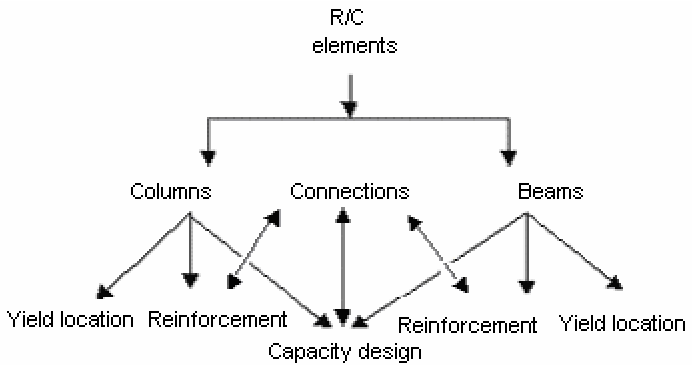


Figure 14-6: Design process of the reinforced concrete elements, the connections shares the beam and the columns design.

Design with monolith reinforced concrete

(3) Design of the columns

The designed of the columns consider three activities:

- Premier design for the reinforcement at the critical regions in the columns, this may be modified according to the capacity design requirements.
- Control the moment curvatures and the available ductility at the yielding locations.
- The capacity control at the columns beams connections, capacity design of the columns should follows the design of the beams, where the moment resistance of the beam can be obtained according to the needed reinforcement in the beams

The calculation of the reinforcement in the columns was carried out using equations (12.1) and (12.2), and the CEB 1982 coefficients.

$$\mu = \frac{M_d}{b \cdot h^2 \cdot f_c}, \quad \nu = \frac{N_d}{b \cdot h \cdot f`_c}$$ Using equation (12.1), and (12.2)

Equations (12.1) and (12.2) are applied for different loading cases at each section to obtain the required reinforcement. Where the moment and the normal force at each case can have different proportions according to the different loading. The ultimate and the yield moment of the columns obtained using the equations (7.7) though (7.10)

$$\begin{aligned} m_d = \frac{M_d}{b \cdot h^2 \cdot f'_c} = \psi_1 \cdot \frac{\sigma_{s1}}{f_s} \cdot \left(\frac{1}{2} - \frac{c}{h}\right) \ldots \\ + \alpha_{non} \cdot k_x \cdot \left(1 - \frac{c}{h}\right) \cdot \frac{\sigma_{c1}}{f'_c} \cdot \left[\frac{1}{2} - \beta_{non} \cdot k_x \cdot \left(1 - \frac{c}{h}\right)\right] \ldots \\ + \psi_2 \cdot \frac{\sigma_{s2}}{f_s} \cdot \left(\frac{1}{2} - \frac{c}{h}\right) \end{aligned}$$

Using equation (7.7) through (7.9)

The nominal shear on the column is calculated in using equation (12.3) as follows:

$$V_{sd} = \gamma_{Rd} \frac{M_{R\_u} + M_{R\_o}}{L_c}$$

Using equation (12.3)

The design of the shear reinforcement at the critical length of the columns is carried out using equation (12.4) as follows:

$$V_{cd} = b_w d \cdot (\tau_{Rd} \cdot (1.6\text{-}\ d)(\ 1.2 + 40\rho_1 + 0.15\sigma_{cp})$$

with

Using equation (12.4)

$$\sigma = \frac{N_v}{b_w h} \text{ and } \rho_1 = \frac{A_s}{b_w h}$$

The columns satisfied the flexural moment with minimum reinforcement at the critical regions. The reinforcement, due to constructive purposes can be use the same reinforcement at each face of the column outside critical regions.

The ground floor columns are longer than the other floors columns. And the first floor is subjected to higher lateral seismic force than the other floors. For that the bending moments subjected on the first floor beams are great, the first floor beams to be more massive than other floors. In order to achieve reasonable dimensioning for the beams and especially the usage of similar ductile members in the connection joint required redistribution of the loading on the first floor. And using yields locations at the first floor connections near the beam-ends, and using rigid connection at base of the interior column and flexible connection at the exterior columns. Shear resistance at the locations have been verified. The shear reinforcement in these locations is the confinement reinforcement and the longitudinal bent bars perform the seismic requirements.

(2) Design of the beams

The calculations of the flexure, and shear reinforcement, and shear force for the beams have been carried out using equation (12.8) and (5.6) as follows:

$$A_s = \frac{M_d}{0.9 \cdot d \cdot f_s}, \quad V_{Rd} = V_{cd} + V_{wd}, \quad \text{Shear strength} \geq \gamma \times \frac{M_{p1} + M_{p2}}{l_p} + \frac{W}{2}$$

With $W = (\ DL + 0.15 \cdot LL\ ) \cdot L_{beam}$

Using equation (12.8) and (5.6)

Shear design of the beams includes the control of the possible plastic hinge formation at the beam-ends. This is carried according to EC8.

The design shear of the beams considered is carried out considering possible plastic hinge formation at the beam-ends as follows:

$$\text{Shear strength} \geq \gamma \cdot \frac{M_{p1} + M_{p2}}{l_p} + \frac{W}{2}$$

With $W = (DL + 0.15 \cdot LL) \cdot L_{beam}$ Using equation (5.6)

Where $M_p = M_u$

To insure the flexural strength ratio in the beams- the degree of joint confinement, development length of bars and joint shear stress are factors to be considered in the design, and to achieve adequate strength and ductility in the joint. In the cyclic loading the degradation and the slip of the bars in the beams, stress penetration reduces the joint strength and ductility, as the yield locations brings away from the columns face, these undesirable effects are reduced and especially in chore of the column beam connection.

The reinforcement in the yield locations is assumed at the base connection. In the other floors additional light reinforcement added to the columns, so that the yield location is deviated from the columns beam connection faces. Equations (7.7) through (7.10) which describe the nonlinear behavior of the r/c section are used to calculate the ultimate, the yield moments, and the ductility of the beams at yield locations.

