STRENGTH AND DUCTILITY DEMAND
A CASE STUDY OF ETHIOPIAN DESIGN PRACTICE TO SEISMIC LOADS

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A THESIS SUBMITTED TO
THE SCHOOL OF GRADUATE STUDIES
OF ADDIS ABABA UNIVERSITY

In Partial Fulfillment of the Requirements
for the Degree of Master of Science in
Civil Engineering

SEPTEMBER 2009

ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES
FACULTY OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING

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NOVEMBER 2009

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Acknowledgement

Glory to the almighty lord, who gave me the patience and determination in overcoming any of the problems on my way to this goal.

My deepest gratitude then extends to Dr. Shifferaw Taye who has been providing me with all the valuable advice starting from the very beginning to the final stages of this work. His vital role in my study has kept me on the right track and helped me accomplish my objectives.

I would also like to gratefully acknowledge Eng. Sofonias Bezabih at GTZ International Services for providing me with all the necessary materials throughout my master's study and further giving me access to a licensed version of SAP 2000 Ver. 12.

Finally I would like to deeply express my gratitude to my father and mother for all the love and unyielding support they have been giving me for it made me achieve all the good including my master's study. And last but not least, I want to state that this work could have not been real with out the support I had from my future wife and my family and I want to express my utmost appreciation.

Abstract

This thesis is conducted to investigate the relationship between strength and ductility demand. The base shear and the story displacement ductility were used as parameters to establish the relationship and a five story and ten story reinforced concrete shear frames were used to this end. Eighteen shear frames with different combinations of ductility class and sub-soil class, according to the Ethiopian Building Code Standard EBCS 8, 1995, were considered. Each model was subjected to one linear static with EBCS 8, 1995 base shear distribution, two linear response spectrum EBCS 8, 1995 and EC 8, 2004, two non-linear static and one non-linear time-history analyses cases using the computer program SAP 2000 Ver.12.

A range of base shear values were applied on each model to study the post-yield displacement ductility demands imposed on it. From the analyses results, different curves showing the relationship between the reduced (%) base shear yield strength and ductility demand were produced. To this effect, the non-linear static or the pushover analysis method was used as a tool. Two types of non-linear properties were considered for all the models; the first, material non-linearity following the recommendations by Federal Emergency Management Agency FEMA 356 Pre-standard and Commentary for Seismic Rehabilitation of Buildings and Applied Technology Council ATC 40 Seismic Evaluation and Retrofit of Concrete Buildings and the second Geometric non-linearity by implementing the iterative P-Δ analysis procedure included in SAP 2000 Ver.12.

The influence of undermining the design base shear on ductility demand and the validity of permitted reductions by EBCS8, 1995 of design base shear through a behavior factor for each ductility class for the five and ten story shear frames was assessed by making use of theses curves. Finally the implementation of the relationship between these two important seismic design parameters in the current seismic design practice of Ethiopia was investigated by conducting a case study on an existing building located in Debre Birhan.

Key words: Base shear yield strength; Ductility demand; Pushover analysis

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1. Introduction

1.1. Background

Strength and ductility demand are two inter-related decisive parameters to assure a better performance of buildings when excited by dynamic loads such as, earthquake induce ground motions. A designer should always bear in mind two of the possible ways of addressing requirements to this end as stated by (Chopra, 2001). Considerations should be given by:

- Designing the structure strong enough so that it will remain with in the elastic limit when subjected to the design ground motion or
- Providing the necessary ductility demand corresponding to the level of available strength so that the damages, after an earthquake of the design level, are with in acceptable limits and repairable.

But the question is how strong should a building be designed? Or in other words, how ductile should it be?

A ductile building has more possibility to deform. As the building deforms, it releases more energy. Energy release is a mechanism by which a building gets reed off excess energy introduced due to ground motion.

The Ethiopian Building Code Standard EBCS 8, 1995 permits reduction of the design base shear by introducing a behavior factor 'γ' for two different ductility classes, ductility class medium (DCM) and ductility class high (DCH). With such reductions on the design base shear, comes a certain level of ductility demand imposed on the structure under consideration for which special detailing of structural members are presented in the code. But a question always remains; will the structure stay with in the range of assumed ductility demand levels when exposed to values of seismic actions of the design level?

Another important issue raised in this work is the influence of undermining the design base shear, which is expressed in EBCS 8, 1995 as a product of the design spectrum and the weight of the structure, on the ductility demand.

This thesis addresses these two main issues among other related ones and investigates the implementation of the relationship between these two parameters in the current seismic design practice of Ethiopia by conducting a case study on an existing building structure.

1.2. Objective

The objective of this thesis work is to:

- Investigate the relationship between strength and ductility demand.
- Study the validity of the reduction of the design base shear permitted by EBCS 8, 1995 for a specified ductility class.
- Summarize the provisions on strength and ductility demand in the building codes EBCS8, 1995 and EC8, 2004.
- Compare findings with the provisions of the building codes considered.
- Carry out a case study to evaluate how much the idea of strength and ductility demand is implemented in the seismic design practice of Ethiopia making use of an existing building in Debre Birhan.

2. Literature Review

2.1. Strength and Ductility

Strength and ductility have been the center of many researches carried out around earthquake engineering since the 1960's (Chopra, 2001). Some have tried to develop their relationship while others studied the effect that one has on the other. The term ductility is defined as the ratio of the total imposed displacements Δ at any instant to that at the onset of yielding Δ_y (Paulay and Priestley, 1992).

$$\mu = \Delta / \Delta_y$$

It is essential for a building structure to posses this behavior in order to have a better performance in resisting dynamic loads such as earthquake-induced ground motions. This property was considered by (Englekirk, 2003) as a system behavior enhancer. A structure is said to posses ductility when it is able to respond in an inelastic fashion to resist the design level earthquake without significant strength degradation (Paulay and Priestley, 1992). Another statement by (Rosenblueth, 1980) states ductile behavior as the ability to undergo large inelastic deformation with little decrease in strength.

The significance of this important behavior comes into picture especially when structures experience post-yield deformation levels. Buildings that posses adequate ductility compared to the seismic ductility demands suffer a lesser strength loss than those which lack this behavior. So an important consideration to this effect in the determination of the required seismic resistance is that the estimated maximum ductility demand during shaking does not exceed the ductility potential/allowable ductility (Paulay and Priestley, 1992). The available ductility of reinforced concrete sections, as stated by (Rosenblueth, 1980), mainly depends on the content of longitudinal tension and compression steel, the content of transverse steel for concrete confinement and restraint against buckling of bars, the concrete and steel strengths and level of axial load among many others. Building codes suggest detailing provisions for structural members to this end.

It is possible to design building structures to remain with in the elastic limit for an assumed level of design earthquake (Chopra, 2001). But the question regarding the economy and possibility of experiencing earthquake levels beyond the assumed design level remains at large. It is generally "uneconomic, often unnecessarily, and arguably undesirable to design structures to respond to design level earthquakes in the elastic range" (Paulay and Priestley, 1992). Another suggestion by (Englekirk, 2003) states that an elastic model for seismic design must be used with caution and understanding for the reason that levels of deformation will significantly exceed the idealized elastic limit of the system. So we can reasonably conclude that ductility, properly introduced into a structural system, will improve its behavior primarily by reducing experienced accelerations (Englekirk, 2003). However, in some structures like nuclear power plants, ductility which may lead to more deformation may not be preferred as this may compromise safety required to prevent escape of radioactive elements.

Currently the approach by many building codes is to apply a reduction factor to their suggested design base shear values to account for the ductile behavior of building structures. To mention, EBCS 8, 1995 and EC 8, 2004 suggest behavior factors as 'γ' and 'q' factors respectively while IBC 2006 gives provisions for inherent ductility and over strength through its 'R' factors. These factors can generally be considered as global response modifiers (Kunnath, 2005). The factors are usually suggested based on structural types and independent of the natural period of vibration and ground motion characteristics as demonstrated by (Chopra, 2001; Fierro and Perry, 2000). Some codes, as EC 8, 2004 'Sec. 5.2.2.2 (7)', recommend verification of these factors for specific cases by a more accurate method such as the non-linear static (pushover) global analysis. This issue is still being investigated by many researchers around the globe.

An effort was made by (Paulay and Priestley, 1992) to compare three building codes, U.S.A, Japan and New Zealand building codes, on their permitted base shear reduction factors and final values of base shear for different structural systems. The study showed that, in all the cases considered, the reduction factors dominate the final base shear coefficients adopted to determine the required seismic resistance.

A study to correlate design strength with ductility demand was carried out by (Fierro and Perry, 2000). It was conducted on families of single degree of freedom (SDOF) systems with different combinations of initial elastic period, design strength and damping ratios subjected to 30 different earthquake time histories recorded at soft soil, alluvium and rock sites. From the strength Vs ductility demand relationships produced, it was showed that the reductions on strength vary from down to 33% for 3sec period soft soil to 89% for 0.2 sec soft soil to attain a ductility demand of 4. It was concluded that the strength required to achieve a target ductility demand, besides characteristics of a building structure, depends upon the initial period, the soil type and the ground motion characteristic and that the approach by the current practice is oversimplified not to consider such factors.

Another study by (Chopra, 2001) investigated the relationship between strength and ductility for multi-degree of freedom (MDOF) systems. 2, 5, 10, 20, 30 and 40 story shear frames were considered for the study. In addition to the factors stated by (Fierro and Perry, 2000), the relation between the two parameters was observed to be influenced by the relative stiffness and yield strength of the stories. The influence of 'soft' and 'weak' stories was observed to confine yielding and in effect impose large ductility demands on such stories. After observing the story-wise distribution of ductility demands, correction factors that may be applied to amplify a design base shear considered for an allowable ductility were developed so that the imposed seismic ductility demands would not exceed the allowable ductility level. These factors enable the determination of reduction on design base shear with ductility demands kept with in the allowable range (Chopra, 2001). The study clearly showed the complexity of the relationship between the parameters in contrast to building codes' simplified approach.

Non-linear ductile behavior of complex systems usually stems as a consequence of local or concentrated ductile deformations that take place at those particular sections of a given structure where yielding strains are reached (Rosenblueth, 1980). It is clear that the reductions applied on the design base shear can not alone capture progressive distribution of non-linearities and the resulting redistribution of seismic demands (Kunnath, 2005). A designer should be careful in reducing the design base shear for a target ductility demand – at least need to be cautious to provide a proper detailing of structural elements considering the provisions suggested by the respective building codes. In general, in designing for a reduced or inelastic spectrum, a designer must be aware of the paramount importance of providing ductility capacity at least equal to that corresponding to the assumed force reduction factor.

The influence of strength, based on the amount of reinforcement provided, on displacement and ductility demand was studied by (Englekirk, 2003). A 16 ft tall structural wall and a 15 story shear frame were considered with different amounts of reinforcements. The study was concerned in evaluating the performance of systems provided with only the required amount of reinforcement to attain a target ductility demand against those provided with excessive reinforcement. It was revealed from the outputs of both the wall and shear frame systems that providing excessive reinforcement resulted in a lower ductility demands but increased displacement demands. And (Englekirk, 2003) further argued that an increase in strength, excess reinforcement, causes a more violent response leading to larger displacements and induced levels of strains.

The influence of the vertical component of earthquake on strength and ductility demands was investigated by (Como, De Stefano and Ramasco, 2000). A simplified three degree of freedom system accounting both material and geometrical non-linearities was used to this effect. It was shown that the inclusion of vertical component in the input ground motion does not lead to significant variations in the ductility demand with respect to those evaluated considering only the horizontal ground motion, because of the dominant effect of interaction phenomena between axial forces due to gravity loads only and lateral forces due to horizontal seismic components. It was further presented that the influence of P-Δ effects on inelastic response is generally not substantially amplified by the earthquake vertical component.

The current study was intended to investigate the relationship between strength and ductility demand of MDOF systems. In addition to this, an attempt was made to evaluate the permitted reductions on the design base shear associated with designs for ductility classes medium and high (DCM and DCH) stated in EBCS 8, 1995. The non-linear static (pushover) analysis procedure was adapted to this end.

2.2. Special Detailing Provisions for Local Ductility (EBCS 8, 1995 and EC 8, 2004)

As discussed in the previous section, the overall ductile behavior of a structure is highly influenced by the deformation capacity of its individual components. All the structural members should have adequate component ductility to meet overall system seismic ductility demands. Building codes recommend special detailing provisions for local (component) ductility. The provisions give special attention to potential locations of plastic hinges also known as critical regions. Tables 2.1a to 2.2c summarize these provisions recommended by EBCS 8, 1995 and EC 8, 2004 for beams and columns.

Table 2.1 Special detailing provisions for Beams

2.1a. Special Provisions For Beams of DCL **EBCS 8, 1995**

Local Ductility				
l_{cr}	ϕ_{hoops}	Spacing of hoops wiith in critical regions (min. of)	Placement of first hoop	Min no of S400 bars
h_w	EBCS 2			
	ρ_{max} = 75% of that allowed in EBCS 2			
Specific Measures				
Min Width, Min. Width : Height ratio, Min amt. of -ve. Bar along the length	EBCS 2			

2.1b. Special Provisions For Beams of DCM

Local Ductility

l_{cr}	ϕ_{hoops}	Spacing of hoops wiith in critical regions (min. of)				Placement of first hoop	Min no of ϕ14mm S400 bars	Code
$1.5h_w$	≥6mm	$h_w/4$	24 ϕ_{hoops}	200mm	$7\phi_{main}$	<50mm	2-top 2-bot	**EBCS 8, 1995**
h_w (from beam- column Joints) $2h_w$ (beam supporting discontinued vertical members)	≥6mm	$h_w/4$	24 ϕ_{hoops}	225mm	$8\phi_{main}$	<50mm	-	**EC 8, 2004**

	Code
$\rho_{max} = 0.65\ (f_{cd}/f_{yd})*(\rho'/\rho) + 0.0015$	**EBCS 8, 1995**
$\rho_{max} = \rho' + (0.0018/\mu_{\varphi}\varepsilon_{sy,d})*(f_{cd}/f_{yd})$ $\rho_{min} = 0.5*(f_{ctm}/f_{yk})$ $\rho' \geq 0.5*\rho$	**EC 8, 2004**

Specific Measures

		Code
Min Width =	200mm	**EBCS 8, 1995**
Min. Width:Height ratio of web =	0.25	
Min amt. of -ve. Bar along the length =	25% of max provided	
To enable efficient transfer of cyclic moments from primary seismic beams to columns, the eccentricity b/n the axes of the two members shall be <	$d_c/4$	**EC 8, 2004**
To take advantage of the favorable effect of column compression on the bond of horizontal bars passing through the joint, the width of the beam shall be ≤	$\min(d_c+h_w;2*d_c)$	

2.1c. Special Provisions For Beams of DCH

Local Ductility

l_{cr}	ϕ_{hoops}	Spacing of hoops wiith in critical regions (min. of)				Placement of first hoop	Min no of ϕ14mm S400 bars	Code
$2h_w$	≥6mm	$h_w/4$	24 ϕ_{hoops}	150mm	$5\phi_{main}$	50mm	2-top 2-bot	**EBCS 8, 1995**
$1.5h_w$ (from beam-column Joints) $2h_w$ (beam supporting discontinued vertical members)	≥6mm	$h_w/4$	24 ϕ_{hoops}	175mm	$6\phi_{main}$	<50mm	2-top 2-bot	**EC 8, 2004**

	Code
$\rho_{max} = 0.35\ (fcd/fyd)*(\rho'/\rho) + 0.0015$	**EBCS 8, 1995**
$\rho_{max} = \rho' + (0.0018/\mu_{\varphi}\varepsilon_{sy,d})*(f_{cd}/f_{yd})$ $\rho_{min} = 0.5*(f_{ctm}/f_{yk})$ $\rho' \geq 0.5*\rho$	**EC 8, 2004**

Specific Measures

		Code
Min Width =	200mm	**EBCS 8, 1995**
Min. Width:Height ratio of web =	0.02*distance between lateral supports (EBCS 2, 4.3.2)	
Min amt. of -ve. Bar along the length =	25% of max provided	
Min Width =	200mm	**EC 8, 2004**
Min amt. of -ve. Bar along the length =	25% of max provided	
To enable efficient transfer of cyclic moments from primary seismic beams to columns, the eccentricity b/n the axes of the two members shall be <	$d_c/4$	
To take advantage of the favorable effect of column compression on the bond of horizontal bars passing through the joint, the width of the beam shall be ≤	$\min(d_c+h_w;2*d_c)$	

Table 2.2 Special detailing provisions for Columns

2.2a. **Special Provisions For Columns of DCL**

Local Ductility						
l_{cr}	ϕ_{hoops}	Spacing of hoops wiith in critical regions (min. of)			Max dist.b/n main bars	Code
max(d_c;l_{cl}/6;450mm)	≥6mm	$b_o/2$	200mm	$9\phi_{main}$	250mm	**EBCS 8, 1995**
ρ_{max} = 0.04 ρ_{min} = 0.01 on symmetrical x-sections, ρ' = ρ						**EBCS 8, 1995**
Specific Measures						
Min x-sectional dimension = 200mm If Θ > 0.1 the x-sectional dimension of the col. Should be ≥ 1/10*larger dist. b/n the pt. of contra flexure and the ends of the columns						**EBCS 8, 1995**

2.2b. **Special Provisions For Columns of DCM**

Local Ductility						
l_{cr}	ϕ_{hoops}	Spacing of hoops wiith in critical regions (min. of)			Max dist.b/n main bars	Code
max(1.5d_c;l_{cl}/6;450mm)	≥0.35$\phi_{main,max}\sqrt{(f_{ydL}/f_{ydw})}$ ≥6mm	$b_o/3$	150mm	$7\phi_{main}$	200mm	**EBCS 8, 1995**
max(d_c;l_{cl}/6;450mm)	≥6mm	$b_o/2$	175mm	$8\phi_{main}$	200mm	**EC 8, 2004**
ρ_{max} = 0.04 ρ_{min} = 0.01 on symmetrical x-sections, ρ' = ρ						**EBCS 8, 1995**
ρ_{max} = 0.04 ρ_{min} = 0.01 on symmetrical x-sections, ρ' = ρ						**EC 8, 2004**
Specific Measures						
Min x-sectional dimension = 250mm If Θ > 0.1 the x-sectional dimension of the col. Should be ≥ 1/10*larger dist. b/n the pt. of contra flexure and the ends of the columns In the lower two stories the min. spacing of hoops recommended in critical regions shall be extended to an additional length of 0.5*l_{cr} At least one intermediate bar shall be provided between corner bars along each column sides in order to enhance the integrity of beam-column joints.						**EBCS 8, 1995**
If Θ > 0.1 the x-sectional dimension of the col. Should be ≥ 1/10*larger dist. b/n the pt. of contra flexure and the ends of the columns At least one intermediate bar shall be provided between corner bars along each column sides in order to enhance the integrity of beam-column joints.						**EC 8, 2004**

l_{cr} – critical region.

d_c – largest cross sectional dimension of the column.

h_w – height of the beam.