The members are verified to full fill the ductility requirements. In the first assessment for the ductility, we may obtain the global ductility of the structure using equation (12.7), and according to CEB, q is an estimation of the global ductility, which describe in general the behavior of the structure beyond the yielding in terms of the ductility. In our study the local ductility has been verified finally, using the nonlinear dynamic analysis program Ruaumoko, The demand ductility of the beams varies from 4 to 5 while the available ductility varies around 9.

$\mu_\varphi = 2 \cdot q - 1 \quad \text{if } T_1 \leq T_c$

$\mu_\varphi = 1 + 2 \cdot (q - 1) \cdot T_c/T_1 \quad \text{if } T_c < T_1$ Using equation (13.4)

The reinforcement in the beams varies from high reinforcement ratio at the first floor to low reinforcement ratio at the roof refer to Table 14-4

a. Capacity design

The essential of the capacity design is to insure the strength required in the columns for the seismic actions of the beams -weak beam strong column connection-. In one direction of the seismic action the top of a beam at the connection will work in tension, and the bottom face will work in compression. As the seismic action is reversed; the seismic force comes from the reverse direction, the same column beam connection functions reversal. The capacity design carried out on the connections using Equation (5.2) through (5.4) according to EC8. The control leads to the conclusion that, in case of the proper design for the beams, as the necessary dimensioning, the reinforcement, and the proportionality of the columns beams inertia. The beams may not exert high moments on the columns, which leads to full fill the capacity requirements without significant modifications in the columns reinforcement. In our building the design reinforcement of the columns at the critical regions is the minimum reinforcement. The sum and the moment ratio factor α, the moment reversal factor, and the capacity required moment M_{sd} are given in equation (5.2) through (5.4).

In direction 1 $\alpha_{cd1} = \gamma_{Rd} \cdot \frac{M_{R1\_L} + M_{R1\_R}}{M_{S1\_O} + M_{S1\_U}}$ In direction 1 $\delta_1 = \frac{\left|M_{S1\_R} + M_{S1\_L}\right|}{\left|M_{R1\_R}\right| + \left|M_{R1\_L}\right|}$

In direction 2 $\alpha_{cd2} = \gamma_{Rd} \cdot \frac{M_{R2\_L} + M_{R2\_R}}{M_{S2\_O} + M_{S2\_U}}$ In direction 2 $\delta_2 = \frac{\left|M_{S2\_R} + M_{S2\_L}\right|}{\left|M_{R2\_R}\right| + \left|M_{R2\_L}\right|}$

The seismic design moment on the columns in direction 1 and in direction 2 are calculated as follows:

$$M_{Sd1\_cd} = \min(\left|1 + (\alpha_{cd1} - 1) \cdot \delta_1\right| \cdot M_{Sd1},\ q \cdot M_{Sd1})$$
$$M_{Sd2\_cd} = \min(\left|1 + (\alpha_{cd2} - 1) \cdot \delta_2\right| \cdot M_{Sd2},\ q \cdot M_{Sd2})$$

Using equation (5.2) through (5.4)

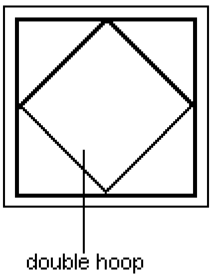
The capacity design shear forces at the columns assuming plastic hinges form at both ends, using equation (5.5) with $M_{Rc1,2}$, the end members moment and $\gamma_{Rd} = 1.2$.

$$V = \gamma_{Rd} \frac{M_{Rc1} + M_{Rc2}}{h}$$

Using equation (5.5)

Table 14-4: Beams reinforcement, design with emulated reinforced concrete

<table>
<tr><td colspan="6">Beams</td></tr>
<tr><td rowspan="2">Type/Member, Node</td><td colspan="2">Flexural reinforcement</td><td rowspan="2">Type/Member, Node</td><td colspan="2">Flexural reinforcement</td></tr>
<tr><td>A_{st}</td><td>A_{sb}</td><td>A_{st}</td><td>A_{sb}</td></tr>
<tr><td>Type1:M25-N5
Middel span +</td><td>4ϕ20+3ϕ16
3ϕ16</td><td>2ϕ20+4ϕ16
6ϕ16</td><td>Type3/M41-N26
+</td><td>2ϕ20+3ϕ16</td><td>3ϕ16
4ϕ16</td></tr>
<tr><td colspan="6">The ductility of all beams ends varies “between 9.3 to 11.9.” The shear reinforcement is: 4ϕ8 @112mm in the critical region and @150mm outside the critical region</td></tr>
<tr><td colspan="6">Columns</td></tr>
<tr><td colspan="6">Flexural reinforcement:
Columns fixation critical region; at the ground floor: 3ϕ16+2ϕ20 each side. Other columns critical region:5ϕ16 each side. All columns outside the critical region: 3ϕ16 each side.

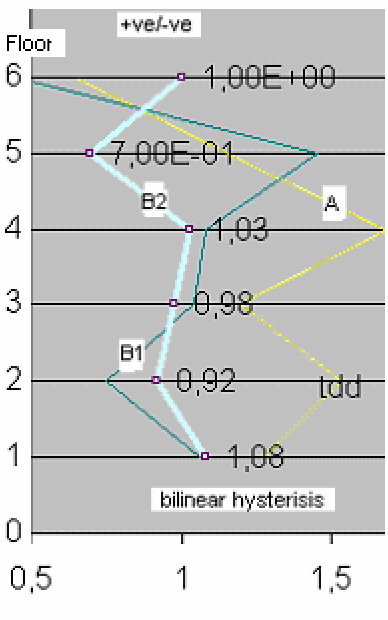
Shear reinforcement: double hoop; 2ϕ8 @110mm in the critical region and double hoop 2ϕ8@150mm outside the critical region.