ϕ_{hoops} – diameter of hoops.

ϕ_{main} – diameter of longitudinal bars.

ρ_{max} – maximum amount of longitudinal reinforcement ratio.

ρ_{min} – minimum amount of longitudinal reinforcement ratio.

f_{ydL} - yield strength of longitudinal reinforcement.

f_{ydw} - yield strength of transverse reinforcement.

Θ - Inter-story drift sensitivity coefficient (EBCS 8, 1995 Sec.2.4.2.2(2))

μ_{φ} – required value of curvature ductility factor (EC8, 2004 Sec. 5.2.3.4(3))

$\varepsilon_{sy,d}$ – design value of tension steel strain at yield.

2.2c. Special Provisions For Columns of DCH

Local Ductility						
l_{cr}	ϕ_{hoops}	Spacing of hoops wiith in critical regions (min. of)			Max dist.b/n main bars	Code
max(1.5d_c;l_{cl}/5;600mm)	$\geq 0.40\phi_{main,max}\sqrt{(f_{ydL}/f_{ydw})}$ $\geq$6mm	$b_o/4$	100mm	5ϕ_{main}	150mm	**EBCS 8, 1995**
max(1.5d_c;l_{cl}/6;600mm)	$\geq 0.40\phi_{main,max}\sqrt{(f_{ydL}/f_{ydw})}$ $\geq$6mm	$b_o/3$	125mm	6ϕ_{main}	150mm	**EC 8, 2004**
ρ_{max} = 0.04 ρ_{min} = 0.01 on symmetrical x-sections, ρ' = ρ						**EBCS 8, 1995**
ρ_{max} = 0.04 ρ_{min} = 0.01 on symmetrical x-sections, ρ' = ρ						**EC 8, 2004**
Specific Measures						
Min x-sectional dimension = 300mm If Θ > 0.1 the x-sectional dimension of the col. Should be $\geq$ 1/8*larger dist. b/n the pt. of contra flexure and the ends of the columns In the lower two stories the min. spacing of hoops recommended in critical regions shall be extended to an additional length of 0.5*l_{cr} At least one intermediate bar shall be provided between corner bars along each column sides in order to enhance the integrity of beam-column joints.						**EBCS 8, 1995**
Min x-sectional dimension = 250mm If Θ > 0.1 the x-sectional dimension of the col. Should be $\geq$ 1/10*larger dist. b/n the pt. of contra flexure and the ends of the columns At least one intermediate bar shall be provided between corner bars along each column sides in order to enhance the integrity of beam-column joints. In the lower two stories the min. spacing of hoops recommended in critical regions shall be extended to an additional length of 0.5*l_{cr} The amount of longitudinal reinforcement provided at the base of the bottom story column (i.e. where the column is connected to the foundation) should be not less than that provided at the top.						**EC 8, 2004**

As presented in the summary above EBCS 8, 1995 gives recommendation for three classes of ductility ‘DCL’, ‘DCM’ and ‘DCH’ while EC 8, 2004 suggests the provisions only for two of the three classes omitting ‘DCL’. It further states that, the design for ‘DCL’ shall be done following EC 2 (EN 1992-1-1), 2004 with only some restriction on the reinforcing steel (see EC 8, 2004 ‘Sec.5.3.2(1)’) and restricts designing for ‘DCL’ only for low seismicity cases. But in the case of EBCS 8, 1995 there is no such restriction regarding seismic design for this class of ductility. According to the definition of low seismicity in EC 8, 2004 zone one, two and three of the Seismic Hazard Map of Ethiopia can be considered as low seismic regions leaving out zone four from this category.

2.3. Non-linear Static Procedures

The non-linear static procedure also referred to as the pushover analysis is a method in which static procedures are used to estimate dynamic seismic demands (Powel, 2007). This procedure takes into consideration material non-linearities by making use of non-linear load-deformation characteristics of individual components. It is also possible to include geometric non-linearities giving consideration to P-Δ effects. (Hasan, Xu and Grierson, 2002) expressed this method as a procedure where, for static-equivalent loading consisting of constant gravity loads and monotonically increasing lateral loads, the progressive stiffness/strength degradation of a building frame work is monitored at specified Performance levels[†] when conducted in the context of performance based seismic design. The monotonically increasing lateral load assumes an invariant height wise distribution until a target displacement or a mechanism is attained. The assumption behind this is that the response is controlled by a selected mode, usually the fundamental mode, and the mode shape remains unchanged after the structure yields (Chopra and Goel, 2001).

The method gains its prominence after it was introduced in the capacity spectrum method[††] (CSM) in 1978 by Freeman (qtd. in Kunnath, 2005). Due to its conceptual simplicity and computational attractiveness, the current civil engineering practice prefers to use this procedure over the more complex and rigorous non-linear response history analysis (Chopra and Goel, 2001). The method gives a good approximation for structures with small high mode effects (Kunnath, 2005). Though this may be a disadvantage of using this procedure, (Chopra and Goel, 2002) introduced a modal pushover analysis in which the contributions of higher modes were included in the pushover analysis by conducting the analysis using the inertia force distribution from higher modes and combining the seismic demands from the modes considered.

There are a number of static pushover analyses procedures recommended by different codes and pre-standards – ATC 40 Capacity Spectrum Method, FEMA 356 Coefficient method, FEMA 440 coefficient method and FEMA 440 Linearization method are the widely used ones. It is not the intention of this study to go into the details of these

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[†] Depending on the amount of damage, that can be observed or quantified, ATC 40 and FEMA 356 suggest three main structural performance levels – immediate occupancy (IO), life safety (LS) and collapse prevention (CP).

[††] The capacity spectrum method is a non-linear static procedure suggested by ATC 40 used to determine building performance point.

methods and this is left to the reader. In an attempt to compare these four methods, (Powel, 2006) explained the methods, studied their accuracy and suggested that FEMA 440 linearization method has a better accuracy for application in performance based seismic design.

The method is still being investigated around the globe by many researchers for a better accuracy and its application on low performance levels, life safety and collapse prevention, for a wide range of buildings and ground motions (Chopra and Goel, 2001).

2.4. Pushover Analysis – SAP2000

SAP2000 provides the following tools needed for pushover analysis (CSI, 2008):

- Material nonlinearity at discrete, user-defined hinges in Frame elements
- Nonlinear static analysis procedures specially designed to handle the sharp drop-off in load carrying capacity typical of frame hinges used in pushover analysis
- Nonlinear static analysis procedures that allow displacement control, so that unstable structures can be pushed to desired displacement targets
- Display capabilities in the graphical user interface to generate and plot pushover curves[†], including capacity and demand curves[††] in spectral ordinates
- Capabilities in the graphical user interface to plot and out put the state of every hinge at each step in the pushover analysis

In addition to these, it is also possible to include non-linear link/support behavior, geometric nonlinearity and staged construction. Out of these, material and geometric non-linearities were implemented in this study.

The general steps required to conduct pushover analysis using SAP 2000, as presented in (CSI, 2008), are as summarized hereunder.

a. Create a model just like for any other analysis.
b. Define frame hinge properties and assign them to the frame elements.
c. Define any Load Patterns and static and dynamic Load Cases that may be needed for concrete design of the frame elements, particularly if default hinges are used.
d. Run the Load Cases needed for design.

[†] Pushover curve also known as capacity curve is a plot of the base shear against displacement of a control point, usually the center of mass at the roof level.

[††] Demand curve is a representation of the response spectrum in spectral acceleration Vs spectral displacement format

e. If any concrete hinge properties are based on default values to be computed by the program, perform concrete design so that reinforcing steel is determined.
f. Define the Load Patterns that are needed for use in the pushover analysis, including:
    - Gravity loads and other loads that may be acting on the structure before the lateral seismic loads are applied. These loads patterns may have already been defined above for design.
    - Lateral loads that will be used to push the structure. If Acceleration Loads or modal loads are going to be used, no new Load Patterns need to be defined, although modal loads require definition of a Modal Load Case.
g. Define the nonlinear static Load Cases to be used for pushover analysis, including:
    - A sequence of one or more cases that start from zero and apply gravity and other fixed loads using load control. These cases can include staged construction and geometric nonlinearity.
    - One or more pushover cases that start from this sequence and apply lateral pushover loads. These loads should be applied under displacement control.The monitored displacement is usually at the top of the structure and will be used to plot the pushover curve.
h. Run the pushover Load Cases.
i. Review the pushover results: Plot the pushover curve, the deflected shape showing the hinge states, force and moment plots, and print or display any other results needed.
j. Revise the model as necessary and repeat.

3. Methods and Procedures

3.1. Assumptions

The following assumptions were made to formulate the research problem and develop the analyses models

a. Member failures due to inadequate anchorage and splicing were not considered as potential failures.

b. Rigid beam-column joints were assumed for modeling and the capacity of adjacent components was not limited by the joint strength.

c. For all the analyses models, the same building importance factor (I=1) and bed rock acceleration ratio (α_o = 0.1) were assumed to determine the respective EBCS8, 1995 base shear values.

d. For the dynamic analyses cases, both the design spectrums of EBCS8, 1995 and EC8, 2004 were scaled to fit a pick ground acceleration of 0.1g.

3.2. Analysis Parameters

The parameters used to develop the analyses models are

a. Ductility class
    i. "DCL" – Ductility class low.
    ii. "DCM" – Ductility class medium.
    iii. "DCH" – Ductility class high.

There is no numerical limit for these ductility classes in EBCS 8, 1995. FEMA 356 and ATC 40 suggest the following numerical values for component ductility demand classifications and these values are adopted for system ductility demand classification throughout this study.

Ductility Class	Ductility Demand
DCL	<2
DCM	2 to 4
DCH	>4

b. Sub-soil classification
    i. Sub-soil class A.
    ii. Sub-soil class B.
    iii. Sub-soil class C.

Each model is a different combination of theses two parameters resulting in eighteen distinct models consisting five and ten story shear frames.

3.3. Analysis Variables

The base shear and the story ductility demand values from the analyses cases were considered as the variables for the study. The base shear was taken as the independent variable for all the models and was applied incrementally by making use of the pushover analysis method. The dependent variable, i.e. the story ductility demand was considered to be the displacement ductility at each of the story levels of the five story and ten story shear frames imposed by the applied base shear value.

3.4. Model Description

The models considered for all the cases are a five story and a ten story reinforced concrete shear frames with a geometrical arrangement shown in Fig. 3.1. The frames are assumed fixed at their base and at each story level, a diaphragm constraint was introduced to enable them to be considered as a shear frame. The beam and column cross sections adopted for the shear frames are presented in Table 3.1. The same cross sections were used on all the story levels to avoid any stiffness variation.

Table 3.1 Model Characteristics

a. 5 Story Model

Beam x-sec (mm)	250*400h	Story	Weight(kN)
Column x-sec (mm)	300*400	1	705
Concrete Grade	C-25	2 - 4	680
Steel Grade	S-400		
Fundamental Period of Vibration	1.3 Sec.		
Damping Ratio	5%	5	585

b. 10 Story Model

Beam x-sec (mm)	250*400h	Story	Weight(kN)
Column x-sec (mm)	500*500	1	705
Concrete Grade	C-25	2 - 9	680
Steel Grade	S-400		
Fundamental Period of Vibration	2.5 Sec.		
Damping Ratio	5%	10	585

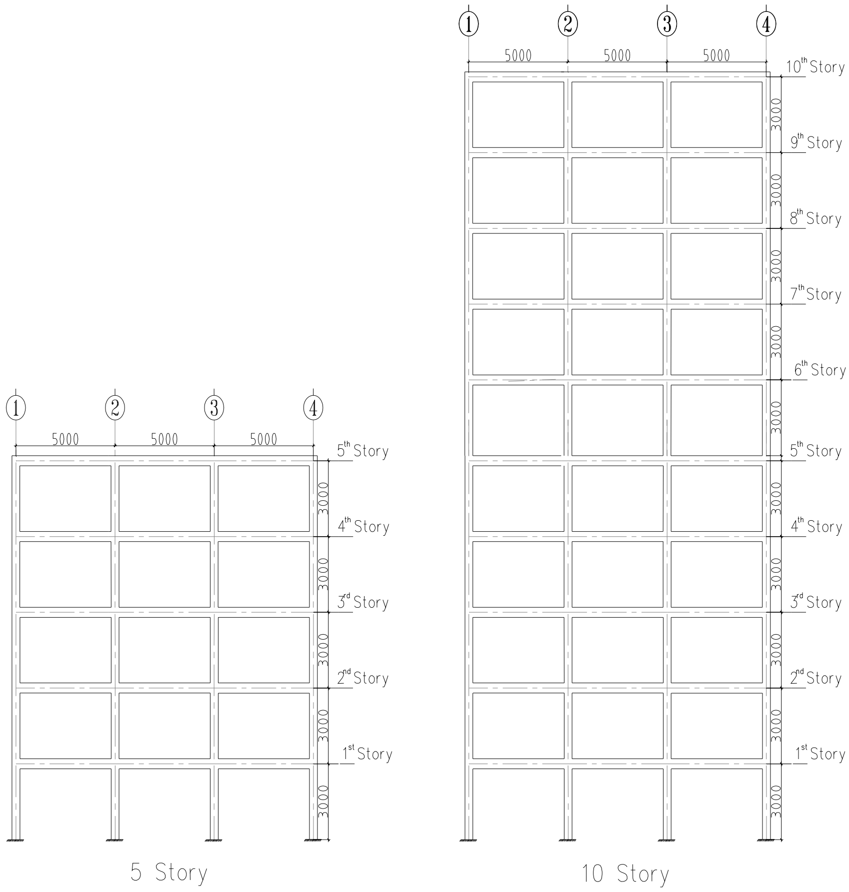


Fig. 3.1 Typical analyses models

The geometrical arrangement of the models with the properties stated in Table 3.1 resulted in EBCS8, 1995 base shear distributions for the eighteen models under considerations as presented in Table 3.2.

Table 3.2 EBCS 8, 1995 base shear distributions for all models

a. 5 Story Model

Model	Ductility Class	Sub-soil Class	Design Base Shear (kN)	Force at Story Level (kN)				
				1	2	3	4	5
1	DCL	A	232.06	16.11	31.07	46.61	62.15	76.12
2	DCL	B	278.47	19.33	37.29	55.93	74.58	91.34
3	DCL	C	333.00	23.11	44.59	66.89	89.18	109.23
4	DCM	A	174.04	12.08	23.31	34.96	46.61	57.09
5	DCM	B	208.85	14.50	27.97	41.95	55.93	68.51
6	DCM	C	249.75	17.34	33.44	50.16	66.89	81.92
7	DCH	A	116.03	8.05	15.54	23.31	31.07	38.06
8	DCH	B	139.24	9.66	18.64	27.97	37.29	45.67
9	DCH	C	174.04	12.08	23.31	34.96	46.61	57.09

-

b.10 Story Model

Model	Ductility Class	Sub-soil Class	Design Base Shear (kN)	Force at Story Level (kN)									
				1	2	3	4	5	6	7	8	9	10
1	DCL	A	331.63	5.98	11.53	17.30	23.07	28.83	34.60	40.37	46.13	51.90	71.93
2	DCL	B	397.96	7.17	13.84	20.76	27.68	34.60	41.52	48.44	55.36	62.28	86.31
3	DCL	C	497.45	8.97	17.30	25.95	34.60	43.25	51.90	60.55	69.20	77.85	107.89
4	DCM	A	248.72	4.48	8.65	12.97	17.30	21.62	25.95	30.27	34.60	38.92	53.95
5	DCM	B	298.47	5.38	10.38	15.57	20.76	25.95	31.14	36.33	41.52	46.71	64.73
6	DCM	C	373.08	6.73	12.97	19.46	25.95	32.44	38.92	45.41	51.90	58.39	80.92
7	DCH	A	165.82	2.99	5.77	8.65	11.53	14.42	17.30	20.18	23.07	25.95	35.96
8	DCH	B	198.98	3.59	6.92	10.38	13.84	17.30	20.76	24.22	27.68	31.14	43.16
9	DCH	C	248.72	4.48	8.65	12.97	17.30	21.62	25.95	30.27	34.60	38.92	53.95

3.4.1. Load Cases

Each analysis model was subjected to a total of ten different types of load cases which include linear, non-linear, static and dynamic types of analyses. Table 3.3 gives a summary of the load cases considered for each model.

Table 3.3 Load cases considered for all the models

Load Case Name	Type	Geometrical Nonlinearity	Initial Condition
MODAL	LinModal	-	Unstressed
DEAD	LinStatic	-	Unstressed
Live	LinStatic	-	Unstressed
EQX	LinStatic	-	Unstressed
RSEC8A	LinRespSpec	-	Unstressed
RSEBCS8A	LinRespSpec	-	Unstressed
Push_Gravity	NonStatic	-	Unstressed
Push_Gravity-P Delta	NonStatic	P-∆ plus Large Displacements	Unstressed
Push-Mat_Nonlin	NonStatic	-	Push_Gravity
Push-P-Delta	NonStatic	P-∆ plus Large Displacements	Push_Gravity-P Delta
El Centro 1940 N-S	NonDirHist	-	Push_Gravity

For the non-linear time history analysis case, the north-south component of the ground motion recorded at a site in El-Centro, California during the Imperial Valley, California earthquake of May 18, 1940 with peak ground acceleration equal to 0.319g was used. This ground motion record, annexed with this document, was adopted from (Chopra, 2001) and contains 1559 data points at equal time spacing of 0.02 sec.

3.4.2. Non-linear properties

Two types of non-linear properties were considered for all the models. The first one, material non-linearity, was modeled by introducing plastic hinges to the models. And the second, geometric non-linearity was taken into account by considering second order effects by the iterative P-Δ method using SAP 2000 ver.12.

3.4.3. Non-linear hinges

SAP 2000 presents different types of non-linear hinges in order to be able to account for the possible sources of non-linearity – uncoupled, moment, torsion, axial force and shear hinges and also coupled P-M_2-M_3 hinges which yield based on the interaction of axial force and uni-axial or bi-axial bending moments at the potential hinge locations as stated in (CSI, 2008).

“Each hinge represents concentrated post yield behavior in one or more degrees of freedom.” (CSI, 2008). (CSI, 2008) states the hinge properties as named sets of rigid-plastic properties that can be assigned to one or more frame elements. For each degree of freedom, a force-displacement (moment-rotation) curve that gives the yield value and the plastic deformation following yield has to be defined. Two types of non-linear hinges were defined for all the models. For the beam elements, M_3 hinges generated using, FEMA 356 ‘Table 6.7(1)’, and for column elements P-M_3 hinges generated using, FEMA 356 ‘Table 6.8(1)’ were defined.

These two types of non-linear hinges were assigned to the respective frame elements at relative lengths of 0 and 1 stated otherwise at the two ends of each frame element. Fig 3.2 shows the moment-rotation relation used to define the non-linear hinges and Tables 3.4 and 3.5 provide the recommended numerical values by FEMA 356. Fig 3.2 , Table 3.4 and Table 3.5 are all adopted from FEMA 356.

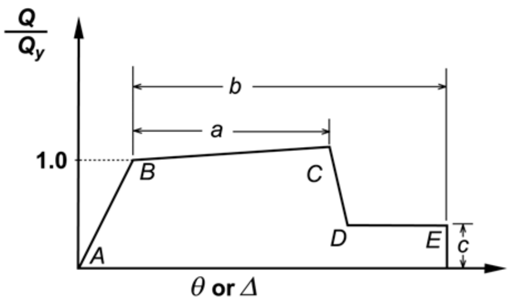


Fig 3.2 FEMA 356 generalized Force-Deformation relations for concrete elements or components

Q/Qy – Corresponds to the states where yielding occurs if the response is associated with flexure or tension and sapling of concrete or attainment of design shear strength if associated with compression or shear.

Table 3.4 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures— FEMA 356

Reinforced Concrete Beams

			Modeling Parameters[2]			Acceptance Criteria[2]				
						Plastic Rotation Angle, radians				
						Performance Level				
			Plastic Rotation Angle, radians		Residual Strength Ratio		Component Type			
							Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Beams controlled by flexure										
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[1]	$\frac{V}{b_w d (f_c')^{0.5}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.01	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0015	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0015	0.005	0.005	0.005	0.01

1. “C” and “NC” are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ d/3, and if, for components of moderate and high ductility demand, the strength provided by the hoops (Vs) is at least three-fourths of the design shear. Otherwise, the component is considered non-conforming.
2. Linear interpolation between values listed in the Table shall be permitted.

Table 3.5 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures— FEMA 356

Reinforced Concrete Columns

			Modeling Parameters[2]			Acceptance Criteria[2]				
						Plastic Rotation Angle, radians				
						Performance Level				
			Plastic Rotation Angle, radians		Residual Strength Ratio		Component Type			
							Primary		Secondary	
Conditions			a	b	c	IO	LS	CP	LS	CP
i. Columns controlled by flexure										
$\frac{P}{A_g f_c'}$	Trans. Reinf.[1]	$\frac{V}{b_w d (f_c')^{0.5}}$								
≤ 0.1	C	≤ 1.2	0.02	0.03	0.2	0.005	0.015	0.020	0.02	0.03
≤ 0.1	C	≥1.5	0.016	0.024	0.2	0.005	0.012	0.016	0.016	0.024
≥ 0.4	C	≤ 1.2	0.015	0.025	0.2	0.003	0.012	0.012	0.018	0.025
≥ 0.4	C	≥1.5	0.012	0.02	0.2	0.003	0.010	0.006	0.013	0.02
≤ 0.1	NC	≤ 1.2	0.006	0.015	0.2	0.005	0.001	0.005	0.01	0.015
≤ 0.1	NC	≥1.5	0.005	0.012	0.2	0.005	0.004	0.003	0.008	0.012
≥ 0.4	NC	≤ 1.2	0.003	0.01	0.2	0.002	0.002	0.002	0.006	0.01
≥ 0.4	NC	≥1.5	0.002	0.008	0.2	0.002	0.002	0.000	0.005	0.008

1. “C” and “NC” are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ d/3, and if, for components of moderate and high ductility demand, the strength provided by the hoops (Vs) is at least three-fourths of the design shear. Otherwise, the component is considered non-conforming.
2. Linear interpolation between values listed in the Table shall be permitted.

3.5. Procedures

It is possible to follow a variety of procedures to study the relationship between strength and ductility demand. But whatever procedure is followed, the outcome will mainly be concerned with the level of ductility demand imposed by a predefined set of lateral load arrangements, of course not neglecting the influence of gravity loads.

For this study the two parameters were first clearly defined. Accordingly, the base shear yield strength was first defined as the design base shear stated by EBCS8, 1995 and corresponding to this, the story shear yield strength values were defined as the resulting story shear values at each story level when subjected to the code force distribution. The load case 'EQX' corresponds to this definition. Consider for example the first model which is prepared for sub-soil class 'A' and ductility class 'DCL', the code suggests a design base shear value of 232.06kN and the resulting code force distribution and in effect the story shear yield strength values are as shown in Fig. 3.3.

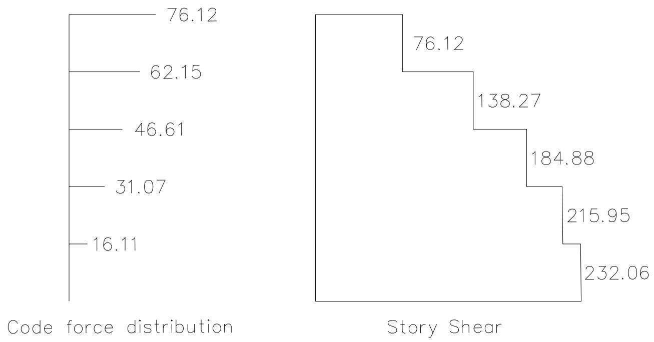


Fig 3.3 Code force and story shear yield strength values (Model 1)

The second parameter, ductility demand, is the ratio of the inter-story drifts at a story shear level to the value at the story shear yield strength level or Δ / Δ_{yield}.

The preceding definitions of story shear yield strength and ductility demand were adapted from (Chopra, 2001) with some major modifications. Once these definitions were clearly stated, the procedures followed to establish the relationships can be summarized as follows.

3.5.1. Summary

a. Develop an analysis model for a specified ductility and sub-soil class.
b. Determine the design base shear and its distribution at each story level according to EBCS8, 1995.
c. Design the frame for load combinations recommended in EBCS 8, 1995 considering both gravity and lateral loads.

d. Analyze the structure for the load cases RSEBCS8 and RSEC8 which represent response spectrum EBCS8 and EC8 respectively.
e. Run the pushover load cases
    i. 'Push_Gravity' – applies the gravity loads, dead and live, incrementally up to the specified levels in the load patterns 'DEAD' and 'live'.
    ii. 'Push_Gravity-P Delta' - This load case is the same as 'Push_Gravity' with the difference that in addition to the material non-linear properties, it also considers geometrical non-linearity.
    iii. 'Push-Mat_Nonlin' – this load case continues from the stressed state at the end of the non-linear case 'Push_Gravity'. It applies the lateral loads with the same pattern as 'EQX' incrementally until a specified target displacement level is achieved at a pre-specified joint or a mechanism is developed or due to other computational reasons when performing the non-linear analysis whichever comes first. It only considers material non-linear properties.
    iv. 'Push-P-Delta' – This load case is the same as 'Push-Mat_Nonlin' with the difference that in addition to the material non-linear properties, it also considers geometrical non-linearity and it continues from the stressed state at the end of the non-linear case 'Push_Gravity-P Delta'.
f. Run the non-linear time history analysis.
g. Compute the ductility demands corresponding to the different base shear values of all the analysis cases.
h. Present the relationship between %base shear yield strength and corresponding ductility demand levels in graphical and tabular forms.

3.6. Case Study

The structure considered for this case study is a G+2 building located in Debre Birhan area which is categorized as zone '4' on the seismic hazard map of Ethiopia. The structural design of this building was carried out for a ductility class 'DCM' and sub-soil class 'A'. Fig. 3.4a through Fig 3.4e show the architectural plans, elevation and section of the building. The structure of this building is a frame system with pre-cast flooring. This frame system also acts as the lateral load resisting system for the building. The maximum beam span is 5.91m and its floor to floor height is 3.09m. For the purpose of this study, the building is considered as a four story building, taking the ground level as the first story and the Roof level as the fourth one.