</td></tr>
</table>

Optimise the design

The design of the concrete structure is optimized using the modified elastic stiffness, which is built based on the elastic frame model reinforcement. The stiffness at the column beam connections are introduced using the stiffness bending moment relations and after the consideration of the yield locations at the first floor (Figure 14-1). The resulting bending moment, at the column beam connections with modified stiffness model is about 80% of the obtained with the elastic model results, and the design reinforcement in the beams and the columns are identical. Further modification for the columns dimensions within this model leads to a reduction in the ground and first floor columns so that all the column dimensions in the building are 500mm×500mm.

The reinforcement of the elastic model is used for the estimation of the modified stiffness to build the modified elastic stiffness model. Then the reinforcement in the members is designed according to new model. The design concrete structures of both models are verified using the non-linear analysis program Ruaumoko. The design with the modified stiffness resulted in better distribution of the ductility among the members, and better ductility proportion of the top and the bottom reinforcement in each member end. The top and the flexural reinforcement share the seismic moment resistance Figure 14-7. The roof beams dimensions are modified to 600×300mm, where it have been recognized that the top beams do not yield enough. Both design models are verified for degradation, using Takeda degrading and plastic hinges at the columns bases. Both design concrete structure have ductility and strength resistance within the nonlinear analysis results. The advantage in the modified stiffness model can be seen clear in Figure 14-8.

	A		B1	
mem	+ve	-ve	+ve	-ve
25	4.5	4.1	5.3	5.3
26	5.7	4.2	5.8	5.3
27	5.8	4.2	5.9	5.4
28	6.3	3.4	4.2	5.7
29	5.1	3.6	4.3	5.9
30	5.1	3.7	4.7	5.9
31	5.0	4.2	5.1	4.8
32	5.1	4.2	5.1	4.9
33	5.1	4.3	5.0	5.0
34	6.5	3.7	4.3	3.9
35	6.4	3.8	4.4	4.1
36	6.4	3.9	4.5	4.2
37	6.2	5.7	4.9	3.5
38	6.8	5.9	5.4	3.7
39	6.9	5.8	5.5	3.7
40	2.1	2.6	1.2	2.6
41	2.1	2.7	1.1	3.1
42	1.7	4.0	1.3	3.2



A design with elastic model
B modified effective stiffness model
B1 substitute the design moments, after steel selection

Figure 14-7: Ductility and ductility ratio, with bilinear hysteresis

	A		B		
mem	+ve	-ve	+ve	-ve
25	7,76	7,16	6,94	6,67
26	9,70	7,00	7,17	7,02
27	9,90	6,88	7,15	7,83
28	10,30	5,21	4,18	5,14
29	8,75	5,45	5,00	5,73
30	8,52	5,81	4,91	5,54
31	8,07	6,58	5,11	5,44
32	7,98	6,64	5,12	5,61
33	8,00	6,70	5,10	5,68
34	5,45	4,18	2,24	3,35
35	5,54	4,39	2,31	3,63
36	5,93	4,32	1,89	3,74
37	1,66	5,85	0,11	2,04
38	3,60	6,16	1,82	2,39
39	2,89	6,44	1,68	2,48
40	0,00	1,76	0,00	2,45
41	0,00	1,89	0,00	2,69
42	0,00	3,49	0,00	2,60

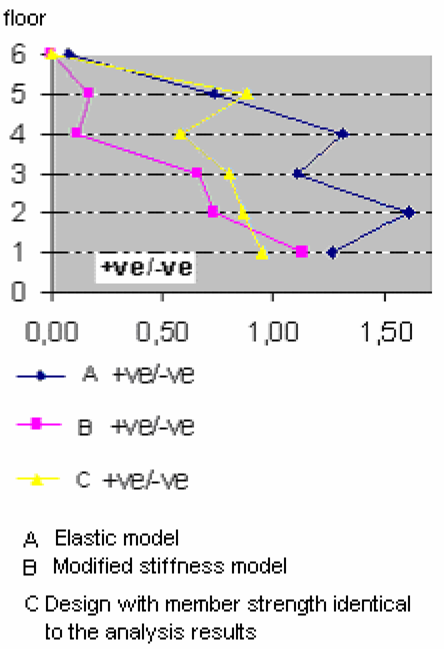


Figure 14-8: ductility and ductility ratio with degrading hysteresis.

Design with hybrid-fibre concrete HFC

The design with steel fibre reinforced concrete has not been yet applied in a wide range. This material performs well in seismic design. The main reason is that the confinement for the concrete provided by the steel fibre improves its toughness and ductility, which is one of the important concepts in the seismic design. Earthquake causes seismic joints subjected to multiple cracking in reverse cycles of loading. Conventional concrete looses its resistance completely after cracking. However, fibre concrete can sustain a portion of its resistance following cracking to resist more cycles. It was found that the fibre concrete joints had better energy dissipation, ductility, and stiffness, as well as spalling than plain concrete joints. Toughness or energy absorption capacity is the area under a load-deflection, moment-rotation. This is especially important for structures subjected to large energy inputs such as earthquakes.