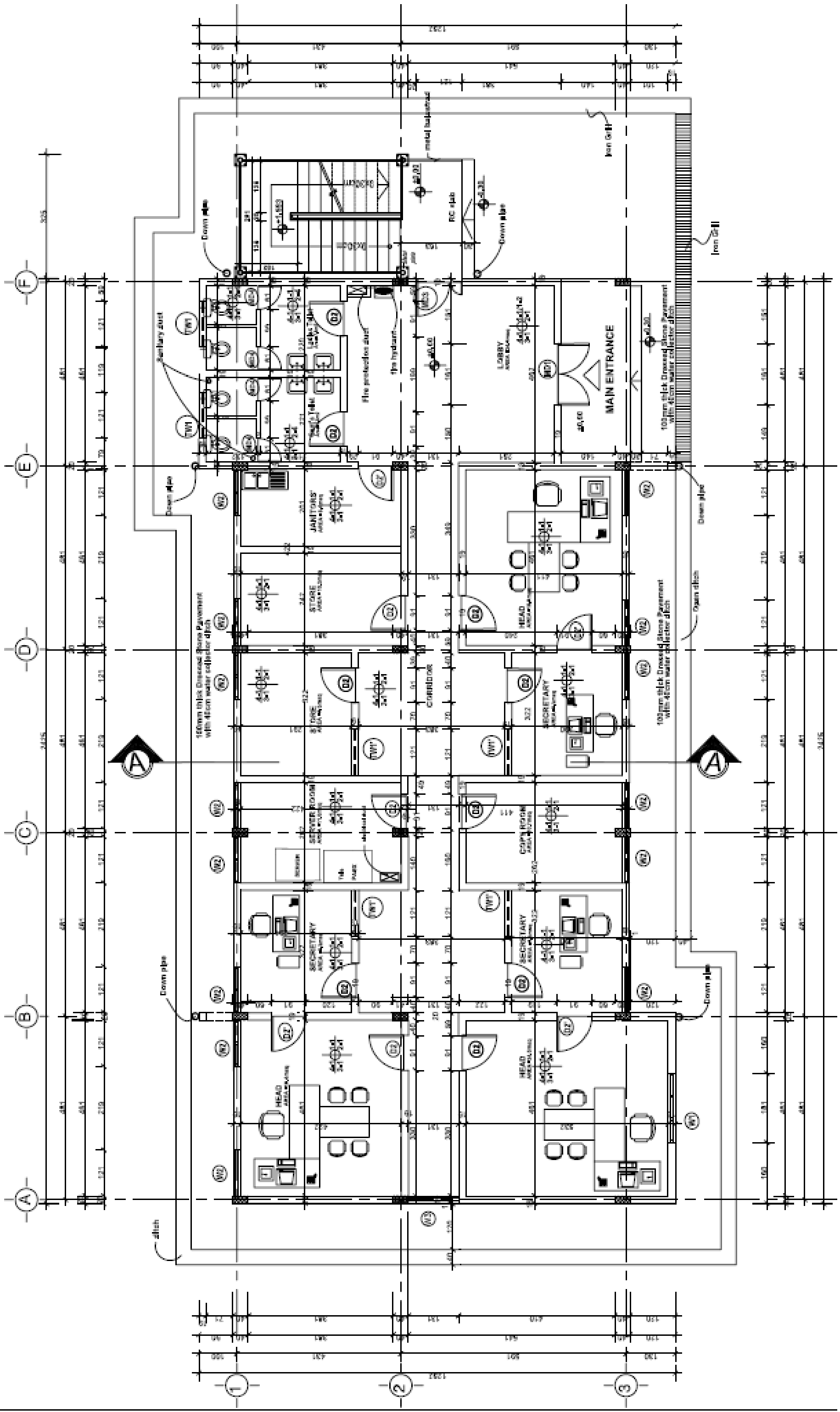


Fig. 3.4a Ground Floor plan

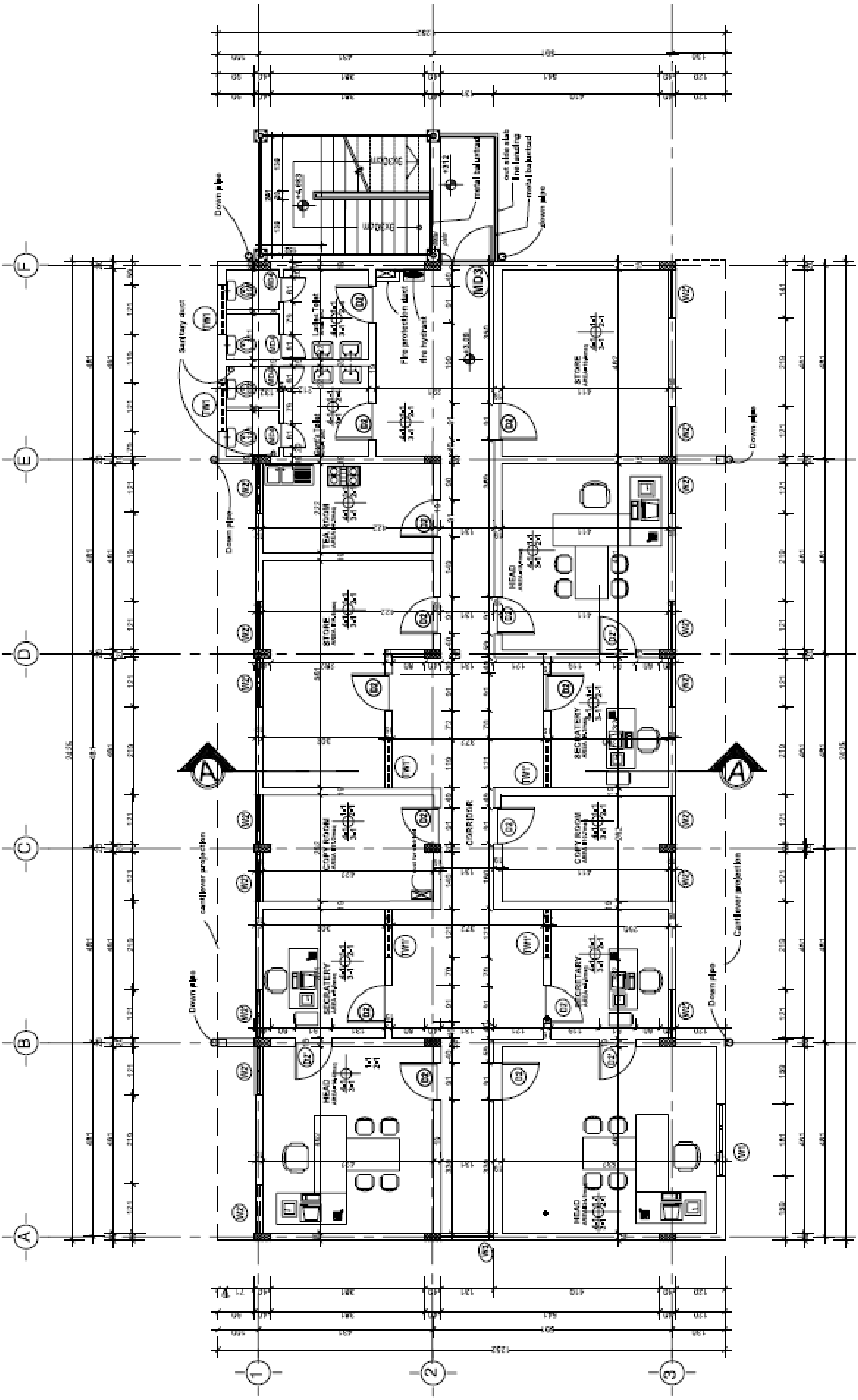


Fig. 3.4b First Floor Plan

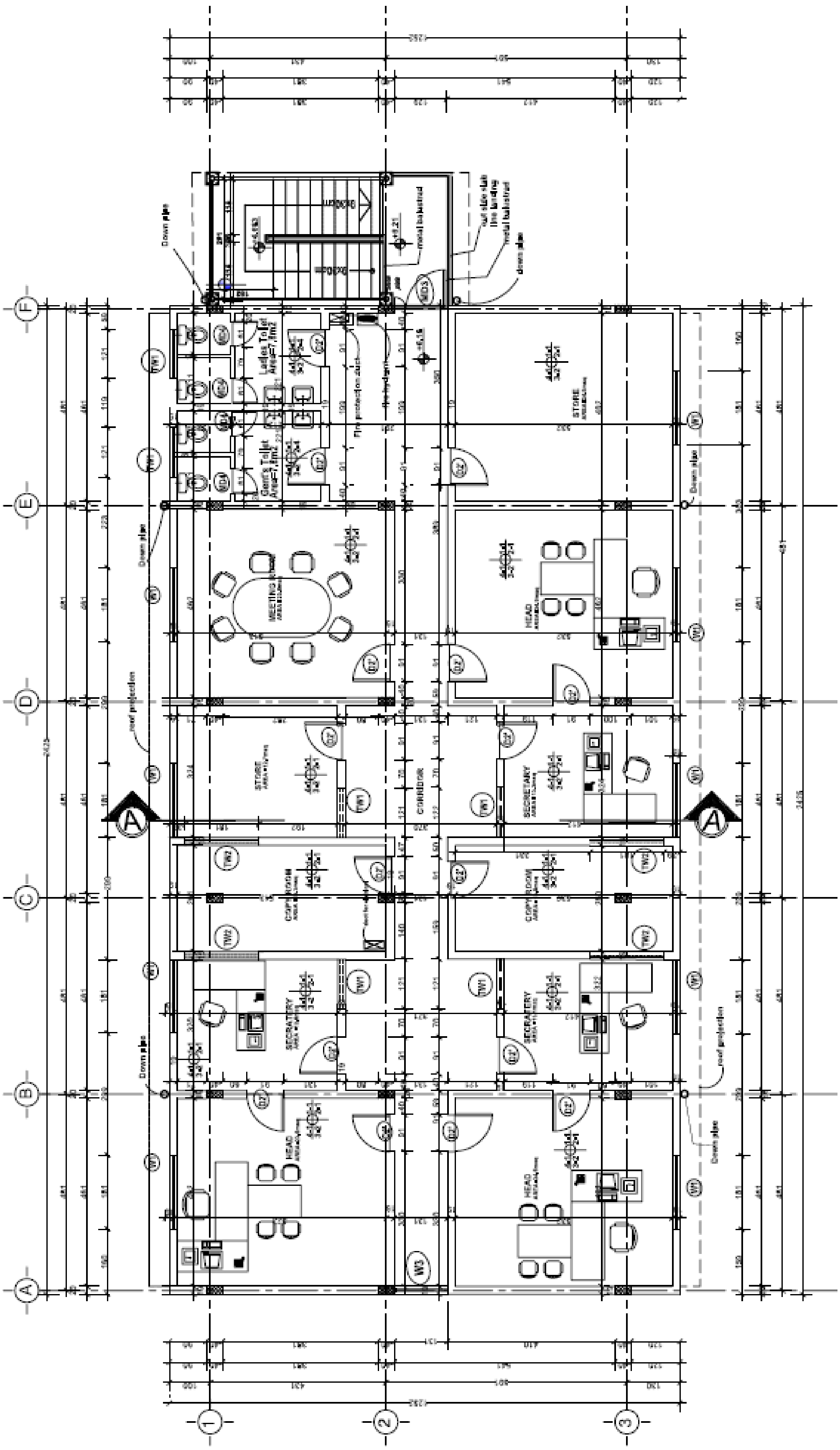


Fig. 3.4c Second Floor Plan

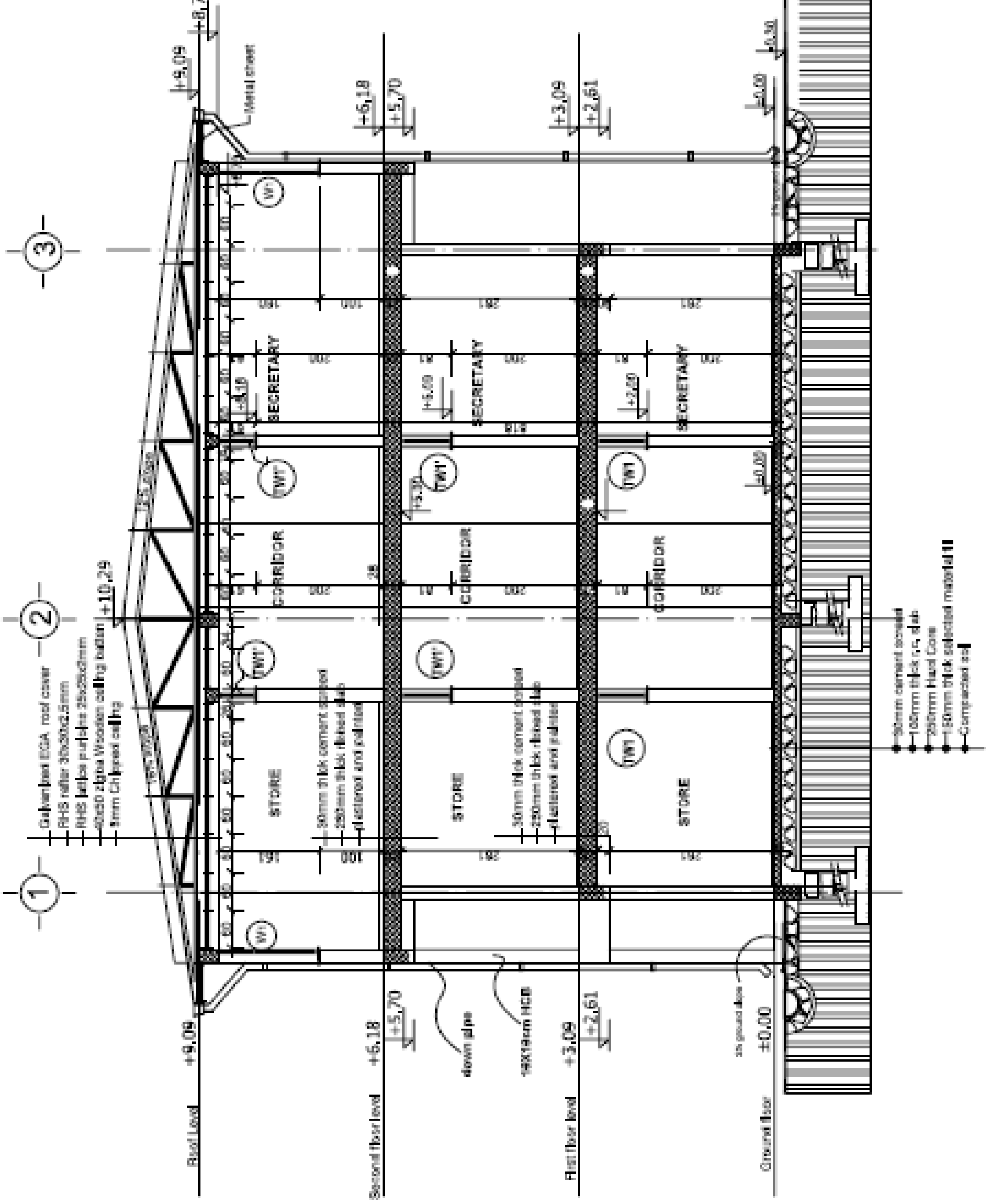


Fig. 3.4d Section A-A

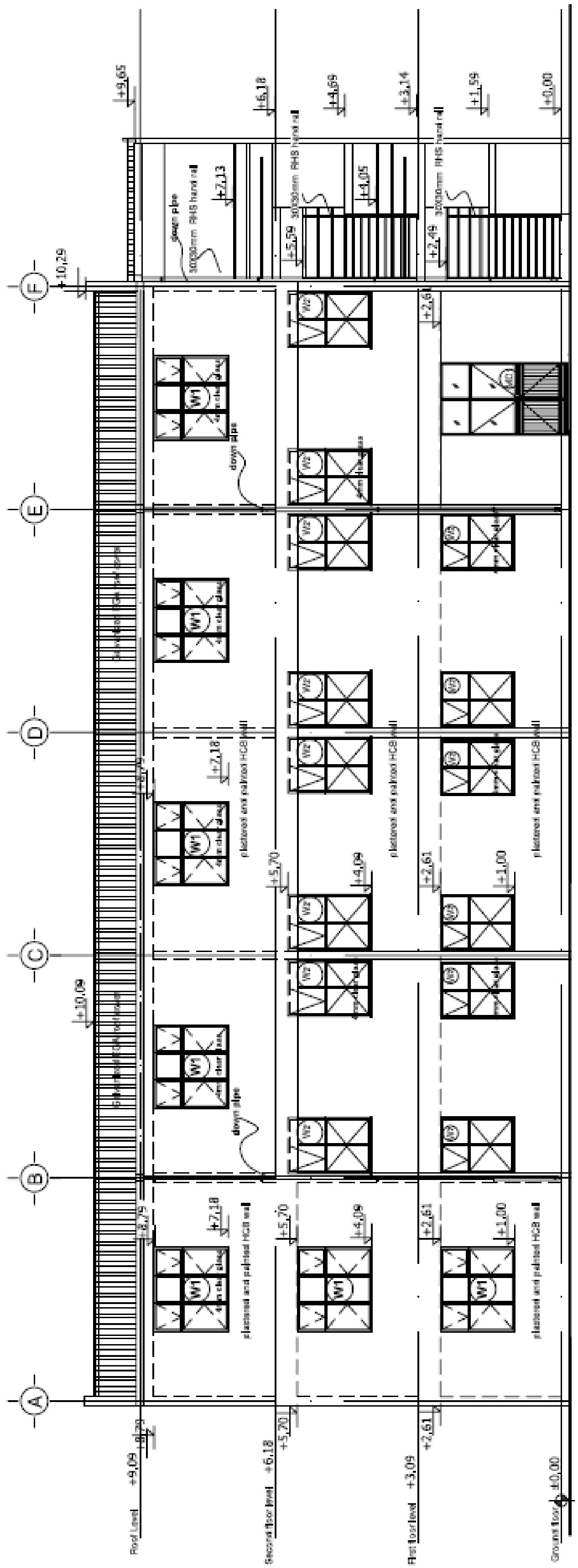


Fig. 3.4e Front Elevation

Fig. 3.4 Floor Plans, Elevation and Section of Case Study Building

All the drawings showing the structural system layout are presented on Fig. 3.5a to Fig. 3.5d. Note on the layout that secondary beams (beams spanning parallel to the pre-cast elements) are not provided on axis ‘2’ of the first floor and on axes ‘1’, ‘2’ and ‘3’ of the second floor and instead these beams are placed as a periphery beam on axes ‘1’’ and ‘3’’ of the second floor. In addition to this, there are no beams on axes ‘B’, ‘C’, ‘D’ and ‘E’ at the roof level to tie the columns together.

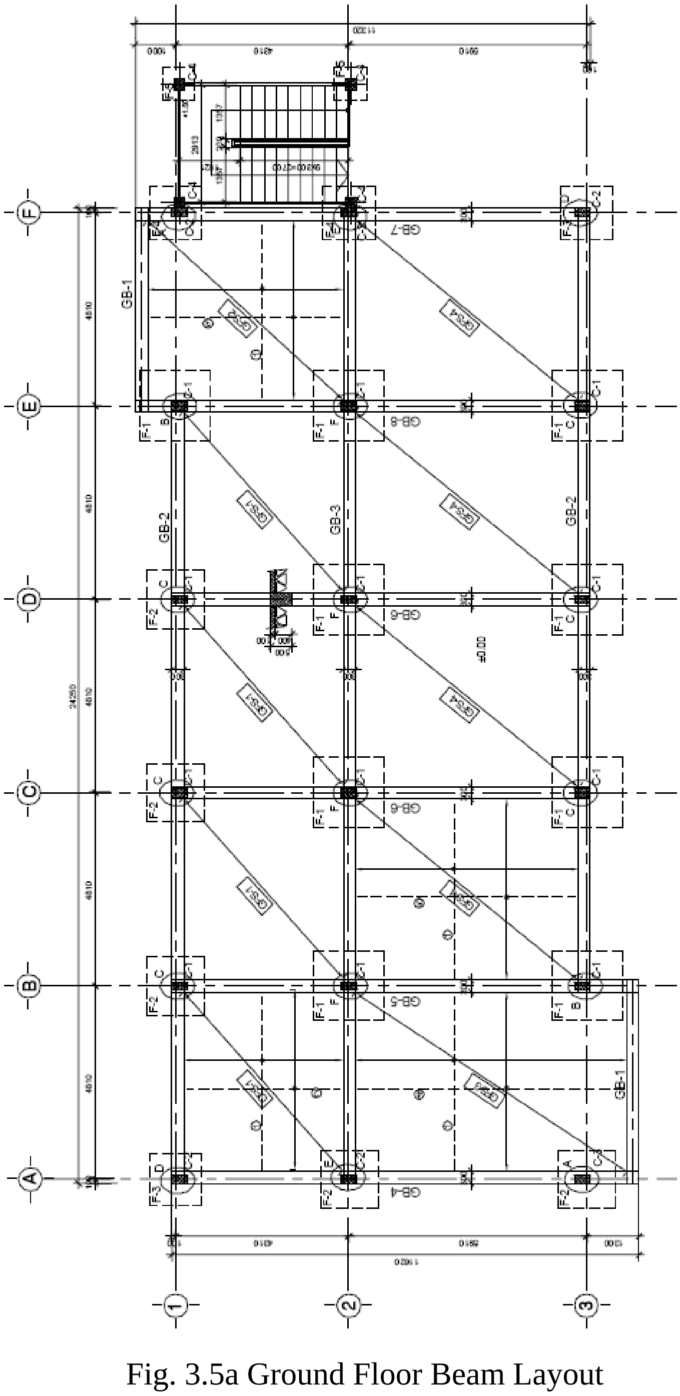


Fig. 3.5a Ground Floor Beam Layout

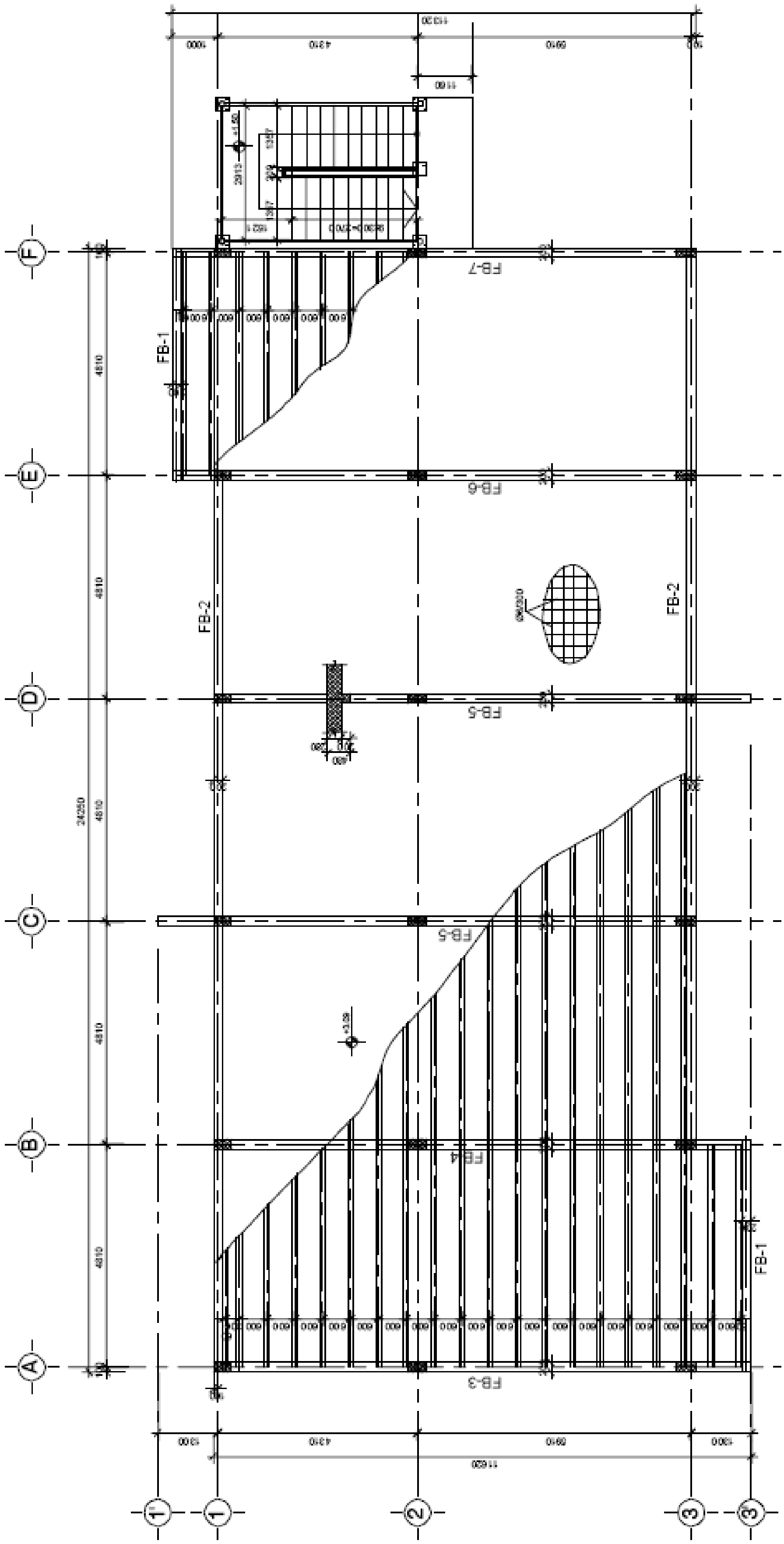


Fig 3.5b First Floor Pre-cast and Beam Layout

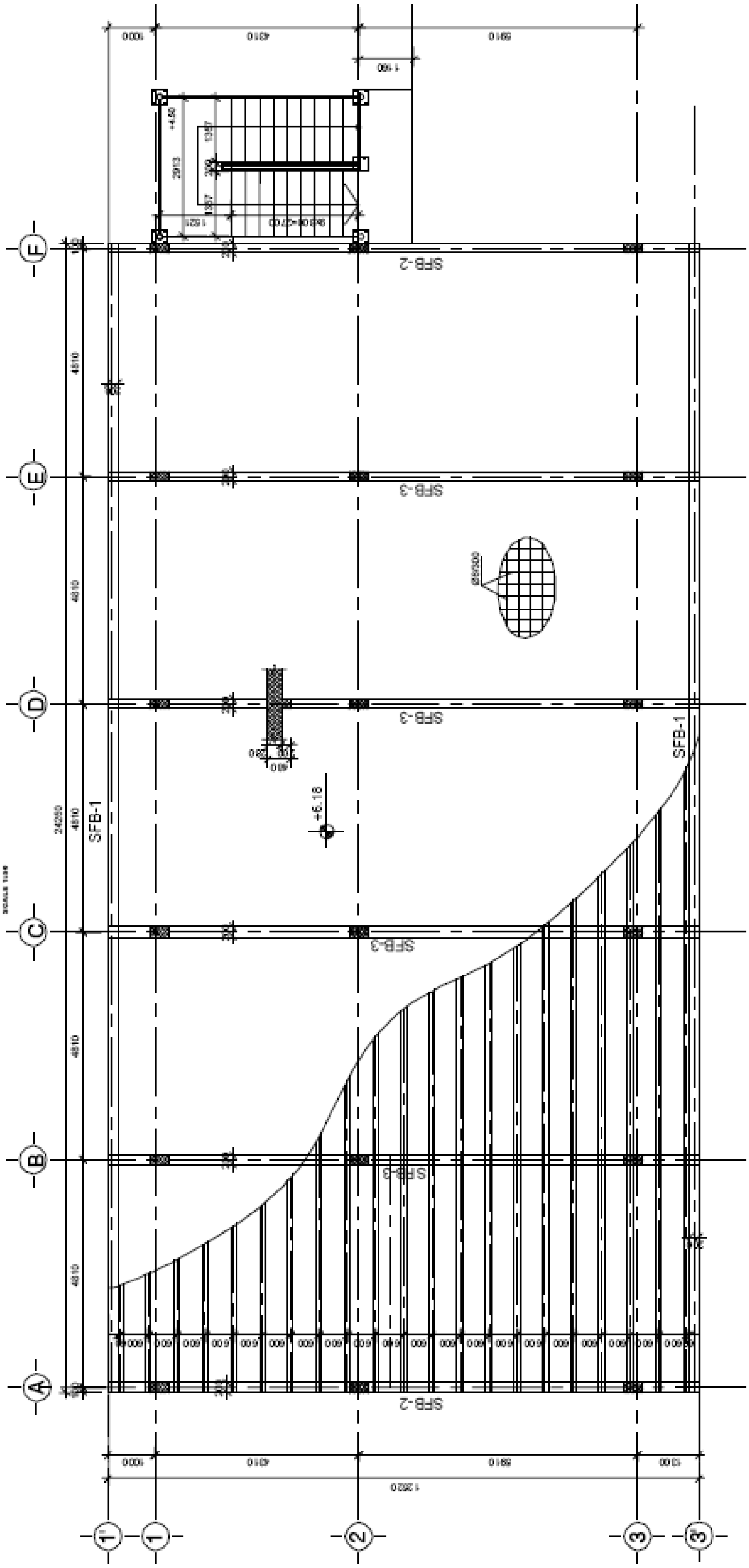


Fig. 3.5c Second Floor Pre-cast and Beam Layout

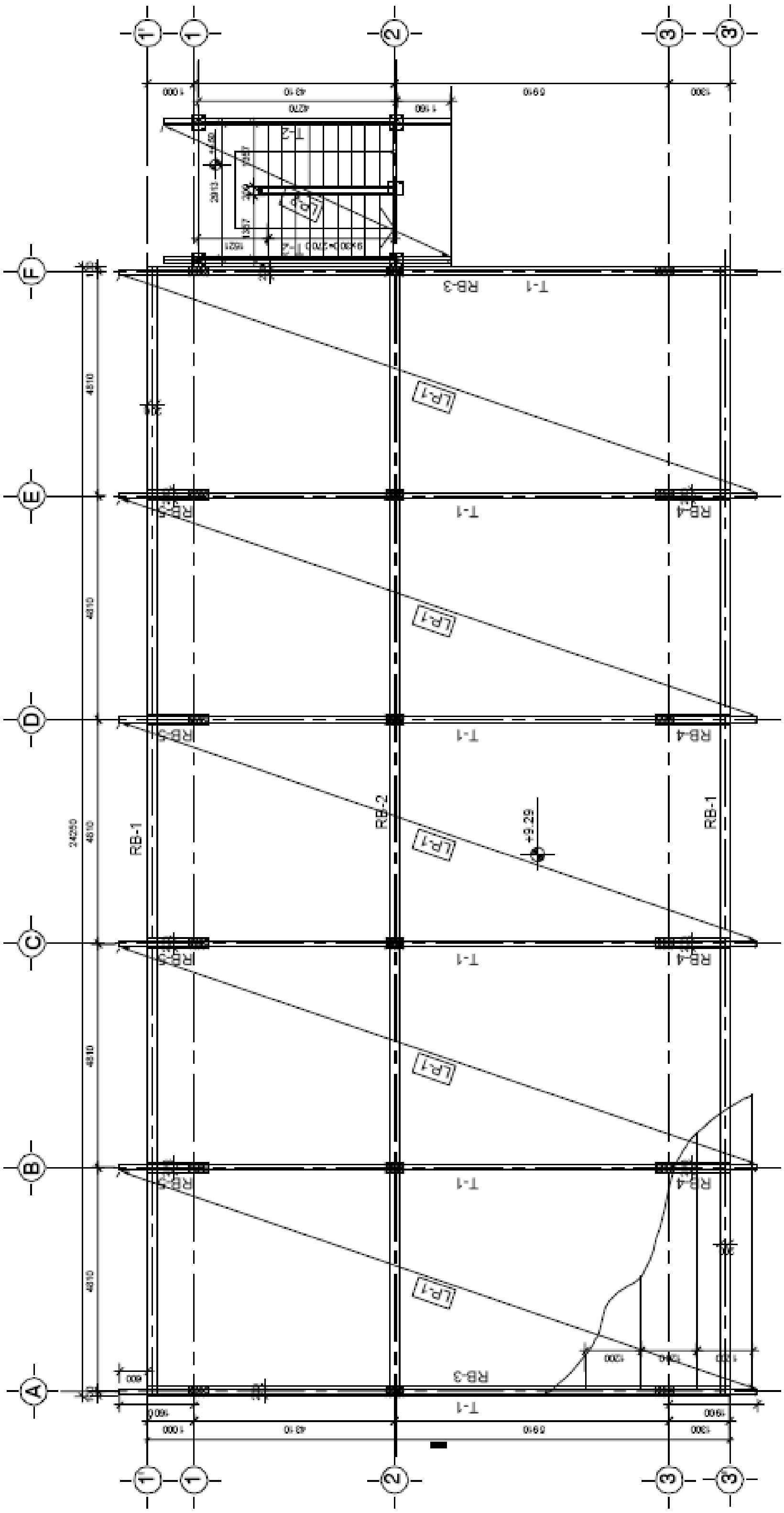


Fig. 3.5d Roof Beam and Roof Truss Layout

Fig. 3.5 Structural Layout of case Study Building

The procedure followed to carry out this case study was first to prepare the mathematical model of the building using SAP 2000 Ver.12 with the same amount of reinforcement and member sizing as provided by the designers of the building. Then the same procedure as presented in the preceding section of this document was followed to prepare the model and conduct pushover analysis. After this the outputs from this analysis were used to evaluate the story ductility demands at different levels of load (base shear) increments from which %base shear yield strength Vs ductility demand curves for the two principal directions were prepared.

On the second part of this case study, the detailing provided for the beams and columns of the building was checked against EBCS 8, 1995 special detailing provisions for these elements while designing for 'DCM'. The structure was evaluated adopting the summary of the special provisions presented in Sec. (2.2).

Table 3.6 Evaluation of detailing of the case study building against the special detailing provisions of EBCS 8, 1995 while designing for 'DCM'

3.6a Special Provisions For Beams of DCM (Ground Floor)

Local Ductility								
l_{cr}	ϕ_{hoops}	Spacing of hoops wiith in critical regions (min. of) (mm)				Placement of first hoop	Min no of ϕ14mm S400 bars	
1.5h_w	≥6mm	$h_w/4$	24 ϕ_{hoops}	200	7ϕ_{main}	<50mm	2-top	
		125	192	200	98		2-bot	**Recommended**
Not defined	**8mm**	**Dia8@200**					**4/3-top, 3-bottom**	**Provided**
Not fulfilled	**OK**	**Not fulfilled**				**OK**	**OK**	**Remark**
	ρ_{max} =	$0.65\,(f_{cd}/f_{yd})*(\rho'/\rho) + 0.0015$ =					**0.0227**	**Recommended**
							0.009	**Provided**
							OK	**Remark**
Specific Measures							**Provided**	**Remark**
	Min Width =	200mm					**300mm**	**OK**
	Min. Width : Height ratio of web =	0.25					**0.5**	**OK**
	Min amt. of -ve. Bar along the length =	25% of max provided					**>25%**	**OK**

3.6b Special Provisions For Beams of DCM (First Floor)

Local Ductility								
l_{cr}	ϕ_{hoops}	Spacing of hoops wiith in critical regions (min. of) (mm)				Placement of first hoop	Min no of ϕ14mm S400 bars	
1.5h_w	≥6mm	$h_w/4$	24 ϕ_{hoops}	200	7ϕ_{main}	<50mm	2-top	
		120	192	200	140		2-bot	**Recommended**
Not defined	**8/10mm**	**Dia8@200, Dia 10@180,Dia10@130mm**						**Provided**
Not fulfilled	**OK**	**Not fulfilled**				**OK**	**OK**	**Remark**
	ρ_{max} =	$0.65\,(f_{cd}/f_{yd})*(\rho'/\rho) + 0.0015$ =					**0.0227**	**Recommended**
							0.016	**Provided**
							OK	**Remark**
Specific Measures							**Provided**	**Remark**
	Min Width =	200mm					**200mm**	**OK**
	Min. Width:Height ratio of web =	0.25					**0.42**	**OK**
	Min amt. of -ve. Bar along the length =	25% of max provided					**>25%**	**OK**

3.6c Special Provisions For Beams of DCM (Second Floor)

Local Ductility								
l_{cr}	ϕ_{hoops}	Spacing of hoops wiith in critical regions (min. of) (mm)				Placement of first hoop	Min no of ϕ14mm S400 bars	
$1.5h_w$	≥6mm	$h_w/4$	24 ϕ_{hoops}	200	7ϕ_{main}	<50mm	2-top	
		120	192	200	112		2-bot	**Recommended**
Not defined	**8/10mm**	**Dia8@200, Dia 10@180,Dia10@130mm**						**Provided**
Not fulfilled	**OK**	**Not fulfilled**				**OK**	**OK**	**Remark**
		ρ_{max} = 0.65 (f_{cd}/f_{yd})*(ρ'/ρ) + 0.0015 =					**0.0227**	**Recommended**
							0.016	**Provided**
							OK	**Remark**
Specific Measures							**Provided**	**Remark**
		Min Width =	200mm				**200mm**	**OK**
		Min. Width : Height ratio of web =	0.25				**0.42**	**OK**
		Min amt. of -ve. Bar along the length =	25% of max provided				**>25%**	**OK**

3.6d Special Provisions For Beams of DCM (Roof)

Local Ductility								
l_{cr}	ϕ_{hoops}	Spacing of hoops wiith in critical regions (min. of) (mm)				Placement of first hoop	Min no of ϕ14mm S400 bars	
$1.5h_w$	≥6mm	$h_w/4$	24 ϕ_{hoops}	200mm	7ϕ_{main}	<50mm	2-top	
		75	192	200mm	98		2-bot	**Recommended**
Not defined	**8/10mm**	**Dia8@200**						**Provided**
Not fulfilled	**OK**	**Not fulfilled**				**OK**	**OK**	**Remark**
		ρ_{max} = 0.65 (f_{cd}/f_{yd})*(ρ'/ρ) + 0.0015 =					**0.0227**	**Recommended**
							0.005	**Provided**
							OK	**Remark**
Specific Measures							**Provided**	**Remark**
		Min Width =	200mm				**200mm**	**OK**
		Min. Width : Height ratio of web =	0.25				**0.67**	**OK**
		Min amt. of -ve. Bar along the length =	25% of max provided				**>25%**	**OK**

3.6e Special Provisions For Columns of DCM

Local Ductility							
l_{cr}	ϕ_{hoops}	Spacing of hoops wiith in critical regions (min. of) (mm)			Max dist.b/n main bars		
max($1.5d_c$;l_{cl}/6;450mm)	≥0.35$\phi_{main,max}\sqrt{(f_{ydL}/f_{ydw})}$=7mm	$b_o/3$	150	7ϕ_{main}	200mm		
	≥6mm	67		112			**Recommended**
Not defined	**6mm**	**200mm**			**>200mm**		**Provided**
Not fulfilled	**NO**	**NO**			**NO**		**Remark**
						Provided	**Remark**
	ρ_{max} =	0.04				**0.0356**	**OK**
	ρ_{min} =	0.01				**0.081**	**Not Fulfilled**
	on symmetrical x-sections, ρ' =	ρ					**Not satisfied on sections'C-C' and 'D-D'**
Specific Measures						**Provided**	**Remark**
	Min x-sectional dimension =	250mm				**200mm**	**NO**
	If Θ > 0.1 the x-sectional dimension of the col. Should be ≥ **Θmax = 0.21 < 0.25**	1/10*larger dist. b/n the pt. of contra-flexure and the ends of the columns **= 310mm**				**200mm**	**NO**
In the lower two stories the min. spacing of hoops recommended in critical regions shall be extended to an additional length of 0.5*l_{cr}						**Not provided**	**Not Fulfilled**
At least one intermediate bar shall be provided between corner bars along each column sides in order to enhance the integrity of beam-column joints.						**Not provided**	**Not Fulfilled**
Inter-story drift limitation Δ ≤ 0.01*h, **Δ_{max}=10.31mm < 0.01*h=30.9mm**							**OK**

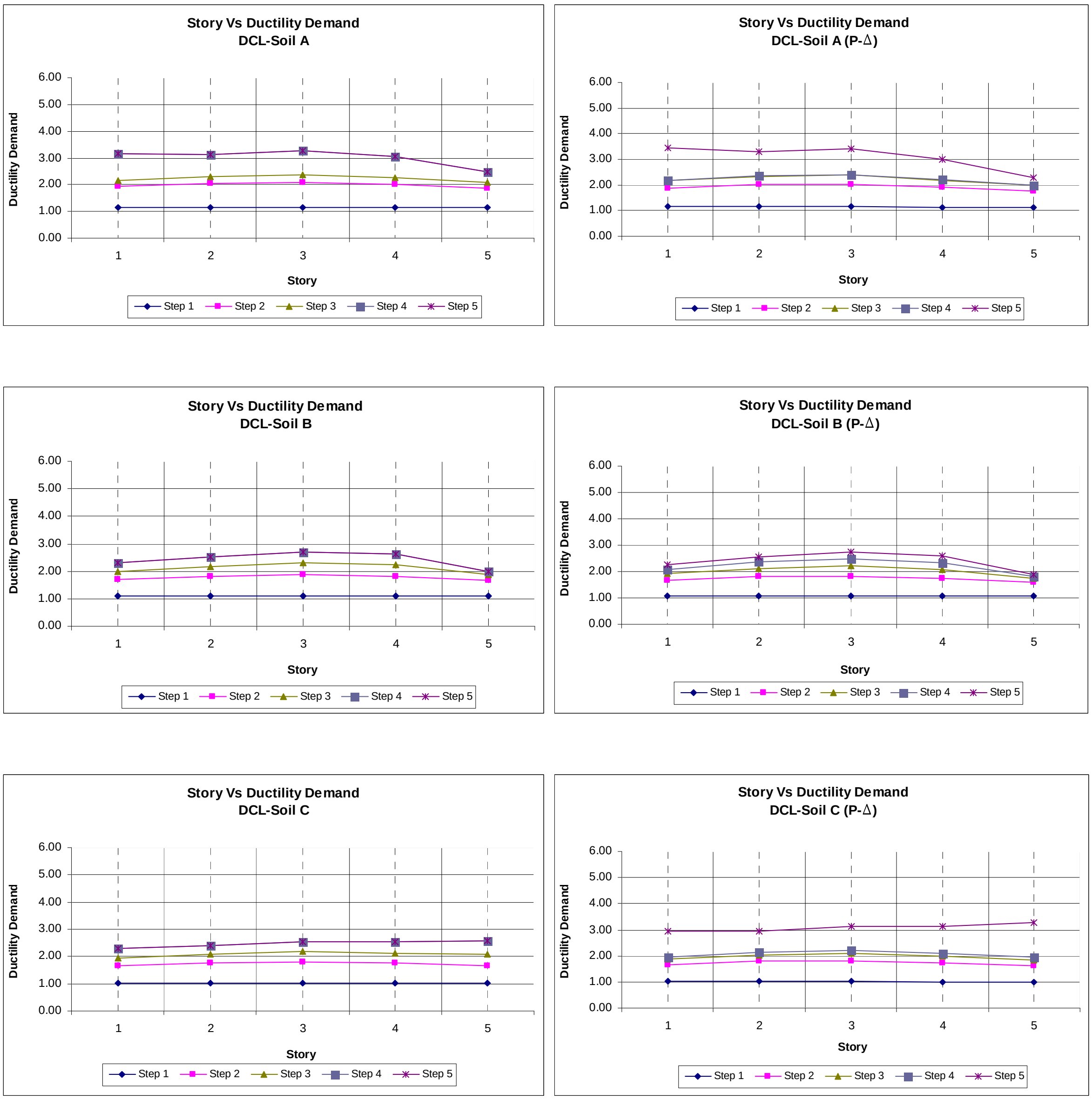
A further investigation on the strength and ductility demand relationship of the G+2 building was carried out after modifying the model to meet the special provisions for columns and beams by EBCS 8, 1995. While modifying the model, there was no change made on the amount of main reinforcement provided. Instead, only the following adjustments were made to the model.

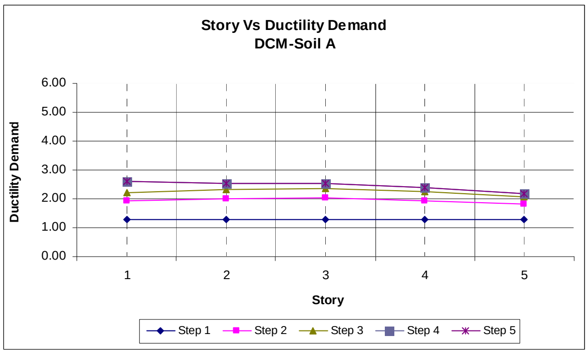
a. Satisfy minimum required dimensions of column size
    - Change column size from 400x200mm to 400x300mm
        - Resulted increase in volume of concrete = 9.63m^3.
b. Provide secondary beams, i.e. beams spanning between columns parallel to the pre-cast beams with the same cross sections as the girders (200x480h).
        - Resulted increase in volume of concrete = 2.88m^3.
c. Re-align beam ‘SFB-1’ on Fig. 3.5c from axes ‘(1’)’ and ‘(3’)’ to ‘(1)’ and ‘(3)’.
d. Adjust the spacing between stirrups on critical regions to satisfy the recommendations for ‘DCM’ design.

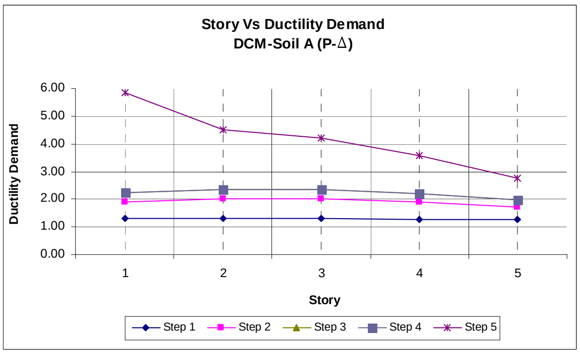
4. Results and Discussions

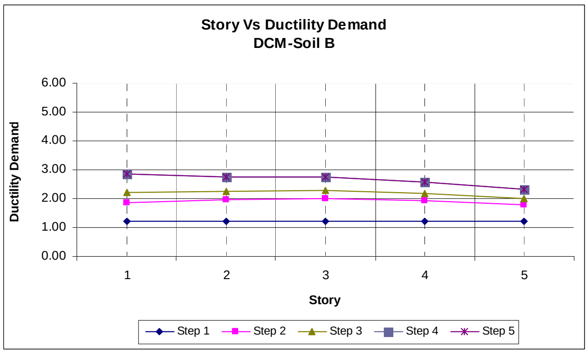
4.1. Results

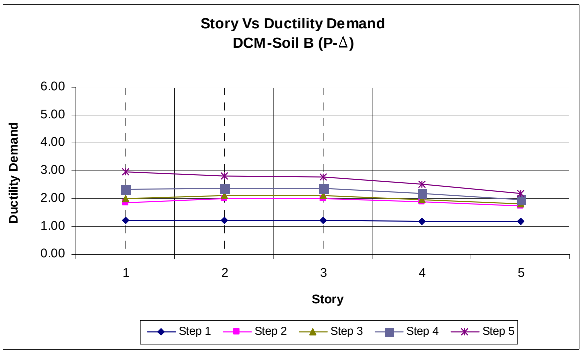
The relationship between strength and ductility demand for the five and ten story shear frames were developed following the procedure presented in the preceding section. The first step to this end was to determine the story ductility demands for each load increment. Plots of the story ductility demands for all the models are presented in Fig 4.1 and the curves clearly show that the ductility demand for multi degree of freedom systems vary over the story levels.