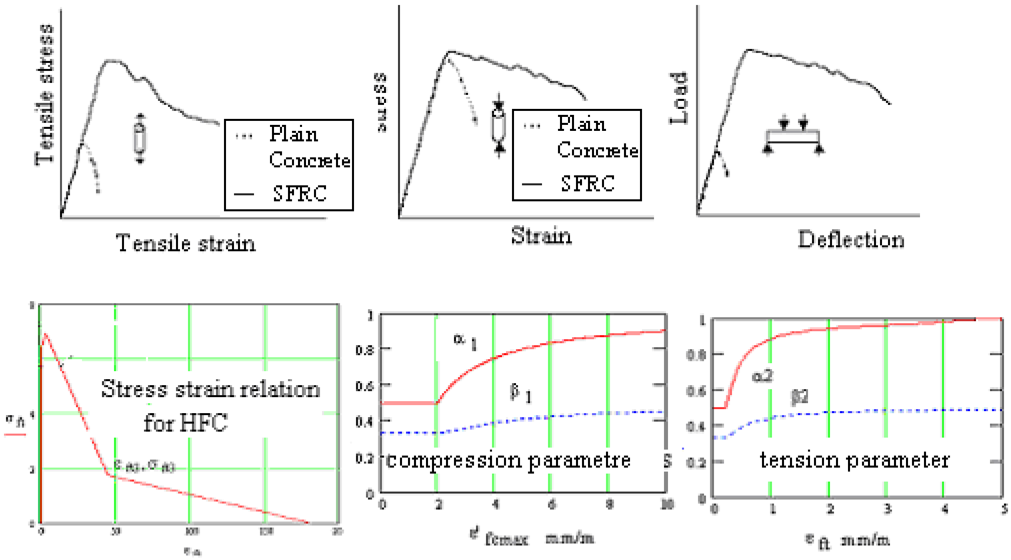


Figure 14-9: Properties of the steel fibre concrete SFRC, stress strain relations and the tension and compression parameters for FRC

Recommendations for the design with FRC

FRC has a high –dissipation capacity, which is of interest when dynamic loads are involved, moreover, because of its high tensile strength, cracking and structural integrity can control even in case of relatively strong impact. For known UHPFRC, the tensile strength f_t increases similar to that for conventional concretes. It means that the cracks at the columns face can be verified with the same level of confidence as that of the conventional concrete, using the similar over-strength factors .The steel reinforcement should yield before that the concrete cracked. In the reversal loading the drop of the stiffness and strength of the concrete due to reversal load should be limited in order to protect the column core from sudden shocks. It is important to insure that the resistance in the column beam intersection core, remains in any case above the transmitted loads, moments, shear, from the columns, and beams.

The main concept in the design with HFC is that we take benefit from its high strength then this strength may serve in the column beam connection that needs protecting it against the crack effect. And it is ductile with high shear resistance then we take benefit from this character, for the yield locations, that at a distance from the column that can be mobilizes and the member connection with the beam conform the shear requirement easily without heavy vertical reinforcement, Then the column beam connection is a connecting strip of reasonable length (Figure 14-5).

In the design of the HFC, the structural analysis results of the elastic stiffness model with 20% reduction of the beam end moments are used.

Hybrid fibre concrete (HFC) is used at the column beams connections (Figure 14-5). Its stress strain relations, the parameters α,β of the tension and compression concrete zone are illustrated in Figure 14-9 The design for the reinforcement is carried out to solving the cross section equilibrium equations (7.16) and (7.17).

$$n_{df} = \frac{N_d}{b \cdot h \cdot f'_c} = \psi_1 \cdot \frac{\sigma_{s1}}{f_s} + \alpha 1 \cdot k_x \cdot \left(1 - \frac{c}{h}\right) \cdot \frac{\sigma_{c1}}{f'_c} - \psi_2 \cdot \frac{\sigma_{s2}}{f_s} - \alpha_2 \cdot \left[1 - kx \cdot \left(1 - \frac{c}{h}\right)\right] \cdot \frac{\sigma_{ft}}{f'_c}$$

$$m_{df} = \frac{M_d}{b \cdot h^2 \cdot f'_c} = \psi_1 \cdot \frac{\sigma_{s1}}{f_s} \cdot \left(\frac{1}{2} - \frac{c}{h}\right) + \alpha 1 \cdot kx \cdot \left(1 - \frac{c}{h}\right) \cdot \frac{\sigma'_{c1}}{f'_c} \cdot \left[\frac{1}{2} - \beta 1 \cdot kx \cdot \left(1 - \frac{c}{h}\right)\right] \ldots$$

$$+ \psi_2 \cdot \frac{\sigma_{s2}}{f'_c} \cdot \left(\frac{1}{2} - \frac{c}{h}\right) + \alpha_2 \cdot \left[1 - kx \cdot \left(1 - \frac{c}{h}\right)\right] \cdot \frac{\sigma_{ft}}{f'_c} \cdot \left[\frac{1}{2} - \beta_2 \cdot kx \cdot \left(1 - \frac{c}{h}\right)\right]$$

Using equation (7.16) and (7.17).

1. Design of the Columns

The columns used in the HFC solution are the same that is used in the emulated monolithic R/C B55. The important different in the design is the forces exerted by the beams on the columns, due to the different strength and stiffness of the two materials. That has been verified through the application capacity design requirements. It have been recognized that the decision for the reinforcement at the beams ends should be closer to the needed flexure reinforcement. The increase in the reinforcement with the HFC leads to higher flexure strength as in the conventional concrete.

2. Design of the beams

The design of the flexure reinforcement has been carried out solving the equilibrium equations (7.16) and (7.17). At the columns beams connections extra flexural reinforcement are added to increase the moment resistance at the core and prevent cracks and splits failure at the core. The control of the yield locations carried out considering an over-strength factor of 1.2, for the cluster connections at N6 and N7.

The design for shear is carried out using equation (12.41). The ultimate shear strength is given by equation (12.41), the design results are given in Table 14-5 for the first floor and the roof .The reinforcement with HFC is considerably les than that with conventional R/C.

$V_u = V_{Rb} + V_a + V_f$, Using equation (12.41)

$V_{Rb} = \frac{1}{\gamma_E}\frac{0.21}{\gamma_b} k\sqrt{f_{cj}} \cdot b_0 d$ with

Using equation (12.41)

$k = 1 + \frac{3 \cdot \sigma_{tm}}{f_{tj}}$ for compression and $k = 1 - \frac{0.7\sigma_{tm}}{f_{tj}}$

Where $V_{Rd,}$ V_s, V_f are the participation in shear of the concrete, steel and fiber respectively. The design for shear includes the control of possible plastic hinge formation at the beams.