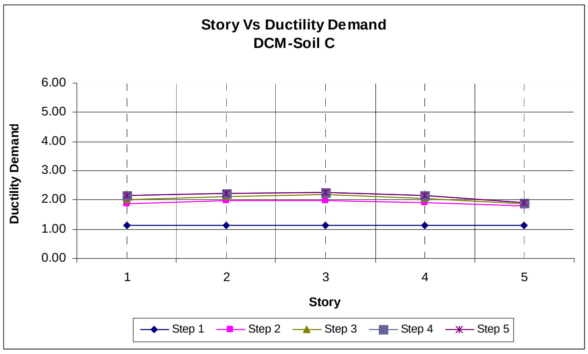


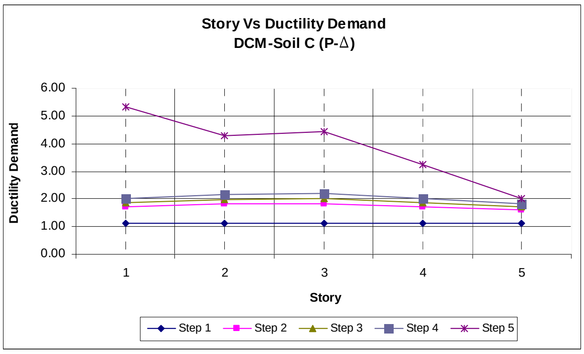

Story Vs Ductility Demand
DCM-Soil A
Ductility Demand
Story
Step 1 Step 2 Step 3 Step 4 Step 5

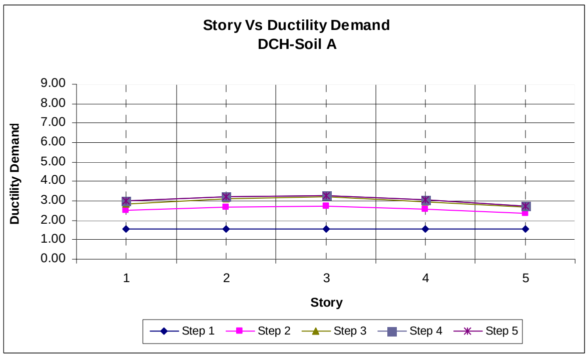

Story Vs Ductility Demand
DCM-Soil A (P-Δ)
Ductility Demand
Story
Step 1 Step 2 Step 3 Step 4 Step 5

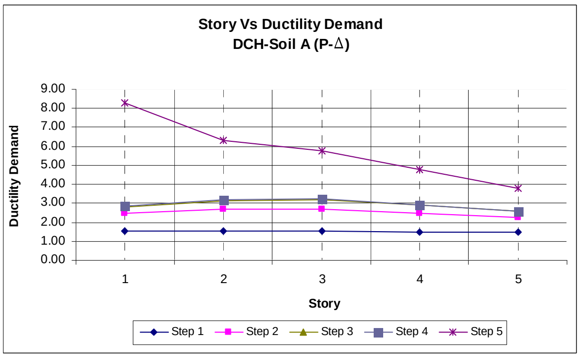

Story Vs Ductility Demand
DCM-Soil B
Ductility Demand
Story
Step 1 Step 2 Step 3 Step 4 Step 5

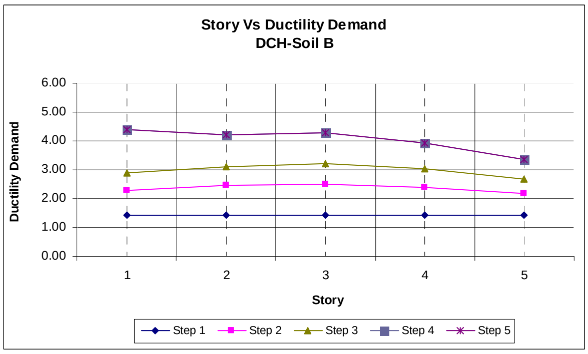

Story Vs Ductility Demand
DCM-Soil B (P-Δ)
Ductility Demand
Story
Step 1 Step 2 Step 3 Step 4 Step 5

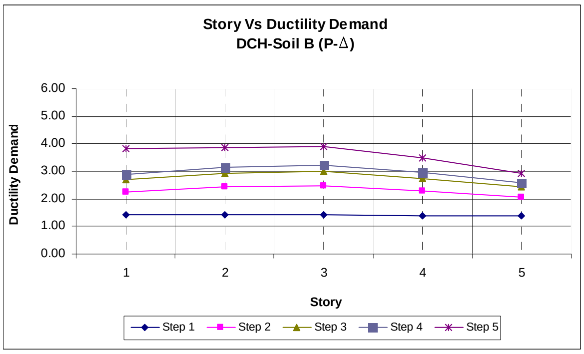

Story Vs Ductility Demand
DCM-Soil C
Ductility Demand
Story
Step 1 Step 2 Step 3 Step 4 Step 5

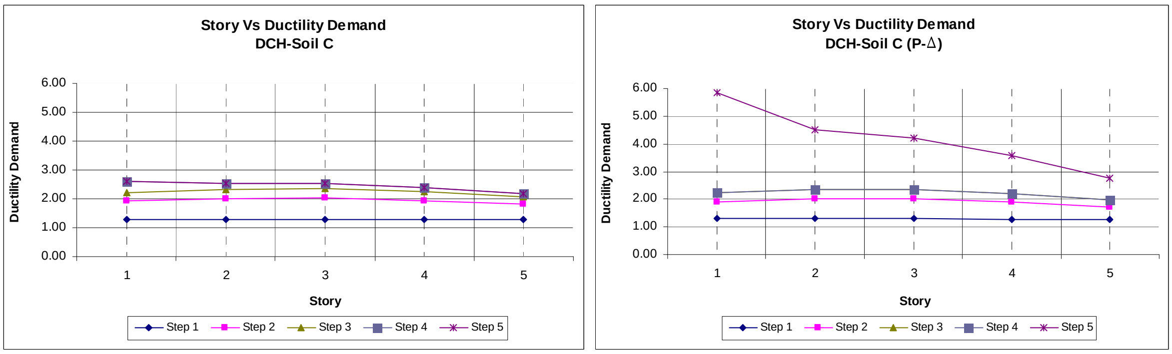

Story Vs Ductility Demand
DCM-Soil C (P-Δ)
Ductility Demand
Story
Step 1 Step 2 Step 3 Step 4 Step 5


Story Vs Ductility Demand
DCH-Soil A
Ductility Demand
Story
Step 1 Step 2 Step 3 Step 4 Step 5

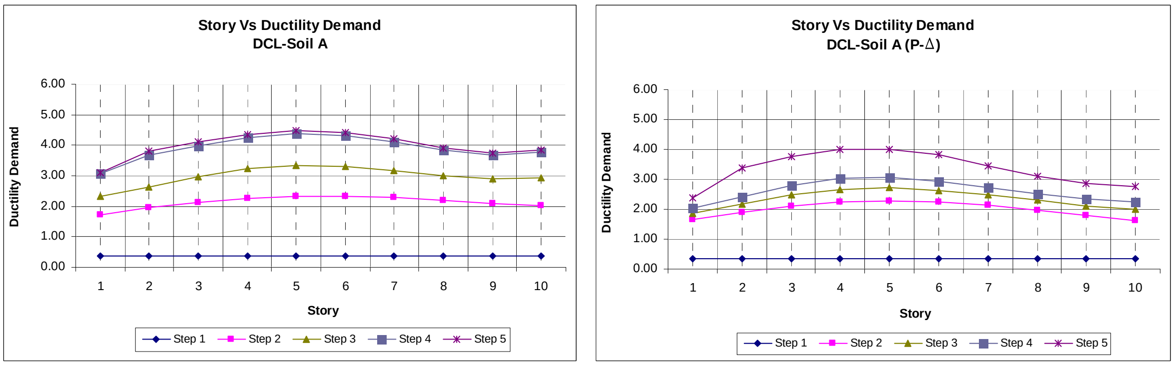

Story Vs Ductility Demand
DCH-Soil A (P-Δ)
Ductility Demand
Story
Step 1 Step 2 Step 3 Step 4 Step 5

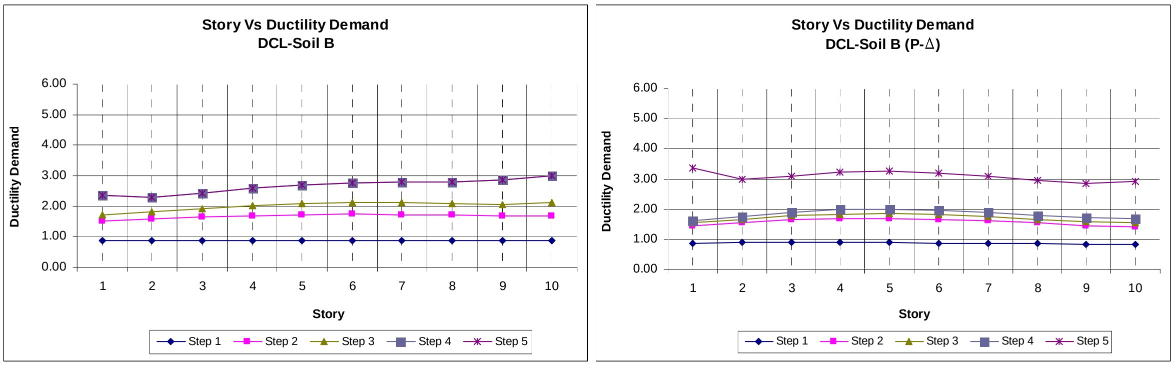

Story Vs Ductility Demand
DCH-Soil B
Ductility Demand
Story
Step 1 Step 2 Step 3 Step 4 Step 5

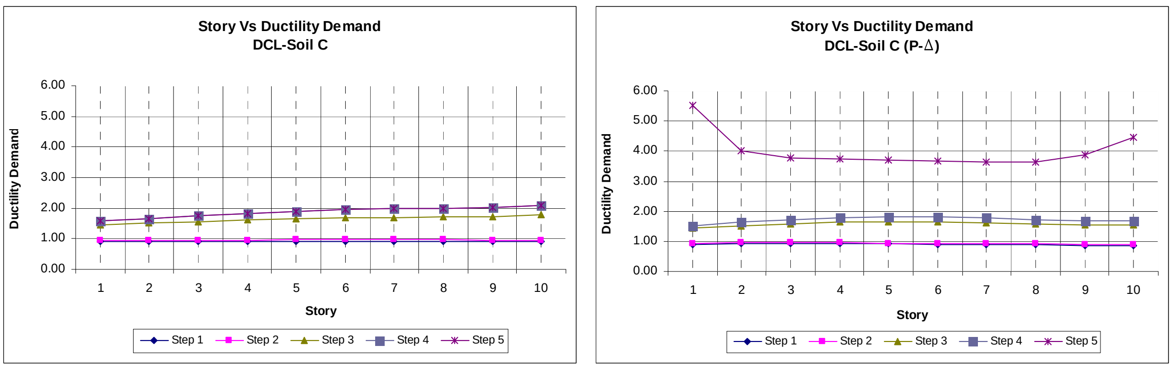

Story Vs Ductility Demand
DCH-Soil B (P-Δ)
Ductility Demand
Story
Step 1 Step 2 Step 3 Step 4 Step 5

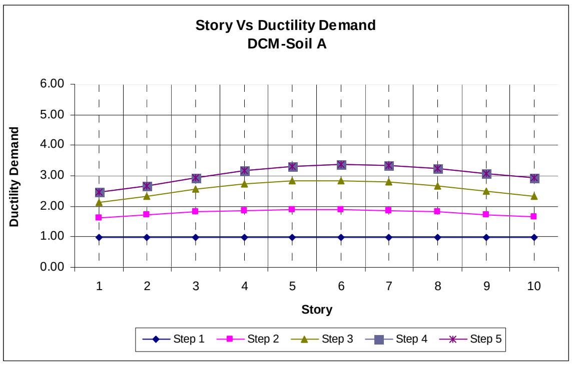


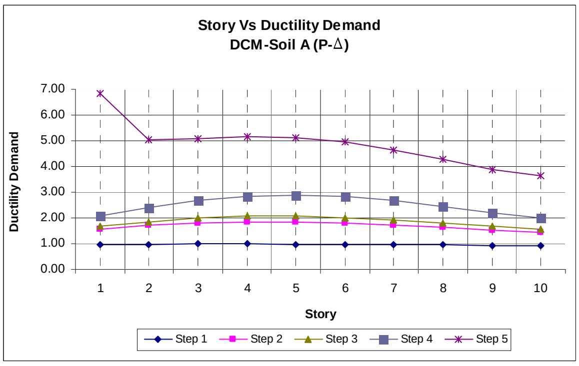
a. 5 story shear frames

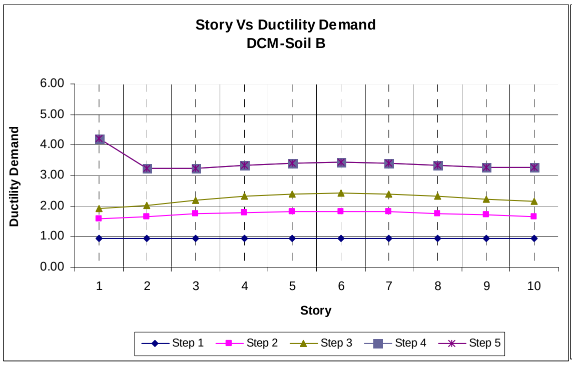


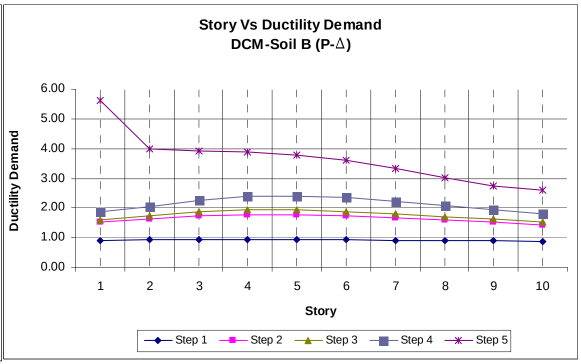


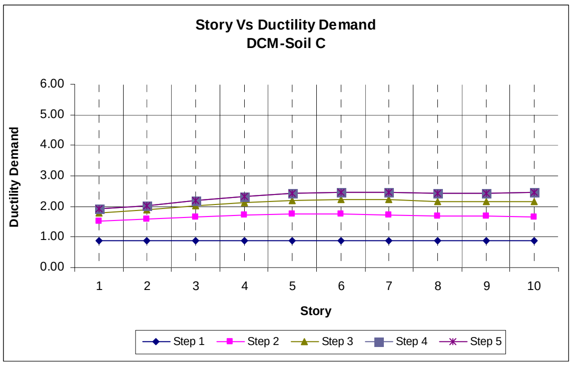


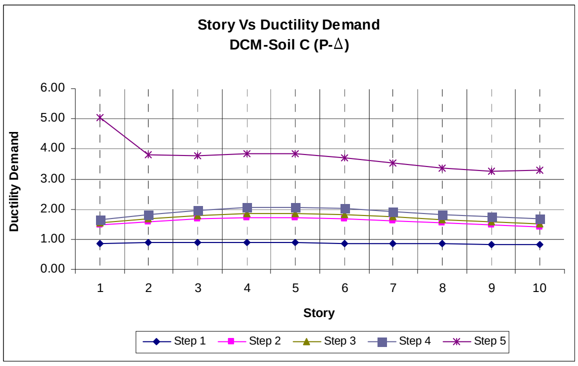

Story Vs Ductility Demand
DCM-Soil A
Ductility Demand
Story
Step 1
Step 2
Step 3
Step 4
Step 5

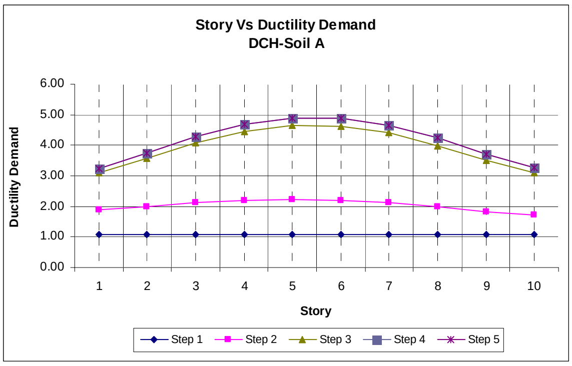

Story Vs Ductility Demand
DCM-Soil A (P-Δ)
Ductility Demand
Story
Step 1
Step 2
Step 3
Step 4
Step 5

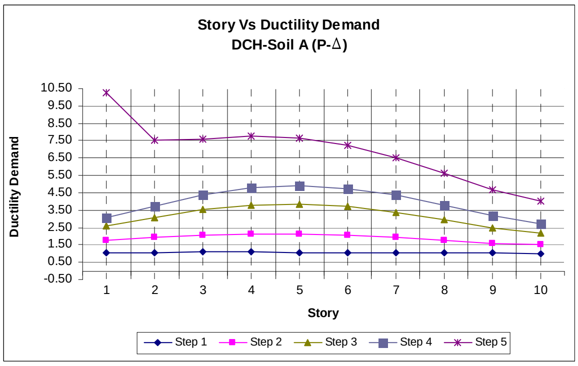

Story Vs Ductility Demand
DCM-Soil B
Ductility Demand
Story
Step 1
Step 2
Step 3
Step 4
Step 5

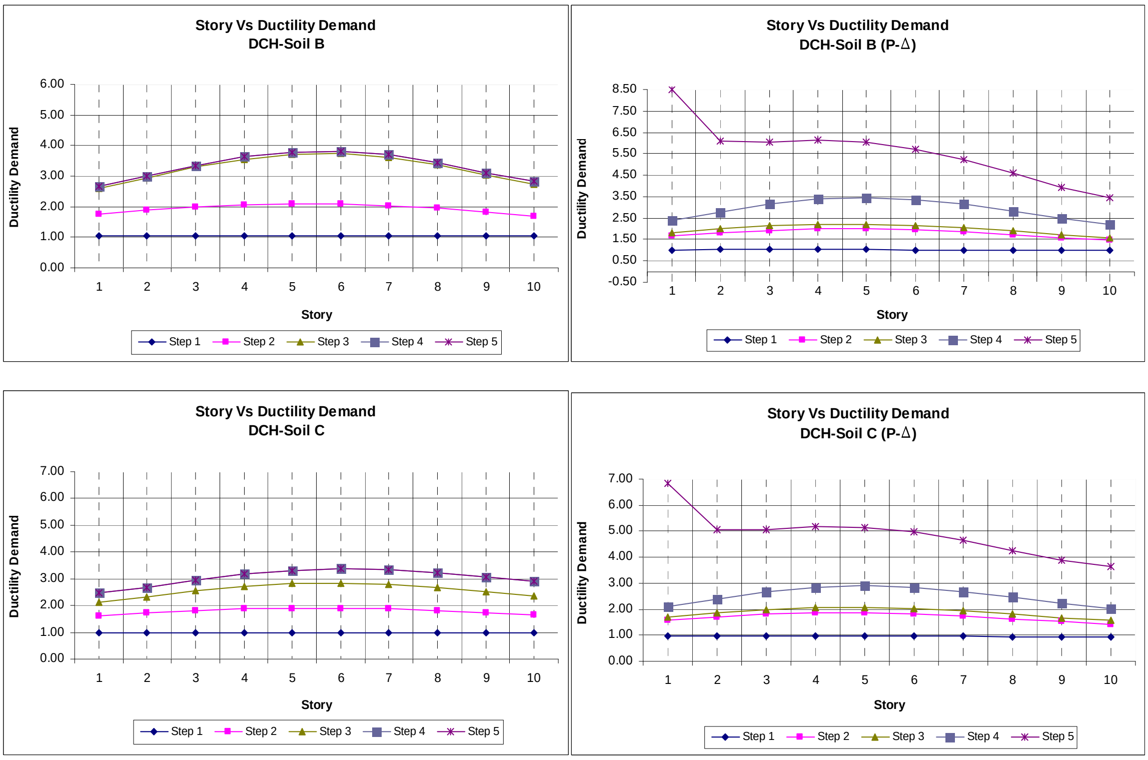

Story Vs Ductility Demand
DCM-Soil B (P-Δ)
Ductility Demand
Story
Step 1
Step 2
Step 3
Step 4
Step 5