The design shear of the beams considered is carried out considering possible plastic hinge formation at the beam-ends as follows:

$$\text{Shear strength} \geq \gamma \cdot \frac{M_{p1} + M_{p2}}{l_p} + \frac{W}{2}$$

With $W = (DL + 0.15 \cdot LL) \cdot L_{beam}$ and $M_p = M_u$ using equation (5.6)

Table 14-5: Reinforcement with HFC

Type: Member, Node	Flexural reinforcement		Type: Member, Node	Flexural reinforcement	
	A_{st}	A_{sb}		A_{st}	A_{sb}
Type1: M25-N5 + (B55)	$3\phi16+2\phi8$(core) $3\phi16$	$2\phi16$ $6\phi16$	Type3: M40-N25 +	$3\phi16$ $4\phi12$	$3\phi12$ $4\phi16$
The available ductility varies between 9.5 to 11, and the demand ductility, within 5.			The shear reinforcement $4\phi6$ mm: @85mm in the critical region, and @150mm outside the critical region		

Designing with DDC and Post tension assemblage

In the design with DDC connection and Post tension assemblage yielding is considered enabled in the connection. The capacity of the connection decides the capacity of the beams, which remains in elastic. The elastic analysis of the structure is considered for the design with DDC and post tension. The displacement and the drift provided by the elastic model is provides a structural solution for the ductile and DDC connection considering that yielding occurs at the connection. The design with elastic model results in the required reinforcement and ductile members in the connection. And elastic modified stiffness model is used as redistribution for the resistance required in the connection that decides the required rotation enabled in the connection.

Design with DDC connection

This type of connection was motivated to improve the post-yield behaviour of the concrete ductile frame. They introduce a ductile road or fuse into the path away from the toe of the beam Figure 14-10). These connectors allow post-yield deformation where the members are jointed. The desired behaviour of the structure achieved through merging the steel technology with the basic objective of seismic loading limitations. The important principles for the adaptation of this connection type for the seismic action is the relocating the causative actions by; Relocate the yielding element to within the column where the confinement of the concrete protect it verse the lateral support action, allow the strain in the toe region of the beam to be verified, and transfer shear force by friction from steel to steel. In the design of the ductile connectors, the capacity design principles are applied trough considering the strength reduction factor and the overstrength factors within the design criteria's.

In the design with DDC connection is carried out for the elastic model and the modified stiffness model. Some of the design results are reported in Table 14-6.

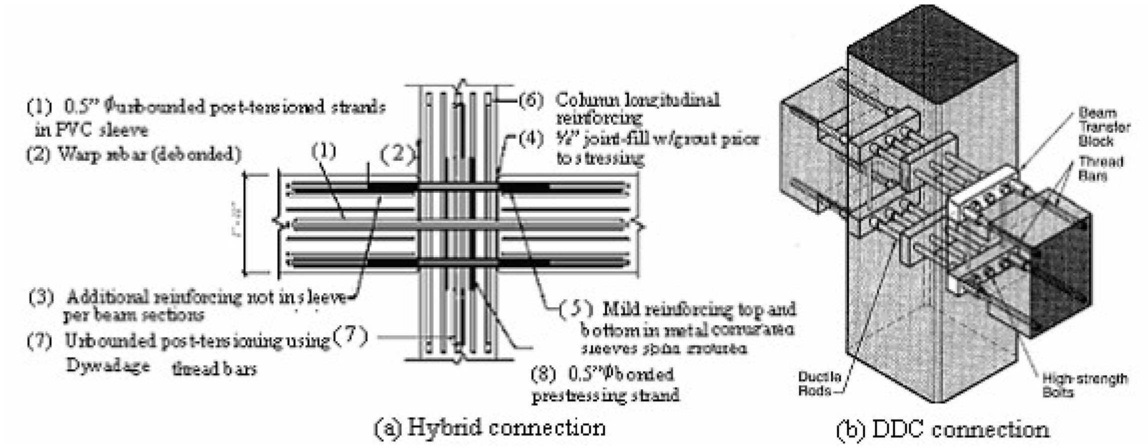


Figure 14-10: Hybrid post-tension and DDC connection

The nominal moment capacity of grope ductile, and the transfer load on the column are designed according to equation (12.10), (12.11)

$$M_n = Num \cdot T_y \cdot (d\text{-}d`),\ T_{bn} = \lambda_0 \frac{M_n}{(d\text{-}d`)}$$

Using equation (12.10),(12.11)

where $T_y = f_{ybolt} A_{bolt}$ and $d` = 2c$

The nominal shear capacity required of the connectors, is calculated as follows:

$$V_{cn} = V_{nE} + V_D + V_L$$

$$\text{With} \quad V_{nE} = 2 \cdot \frac{\lambda_0 M_n}{L_c} \quad \text{and} \quad V_D + V_L < 2 \cdot Num T_p f$$

Using equation (12.13)

The applied force on the ductile rod is unaffected by the level of preload. The members should be logically verified for the uncertainties associated with each of the considered transfer mechanisms. The factor ϕ / λ_0 should be used to the account for the ultimate moment as in equation (12.18).

$$M_u \le M_n \cdot \frac{\phi}{\lambda_0}$$

Using equation (12.18)

Summary of the steel area and the post-tension force on the connection rods at the first floor and the roof is reported in Table 14-6.

Table 14-6: Design details with bolted assemblages connection (DDC)

Type: member number*	A_{ns} [mm^2]	T_p [kN]	Type: member number*	A_{ns} [mm^2]	T_p [kN]
1/25	2880	130	3/38	3200	130
3/31	2890	130	3/ 40,41	3850	150
$A_{ns;}$ Ductile rod area of one set. $T_{p;}$ Bolt pretension					

The average reinforcements ratio is a reasonable value of 0.0133 per set. The pretension of the bolts ranges between 130 to 150 kN. Connections at the roof the may be built with emulated monolithic simple connection, while the shear and the bending moments is not significant.

Design with post-tension assemblage

The Hybrid beam system consists of concentric post-tensioned cables anchored at both ends of the frame The clamping force creates a friction force between the beams and columns, which transfers the shear demands. Mild reinforcing steel - the straining of which provides the necessary energy dissipation during a seismic event - is placed at the top and bottom of the beam through the joint and is grouted in place. These bars are also wrapped, rebounded in a region adjacent to the column to reduce inelastic strain and to force all post-yield rotation to occur at the beam-column interface. By limiting post-yield rotations to the joint, damage to the system is minimized. An additional benefit of the Hybrid Beam system is the restoring force provided by the elastic post tensioned. Flexure strength in the hybrid beam is provided by combination of unbounded post tensioning strand and bonded mild steel, if the strand stressed up to T_{nps} then the moment of M_{nps} is developed. Mild reinforcement grade 60, where placed in the top and bottom of the beam in tube that where subsequently grouted with high-strength grout (Figure 14-10). The design is carried out using equation (13.13) through (13.17). The design carried out for the elastic model and the modified stiffness model. Summary of the design is reported in
Table 14-7.

The nominal moment capacity M_n, the flexural moment provided by the mild steel, and the flexural strength are calculated as follows:

$$M_n = M_{ns} + M_{nsp}\,,\ M_{ns} = T_{ns} \cdot (d - d`)\,,$$

$$T_{nsp} = T_{nsp} \cdot f_{pse}$$ Using equation (13.13) through, (13.16)

The concrete compression depth and nominal moment are given in equation (13.15)

$$a = \frac{T_{nsp}}{0.85 f`_c b_b} \text{ and } M_{nsp} = T_{nsp} \cdot (\frac{h_b}{2} - \frac{a}{2})$$ Using equation (13.15)

The shear, and the associated column shear force F_{col} are calculated as follows:

$$V_{nb} = \frac{M_n}{L_b} \text{ and } V_c = F_{col} = V_{nb} \cdot (\frac{L_b}{h_x})$$ Using equation (13.17)

Table 14-7: Design detail with post tension assemblage connection.

Type: member number	A_{nsp} [mm^2]	A_{ns} [mm^2]	Type: member number*	A_{nsp} [mm^2]	A_{ns} [mm^2]
1/25	446	592	3/38	335	802
1/26	337	447	3/ 40,41	194	802
A_{nsp}; Area of post-tension steel., A_{ns}; The nominal steel area.					

Comparison

The construction with hybrid or simplified connections is possible. In the design information, knowledge covers the analysis and the function of the connection parts is required. In the design information, knowledge concerns the analysis and the function of the connection parts is required. The use of different connection types in a building enables to control the specific needed characters in a connection, and the characteristics of one connection type may be used in to fulfil a specific design requirement. The usage of the ductile rods in the hybrid connection may be reduced or eliminated if there is no need for high bending moment resistance or energy dissipation in the upper floors. In jointed connections system reinforcement strain is often verified by unbounded specific length of the reinforcement at the location that accommodate seismic action. In the comparison between the different connection concepts as the strength, availability and realization are used as follows:

The strength: The required strength in the connection is decided by the seismic performance of the structure system that required specific demand strength in the connection as shear, moment and ductility.

Availability and realization: The possible different solutions depend on the availability of the required materials and its adaptation within the connection specifications and details. The realization method may be considered on the building structural level. For simple building the emulated monolithic connections is the available solution, while in complicated system where the prefabrication is the absolute dominate aspect where the posttension used in the beams, then the construction with posttension assemblage is the available system.

The connection provides the building structure with different function reported in Table 14-9. Some of these required clarifying such as:

The building integrity: The connection design required high attention to assure the integrity of the connected elements, column, beams, and floor. The transfer of the force: the dead, live, and the seismic load. Enable the reversal action: The acting seismic force in should be studies in its effect in one direction and the reversal one. Where in places where the actions and the reversal actions are identical it is preferred using ductile DDC and post tension assemblage while it provides this requirement. Summary of the different connection functioning is reported in Table 14-9.

Seismic performance of the connections requires in addition to the designing according to the codes and the specification of the used material requires engineering judgment. Important connection seismic performance requirements to which the designer should take attention (

Table 14-10) are the followings:

Conventional and capacity design of the concrete structures may be used in seismic design in the different codes. The design with capacity design insures the strength required for the seismic actions and leads to the strengthening and weakening in the adequate locations of the different structural elements, for that the redundancy and the safety is assured within economical solutions. The seismic design requires the solution of the structural non-linear behaviour using non-linear programs, in the effective stiffness model the linear solution is used as a tool approaching and ease the non-linear solution. The capacity design method is applied in the design with elastic model and in the design with non-linear model. Seismic problems to be solved covers the conventional design and the structural behaviour in the post yield, using linear program and considering the structure mechanism and the non-linear behaviour in the post-yield or using non-linear analysis methods and programs. The modified stiffness model used the linear program and the non-linear behaviour of the beam-ends and the connections and provides explicit link to the non-linear analyses, and as a tool for the design concept building. An overview of the design process with the conventional and capacity design with elastic model and modified non-linear model is reported in Table 14-11.

Table 14-8: Comparison between different connection types.

Connection type	Availability	Resistance			Realization
		Shear	Moment	Ductility	
Emulated R/C	++++	+++	+++	++	++
Hybrid HFC	+	++++	++++	++++	+++
DDC	++	+++	++	++++	++++
Post tension	++	++++	++	+++	++++
The comparison grades (+) for the different types are judged depending on the relative characteristic difference in each item. The reinforcement in HFC is considerable less than that in R/C. The coast of the construction with HFC is reduced in related its high material prices, where short precast column strip beams are used.					

Table 14-9: Function and performance of the different types of connection.