Story Vs Ductility Demand
DCM-Soil C
Ductility Demand
Story
Step 1
Step 2
Step 3
Step 4
Step 5


Story Vs Ductility Demand
DCM-Soil C (P-Δ)
Ductility Demand
Story
Step 1
Step 2
Step 3
Step 4
Step 5


Story Vs Ductility Demand
DCH-Soil A
Ductility Demand
Story
Step 1
Step 2
Step 3
Step 4
Step 5

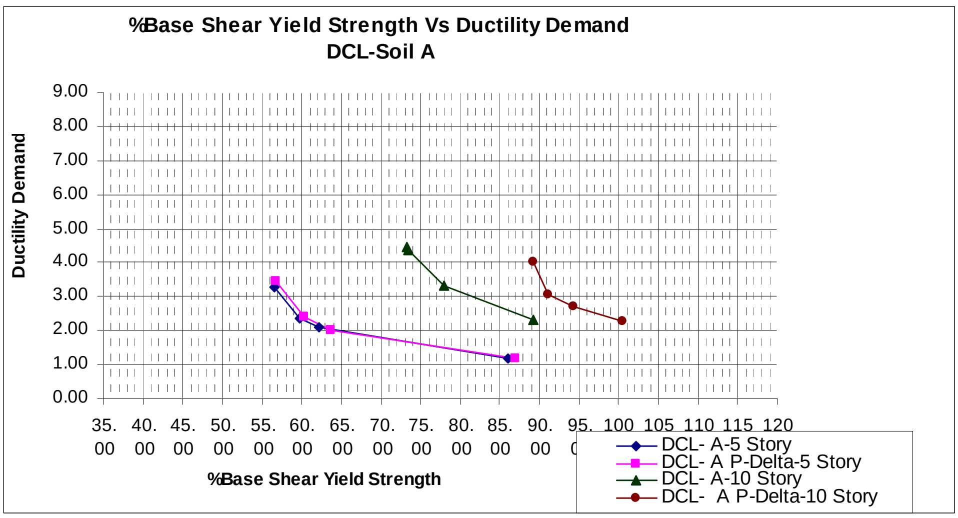

Story Vs Ductility Demand
DCH-Soil A (P-Δ)
Ductility Demand
Story
Step 1
Step 2
Step 3
Step 4
Step 5

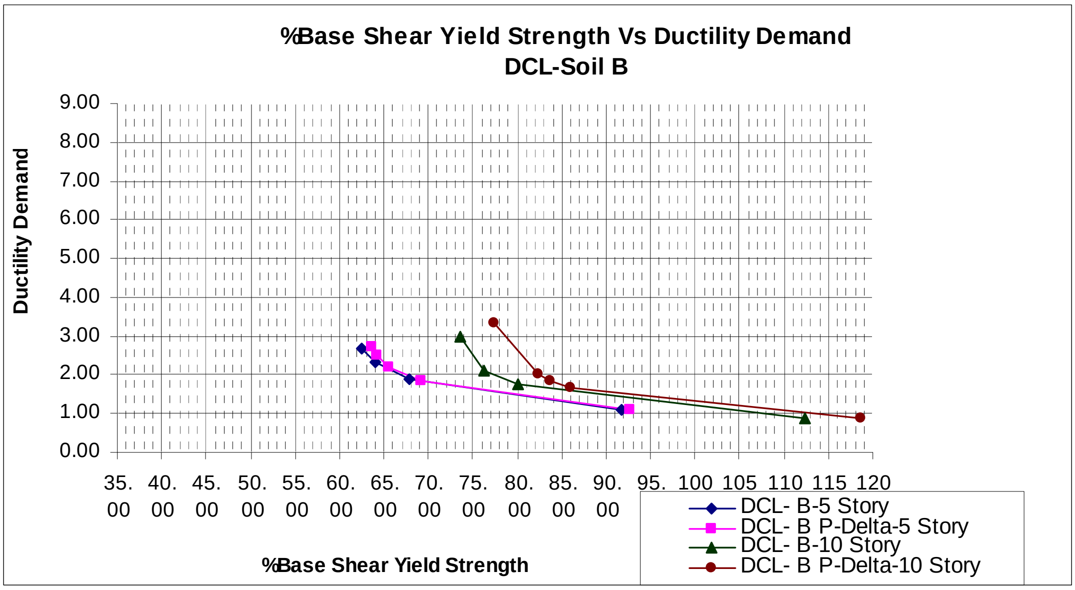


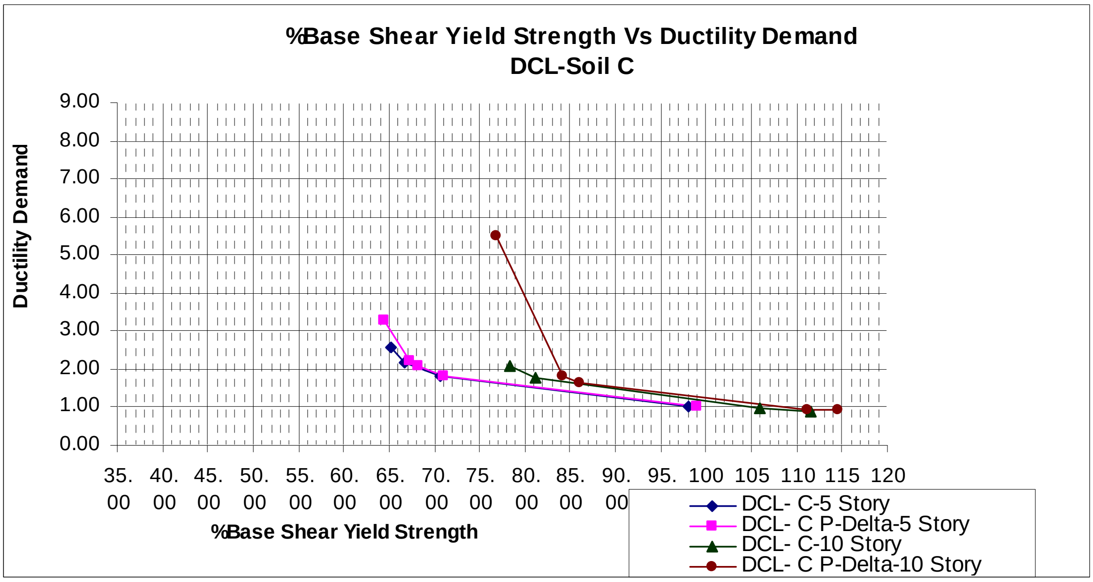
b. 10 story shear frames

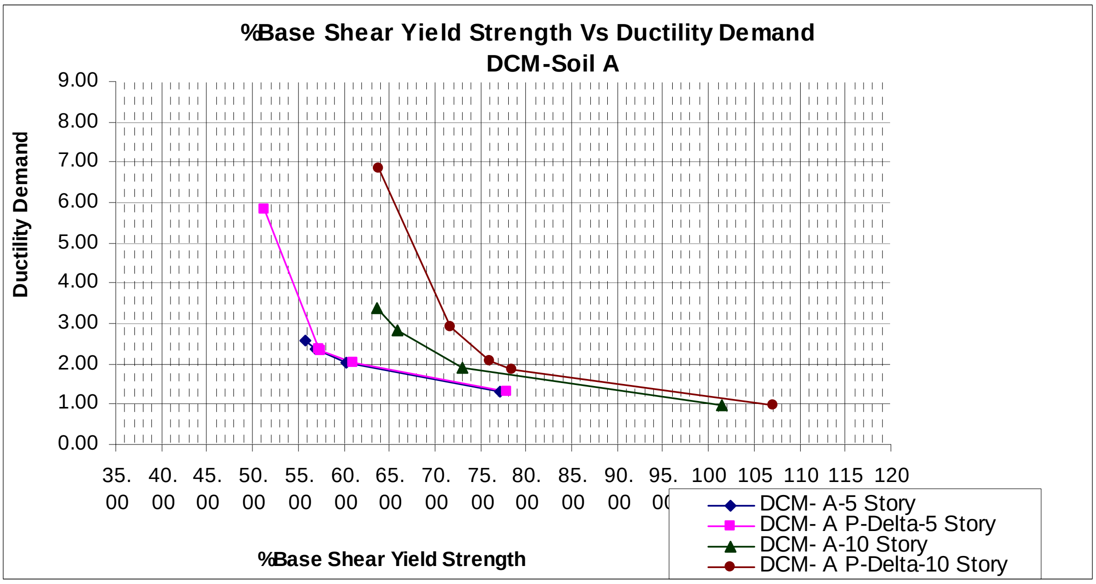
Fig 4.1 Story Ductility Demand

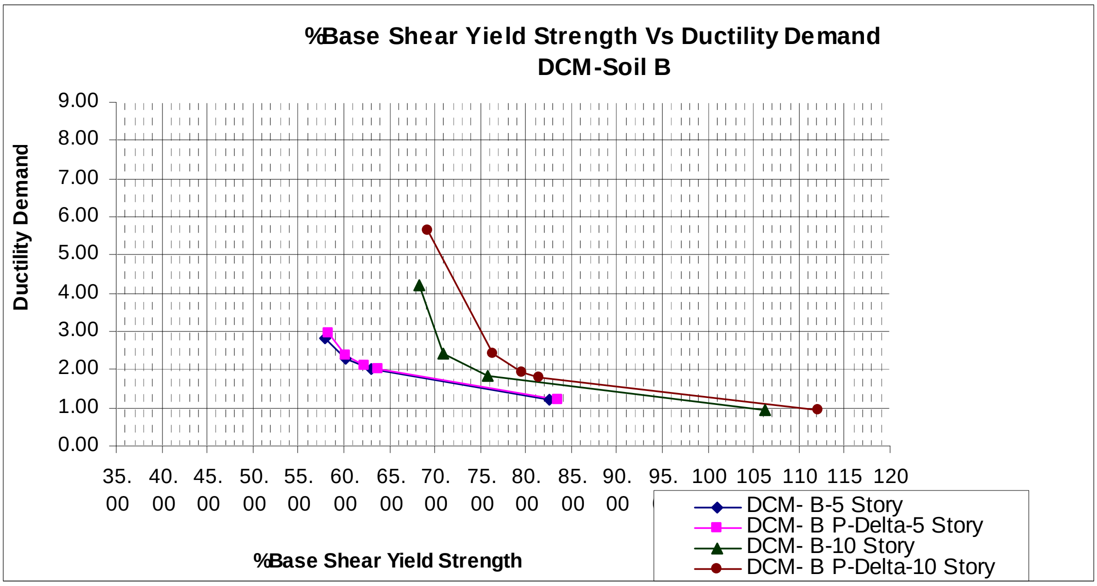
The curves on the left row were developed using the pushover analysis case 'Push-Mat_Nonlin' while 'Push-P-Delta' was used to develop those on the right row. After determining these story ductility demands, plots of the max story ductility demand against the %base shear yield strength as shown in Fig 4.2 were prepared to establish a relation between %base shear yield strength (expressed as a percentage of the design base shear of EBCS8, 1995) and ductility demand for the five and ten story shear frames. The same procedure was also followed for the case study building and the results are presented on Fig. 4.3.

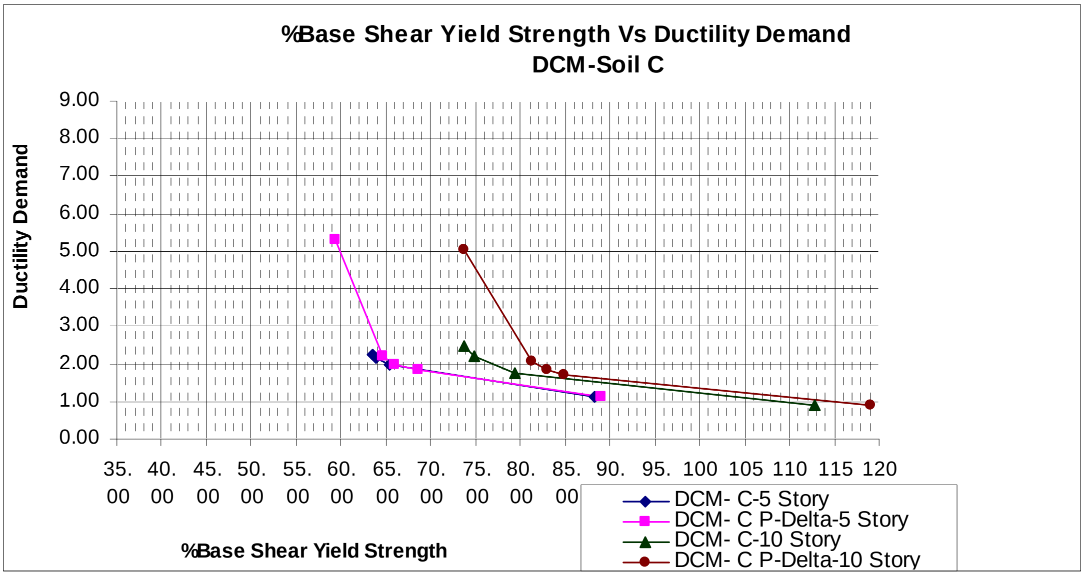


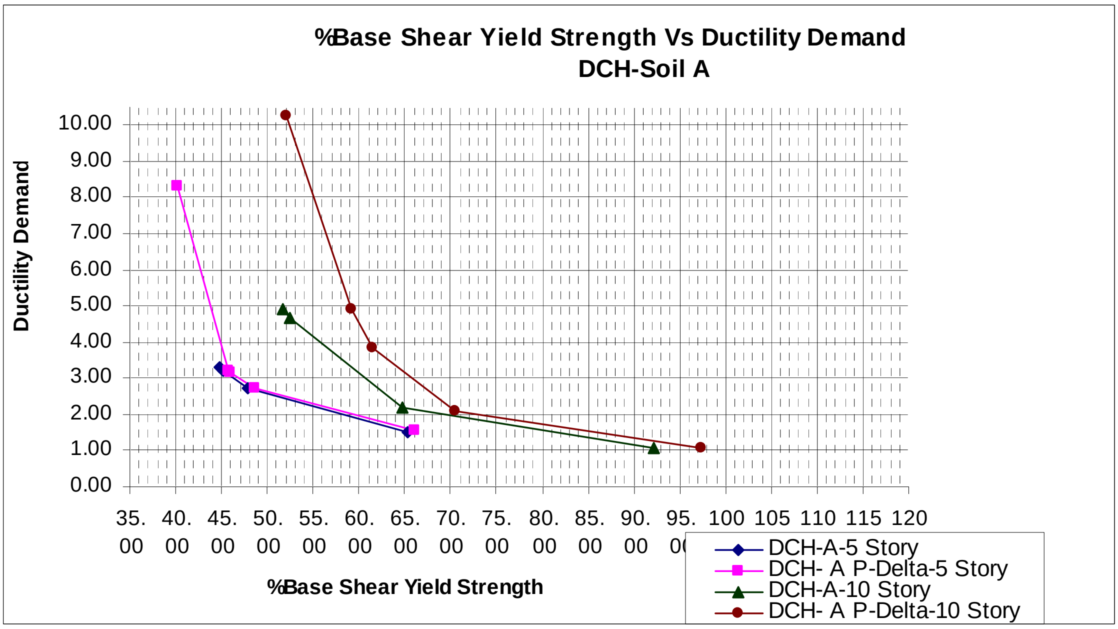

%Base Shear Yield Strength Vs Ductility Demand
DCL-Soil B
Ductility Demand
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
35. 00
40. 00
45. 00
50. 00
55. 00
60. 00
65. 00
70. 00
75. 00
80. 00
85. 00
90. 00
95.
100
105
110
115
120
%Base Shear Yield Strength
DCL- B-5 Story
DCL- B P-Delta-5 Story
DCL- B-10 Story
DCL- B P-Delta-10 Story

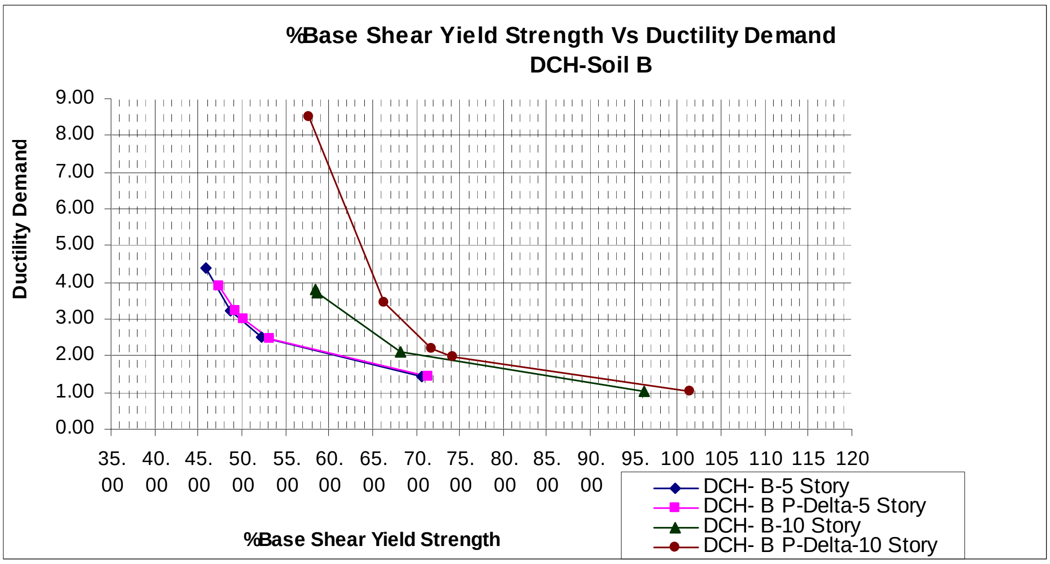

%Base Shear Yield Strength Vs Ductility Demand
DCL-Soil C
Ductility Demand
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
35. 00
40. 00
45. 00
50. 00
55. 00
60. 00
65. 00
70. 00
75. 00
80. 00
85. 00
90. 00
95.
100
105
110
115
120
%Base Shear Yield Strength
DCL- C-5 Story
DCL- C P-Delta-5 Story
DCL- C-10 Story
DCL- C P-Delta-10 Story


%Base Shear Yield Strength Vs Ductility Demand
DCM-Soil A
Ductility Demand
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
35. 00
40. 00
45. 00
50. 00
55. 00
60. 00
65. 00
70. 00
75. 00
80. 00
85. 00
90. 00
95.
100
105
110
115
120
%Base Shear Yield Strength
DCM- A-5 Story
DCM- A P-Delta-5 Story
DCM- A-10 Story
DCM- A P-Delta-10 Story