Connection type	Function	Connection type	Function
R/C (1)	Integrity and continuity of the building due to the use of the same material and regulations. Possible mixing with (3) and (4), by adding extra posttension rods or ductile members to increase the shear resistance and its ductility. Yield locations are relocated from the column face.	Hybrid HFC (2)	Provides more ductility, high shear resistance. Yield locations are relocated from the column face. Possible mixing with (1), by using the R/C in the locations where no high shear resistance or ductility is required.
DDC (3)	Ductile members provide high ductility in the column faces. Extra moment, transfer the acting force inside the column, enables reversal action. Shear resistance due friction caused by the pretension of the ductile members	Post tension (4)	Add pretension force on the column increasing the shear resistance due to the friction. Dissipation of the energy is provided by the addition of the ductile rods, which provides flexural moment also.

Table 14-10: Seismic performance requirement

Factors		Emulated monolithic	HFC	DDC	Post-tension
Yield Location	Beam Location (1)	++	+++	++++	+++++
		0.5d Cracked beam	0.5d Cracked beam	(1) Yield in the connection	(1) Yield in the connection
	Location (2)	++++	+++++	++	++
Ductile members		Insert parts ++	Insert parts ++	Ductile pretension members ++++	Ductile rods members,
Bars slip control		+++++ ϕ, Detail, embedment length	+++ ϕ, Detail, embedment length	++ Detail	++ Detail
Shear resistance.		++++ Shear reinforcement	++ Shear reinforcement	++ Friction by pretension members	++ Friction by posttension tendons.
Core reinforcement		Diagonal shear	Diagonal shear	Diagonal shear. Stresses on the beam column face against inside ductile members action	Diagonal shear. Stresses in the core faces against local posttension tendons action.

Yield location: Column, column beam (1), beam (2). The columns are important element for control and especially in the design with the post tension assemblage.

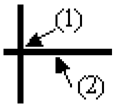


Table 14-11: Procedures in conventional and capacity design

Procedure		
Conventional design	Capacity design	
	Elastic model	Modified non-linear model
Preliminary design of the structure. Derivation of the sectional forces using a structural model and appropriate: -Gravity loads. -Earthquake forces. Design of the structural components -Dimensions. -Verifications. -Detailing.	Preliminary design of the structure Derivation of the structural forces using structural model and appropriate: -Gravity loads -Earthquake forces Design of the structure components -Choose a suitable mechanism -Determine critical sections after inelastic redistribution -Proportion and detail plastic hinge regions Proportion and detail parts of the structure intend to remain elastic considering the overstrength of plastic hinge region.	Preliminary design of the structure Derivation of the structural forces using structural model and appropriate: -Gravity loads -Earthquake forces Design of the structure components -Choose a suitable mechanism -Determine critical sections after suitable inelastic redistribution -Proportion and detail plastic hinge regions. Optimise the design of the structure components A: Emulated monolithic R/C and HFC. -Analyse with the modified stiffness. -Control the critical section after inelastic redistribution considering the connection properties. -Control proportion and detail plastic hinge regions. Proportion and detail parts of the structure intend to remain elastic considering the overstrength factor in the plastic hinge regions. B: Ductile DDC and Hybrid posttension assemblage HFC. -Analyse with the modified stiffness. -Control the ductile connection after inelastic redistribution considering the connection properties -Control proportion and detail plastic hinge regions Proportion and detail parts of the structure intend to remain elastic considering the overstrength factor in the plastic hinge regions.

Conclusion

The design with the four connection types in the building, and the construction with more than one connection type and simplified model of connections are possible. The construction with emulated monolithic R/C connections is a conventional solution, it required building with precast beam of column strip and middle strip units connected in cast in place R/C with the middle beams, and that is applicable for the HFC. The advantage of the HFC connection is that the shear resistance and the ductility in the connection are assured under control during the manufacturing of the precast concrete units. The coast of the construction with HFC is reduced in related its high material price, where short precast column strip beams are used. The construction with DDC and Post tension requires building with precast beam units connected in the columns with posttension and ductile rods, which requires a specific technical organization and specialist labours. The construction with DDC and Posttension assemblage can be organized and results in high quality and rapid construction.

In the design information, knowledge concerns the analysis and the function of the connection parts is required. The use of more than one connection type in a building provides the application of specific needed character in a specific location. It requires the determination of the needed stiffness, strength, ductility in the connections and the application of the suitable connection type that provides the better solution. The usage of the ductile rods in the hybrid connection may be reduced or eliminated when there is no high resisting shear force needed. In the building, for the first second and third floor construction with Dywidag or posttension assemblage is preferable where high need for ductility, while the upper floors may be constructed with emulated monolithic R/c or hybrid-fibre concrete. The required shear resistance can be obtained in its place using the friction apply the compression force using the posttension tendons and the ductile members with the posttension and DDC connection or relocating the joint location using precast unit or cast in place unit that resist the maximum shear at the column face and jointed with the interior beams in places where shear is low shear as in the construction with type (1) and type (2). The bending moment in the beam-ends is an important issue that influences the behaviour of the connection and can be designed and verified using redistribution takes into the account the postyield behaviour of the reinforced concrete. Construction with HFC required the accurate design for the bending moment reinforcement to reduce the exerted beams action on the columns, the core of the connection required to extra reinforcement to assure its resistance against cracking, and yielding of the beams away from the column face. The construction with HFC is efficient where the steel fibres provide the confinement.

The bending moment, and the resistance moment in the beam-ends is important factor that influences the behaviour of the connection. It can be performs through the design with reasonable redistribution of the response considering the postyield behaviour of the reinforced concrete, the variation of the stiffness and the strength as a function of seismic, and the intervention of the designer to apply stiffness and flexibility in location that lead to improve the response.

Bibliography

[1] George G. Penelis and Andreas J. Kappos. *Earthquake resistant concrete structure*
[2] Edmund Booth, Richard Fenwick. *Concrete structures in earthquake regions*
[3] Thomas Paulay, Hugo Bachmann, Konrad Moser. *Erdbebenbemessung von stahlbetonhochbauten*
[4] Ir.J.A.den Uijl. *Seismisch ontwerpen van betonconstructies*.
[5] Prof, Dip. -Ing.J.N.J.A.Vambersky / Prof.dr.ir.J.C.Walraven / Ing.J.P.Straman. *Designing and understanding precast concrete structures in building.*
[6] Kim S. Elliott. *Precast Concrete Structures*
[7] CEB, Bulletin D information No 220 and No 210. Volume1: General models, volume 2 *Frame Members. Behaviour and analyses of reinforced concrete structures under alternate actions including inelastic response*
[8] Hajime Umemura, Haruo Takizawa. *Dynamic response of reinforced concrete buildings.*
[9] Anil K. Chopra. *Dynamics of structures, theory and applications to earthquake engineering.*
[10] A Ghali, R Favre and M Elbadry. *Concrete Structures, Stresses and deformation.*
[11] Bob Park. *Some controversial aspects of the seismic design of reinforced concrete building structures.*
[12] International Federation for Structural Concrete (fib): *State-of-the-art report on the seismic design of precast concrete structures,* Task Group 7.3 of Commission 7, Lausanne, 2003.
[13] Standards New Zealand: *The design of concrete structures NZS 3101:1995, Wellington, 1995.*
[14] American Concrete Institute: *Building code requirements for structural concrete ACI 318-02*,Farmington Hills 2002.
[15] Architectural Institute of Japan: *Draft Japanese design guidelines for precast construction of equivalent monolithic reinforced concrete buildings*, Tokyo, 2000.
[16] Centre for Advanced Engineering: *Guidelines for the use of precast concrete in buildings.* University of Canterbury, Christchurch, New Zealand, 1st edition 1991, 2nd edition 1999, 144 pp.
[17] Priestly. MJN, Stiharan.S. Conley, J.R. and Pampanin, S: *Preliminary results and conclusions from the PRESS five story precast concrete test building*. Journal of the precast/ Prestressed concrete Institute. Vol 44.No . 6, pp. 42-67, 1999.
[18] Zahn, F.A., Park, R. and Priestley, MJN. *Design of reinforced concrete bridge columns for strength and ductility,* Research report 86-7, Department of Civil Engineering, University of Canterbury, March 1986, 330 pp.
[19] Park, R. *Some considerations in the seismic design of reinforced concrete interior beam-column joints of moment resisting frames,* Journal of the Structural Engineering Society of New Zealand, Vol. 15, No. 2, September 2002, pp. 53-64.
[20] Park, R. and Paulay, T. *Reinforced concrete structures*. John Wiley and Sons, New York, 1975,769 pp.

[21] Mander, J.B., Priestley, MJN and Park, R.: *Theoretical stress-strain model for confined concrete*. Journal of Structural Engineering of the American Society for Civil Engineers, Vol. 114, No. 8, August 1988, pp. 1804-1826.
[22] Watson, S., Zahn, F.A., and Park, R. *Confining reinforcement for concrete columns*. Journal of Structural Engineering of American Society of Civil Engineers, Vol. 120, No. 6, June 1994, pp.1798-1824.
[23] European Committee for Standardization. *Design provisions for earthquake resistance of structures EC 8* (Drafts), Brussels, 1994-2003.
[24] Booth, E.D., Kappos, A.J., Park, R., Moehle, J.P. and Hikone, S. *Seismic design of concrete frame structures:* A comparison of Eurocode 8 with other international practice, Proceedings of the 6th Conference of the Society for Earthquake and Civil Engineering Dynamics, Oxford,
United Kingdom, March 1998, pp. 481-492.
[25] D.E. Beskos, S.A. Anagnostopoulos. *Computer Analysis and Design of earthquake resistant structures, handbook*
[26]: Campilo, M., Gariel, J.C. Aki,K &Sanchez-Sesma, F.J. *Destructive strong ground motion in Mexico City. source, path and site effects during the great 1985 Michoacan earthquake,* Bull. Seism, Soc.
[27] S.(sri) Sriharan, M.J.Nigel Priestley, Frieder Seible and Akira Igarachi (12 WCEE 2000). A five-storey Precast concrete test building for seismic conditions and overview
[28] Athol. J Carr. *Ruaumoko, Inelastic Dynamic analyses.*
[29] Seismic construct program
[30] Anderson, J.C. and Bertero, V, V. *Seismic performance of instrumental six story steel building,* Report No. UCB/ EERC-91/111,Earthquake Engineering Research Center, University of California at Berkely, Berkely, California, 1991.
[31] T. Paulay, M.J.N. Priestley. S*eismic design of reinforced of reinforced concrete and masonry buildings.*
[32] Research school structural engineering. *Earthquake engineering Modeling Concept& Structural Design according to Euro code 2003*
[33] Farazad Naiem. *The seismic design handbook, second edition*
[34] Tomas Telford. *Frame under earthquake loading, state of arts*
[35] CEB, May 1904. *Behavior and analysis of reinforced concrete structures under alternated actions including inelastic response, volume 2.*
[36] Edmund & Partners(edition). *Concrete structures in earthquake regions design & analysis*
[37] De, Beskos, S. A. Anagnostopoulos (edition). *Computer analysis and design of earthquake resistance structures, Handbook*
[38] Robert E. Englekirk. *Seismic design of reinforced and precast concrete building*
[39] CEB-fib. *Displacement-based seismic design of reinforced concrete building*
[40] Johnston, C. *Fibre Reinforced Concrete*. Significance of Test and Properties of Concrete and Concrete –Marketing Material, ASTM STP 169C, 1994, pp. 547-561.
[41] Michael Gebman. *Application of steel fibre reinforced concrete in seismic beam column joints*. Thesis presented to faculty of San Diego State University.

Appendix

Refer to Ruaumoko manual for the nonlinear modeling and analyses.
Test result on the hybrid fiber concrete in the following page