%Base Shear Yield Strength Vs Ductility Demand
DCM-Soil B
Ductility Demand
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
35. 00 40. 00 45. 00 50. 00 55. 00 60. 00 65. 00 70. 00 75. 00 80. 00 85. 00 90. 00 95. 100 105 110 115 120
%Base Shear Yield Strength
DCM- B-5 Story
DCM- B P-Delta-5 Story
DCM- B-10 Story
DCM- B P-Delta-10 Story


%Base Shear Yield Strength Vs Ductility Demand
DCM-Soil C
Ductility Demand
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
35. 00 40. 00 45. 00 50. 00 55. 00 60. 00 65. 00 70. 00 75. 00 80. 00 85. 00 90. 95. 100 105 110 115 120
%Base Shear Yield Strength
DCM- C-5 Story
DCM- C P-Delta-5 Story
DCM- C-10 Story
DCM- C P-Delta-10 Story


%Base Shear Yield Strength Vs Ductility Demand
DCH-Soil A
Ductility Demand
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
35. 00 40. 00 45. 00 50. 00 55. 00 60. 00 65. 00 70. 00 75. 00 80. 00 85. 00 90. 00 95. 100 105 110 115 120
%Base Shear Yield Strength
DCH-A-5 Story
DCH- A P-Delta-5 Story
DCH-A-10 Story
DCH- A P-Delta-10 Story



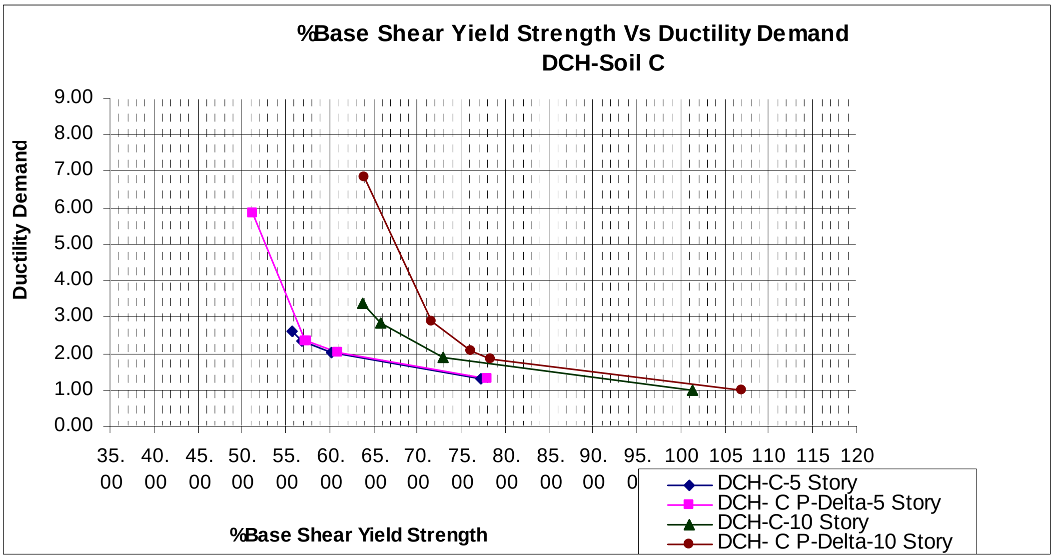
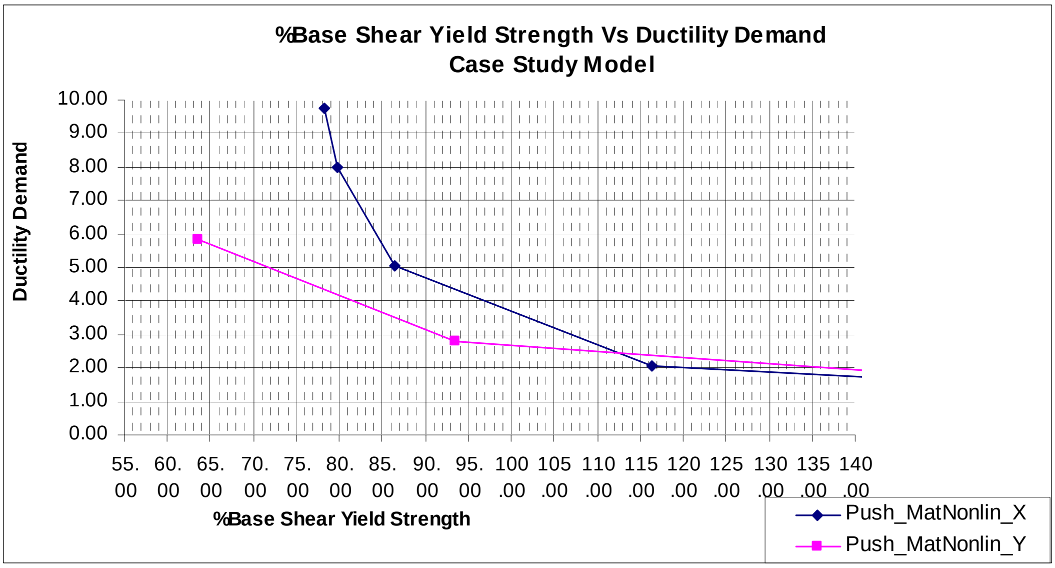
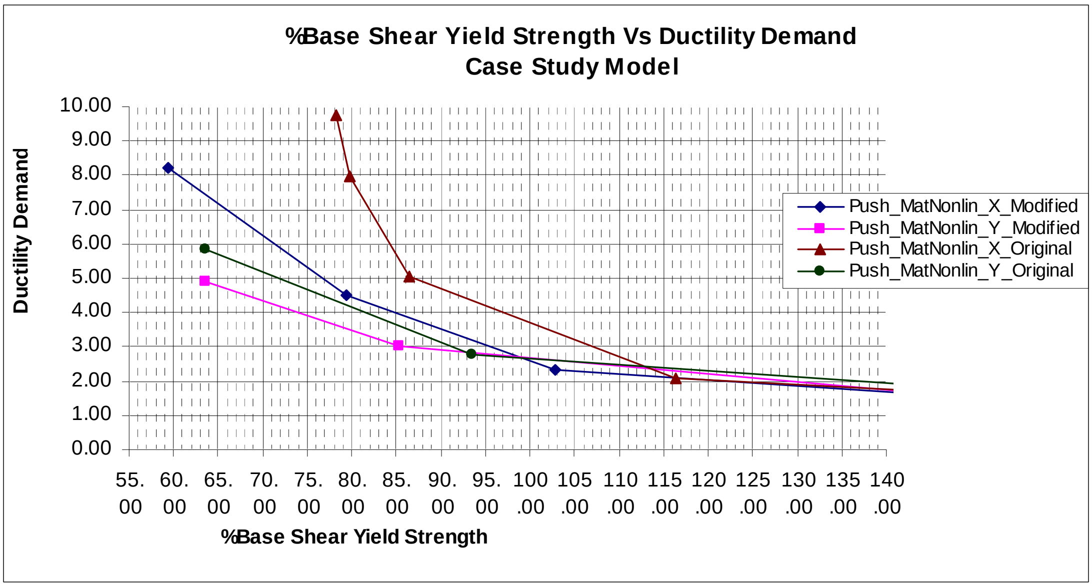


Fig 4.2 %Base shear yield strength Vs Ductility demand curves



a. Original Structure



b. Original and Modified Structure

Fig 4.3 %Base shear yield strength Vs Ductility demand curves (Case Study Building)

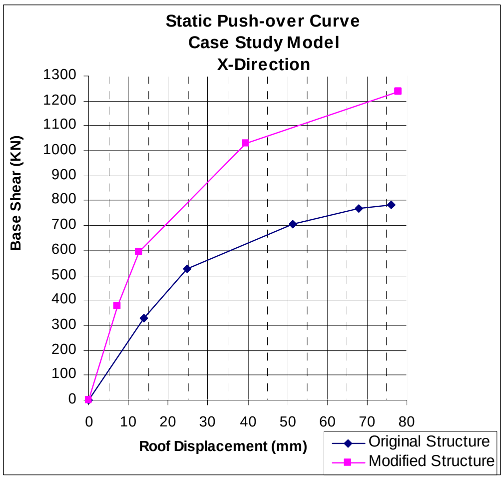


Fig 4.4 Static Pushover curves (Original and Modified Structures)

Once the curves were developed it was possible to determine the ductility demand imposed by the base shear values of the equivalent static method of EBCS 8,1995 and the elastic design spectrum of EBCS 8,1995 and EC8, 2004 on all the models as presented in Table 4.1. Some of the ductility demand values stated on the table are not exact values. Consider for example the ductility demand imposed on the model with sub-soil class ‘B’ and ductility class ‘DCH’ by RSEBCS8 stated as ‘>4.99’, this was so because the model could not be pushed beyond the base shear value corresponding to the ductility demand level of ‘4.99’ to reach the base shear value of RSEBCS8. Table 4.1 also contains the ductility demand values imposed by El-Centro 1940.

Table 4.1 Imposed ductility demands

		5 Story						10 Story			
Soil Class	Ductility Class	Imposed Ductility Demand				Soil Class	Ductility Class	Imposed Ductility Demand			
		EQX	RSEBCS8	RSEC8	El-Centro			EQX	RSEBCS8	RSEC8	El-Centro
Sub-Soil Class A	DCL	1.00	<1.16	<1.16	2.27	Sub-Soil Class A	DCL	1.00	1.71	1.71	3.53
	DCM	1.39	<1.16	<1.16	3.46		DCM	1.83	<0.99	<1.08	3.21
	DCH	2.59	1.71	1.71	5.25		DCH	>4.9	1.79	1.79	5.31
Sub-Soil Class B	DCL	1.00	<1.09	<1.09	1.79	Sub-Soil Class B	DCL	1.00	<.89	<.89	3.08
	DCM	1.52	1.35	1.35	2.29		DCM	1.92	1.30	1.30	2.54
	DCH	2.96	2.47	2.47	3.58		DCH	>3.81	3.03	3.03	3.66
Sub-Soil Class C	DCL	1.00	1.35	<1.02	1.62	Sub-Soil Class C	DCL	1.00	0.93	<0.9	2.39
	DCM	1.63	2.07	1.42	2.11		DCM	2.21	1.69	1.21	2.08
	DCH	>2.6	>2.6	2.52	3.46		DCH	>3.37	>3.37	2.74	3.05

The results used for preparation of the curves in Fig. 4.2 are made available in Table 4.2. The author feels that some of the values, especially the highlighted ones, are very difficult to attain practically. Further more, values in the red-filled cells are very unrealistic and can be considered as points of indication of a complete sway mechanism. For a better interpretation and further comparison of the values, reference can be made to Table 4.2.

4.2. Discussions

- Variation of ductility demand over the story levels was observed increasing for higher increments of base shear i.e. steps 4 and 5 and larger ductility demands were observed on the first story for the five story and on the middle stories (5^{th} and 6^{th}) for the ten story shear frames. The distribution is noted relatively consistent for the five story than for the ten story models. This observation was made from the curves developed considering only material non-linearity. But from the curves considering both non-linearities, larger ductility values were noted confined to the bottom story in many instances.
- The influence of undermining the design base shear, for possible reasons of considering lower values of building weight or design spectrum than the true values that should be taken, on ductility demand can clearly be observed from the curves developed. The imposed ductility demands were observed to increase as high as 10.24 for the ten story shear frame with 'DCH' and sub-soil class 'A' when the design base shear yield strength was considered reduced to 52.02% of the elastic design base shear yield strength.
- Considering the design base shear for 'DCL', i.e. load pattern 'EQX' which is considered as the base shear yield strength, the ductility demand imposed by it

Table 4.2 %base shear yield strength and ductility demand relationships

5 Story

Ductility Class	Sub-Soil Class											
	A				B				C			
	Push Mat		Push P-D		Push Mat		Push P-D		Push Mat		Push P-D	
	%yield strength	Ductility Demand	%yield strength	Ductility Demand	%yield strength	Ductility Demand	%yield strength	Ductility Demand	%yield strength	Ductility Demand	%yield strength	Ductility Demand
DCL	86.09	1.16	86.96	1.16	91.77	1.09	92.68	1.09	98.03	1.02	99.01	1.02
	62.28	2.08	63.63	2.02	67.92	1.87	69.24	1.83	70.56	1.81	71.11	1.81
	59.71	2.37	60.34	2.38	63.98	2.32	65.58	2.21	66.70	2.19	68.21	2.09
	56.54	3.28	60.24	2.40	62.51	2.69	64.27	2.49	65.24	2.57	67.36	2.21
	70.15	23.53	56.83	3.44	74.20	18.64	63.57	2.73	70.96	18.27	64.55	3.26
DCM	77.15	1.30	77.92	1.30	82.62	1.21	83.45	1.21	88.21	1.13	89.09	1.14
	60.29	2.03	61.01	2.02	63.00	2.00	63.68	2.00	65.42	1.99	68.59	1.83
	56.82	2.35	57.39	2.34	60.18	2.29	62.18	2.11	63.94	2.17	66.03	2.00
	55.82	2.60	57.28	2.36	58.03	2.84	60.16	2.37	63.55	2.27	64.60	2.19
	63.46	32.82	51.27	5.85	69.77	27.15	58.23	2.95	71.00	21.91	59.27	5.31
DCH	65.35	1.53	66.00	1.53	70.61	1.42	71.32	1.42	77.15	1.30	77.92	1.30
	47.82	2.74	48.60	2.70	52.26	2.50	53.16	2.46	60.29	2.03	61.01	2.02
	45.22	3.19	45.87	3.18	48.74	3.22	50.22	2.99	56.82	2.35	57.39	2.34
	44.87	3.29	45.69	3.22	45.94	4.39	49.31	3.24	55.82	2.60	57.28	2.36
	51.90	50.21	40.20	8.29	57.87	40.87	47.30	3.89	63.46	32.82	51.27	5.85

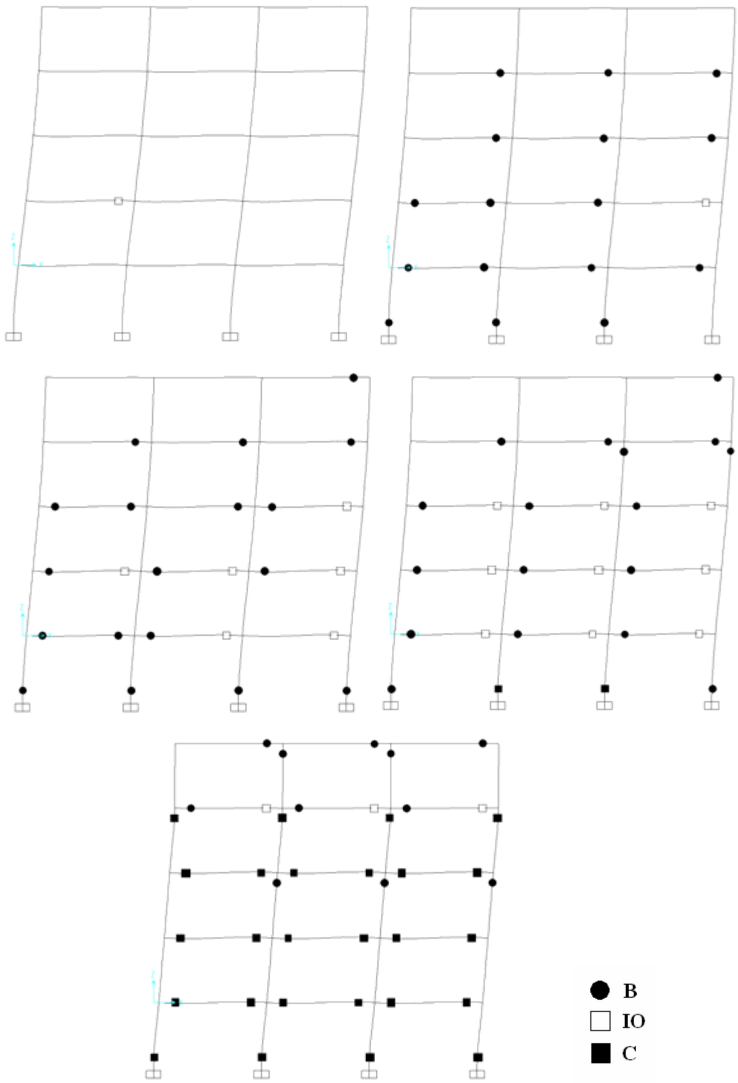
10 Story

Ductility Class	Sub-Soil Class											
	A				B				C			
	Push Mat		Push P-D		Push Mat		Push P-D		Push Mat		Push P-D	
	%yield strength	Ductility Demand	%yield strength	Ductility Demand	%yield strength	Ductility Demand	%yield strength	Ductility Demand	%yield strength	Ductility Demand	%yield strength	Ductility Demand
DCL	281.05	0.36	296.56	0.36	112.45	0.89	118.69	0.89	111.57	0.90	114.63	0.92
	89.18	2.33	100.55	2.28	80.06	1.74	86.03	1.69	105.84	0.97	111.22	0.95
	77.99	3.34	94.21	2.71	76.20	2.12	83.71	1.84	81.18	1.77	86.13	1.65
	73.52	4.37	91.19	3.06	73.52	3.00	82.37	2.00	78.31	2.10	84.17	1.82
	73.34	4.47	89.31	4.02	84.04	21.44	77.40	3.35	83.21	16.97	76.92	5.51
DCM	281.05	0.36	296.56	0.36	106.22	0.94	112.07	0.94	112.75	0.89	118.96	0.89
	89.18	2.33	100.55	2.28	75.78	1.83	81.52	1.78	79.44	1.74	84.93	1.72
	77.99	3.34	94.21	2.71	70.89	2.41	79.52	1.93	74.85	2.22	82.99	1.85
	73.52	4.37	91.19	3.06	68.26	4.20	76.38	2.41	73.66	2.47	81.26	2.07
	73.34	4.47	89.31	4.02	79.90	30.01	69.18	5.63	80.81	22.01	73.73	5.04
DCH	92.23	1.08	97.32	1.08	96.18	1.04	101.49	1.04	101.40	0.99	106.99	0.98
	64.76	2.21	70.47	2.11	68.19	2.09	74.18	1.98	72.98	1.90	78.42	1.85
	52.47	4.65	61.56	3.85	58.64	3.73	71.85	2.22	65.89	2.84	76.02	2.08
	51.78	4.90	59.17	4.90	58.37	3.81	66.25	3.45	63.73	3.37	71.68	2.90
	61.96	54.43	52.02	10.24	68.99	45.61	57.79	8.49	73.37	34.45	63.88	6.85

on the ‘DCM’ models varied between ‘1.83’ and ‘2.21’ for the ten story models. With the same observation on the ‘DCH’ models, the ductility demand exceeded ‘4.9’ for the ten story models and it ranged between ‘2.5’ and ‘2.96’ for the five story models.

- The ductility demands imposed by the response spectrum cases ‘RSEBCS8’ and ‘RSEC8’ are well with in the limits for the ductility classes. The exceptions to this are the demands imposed by ‘RSEBCS8’ on the five story model ‘DCH’ sub-soil class ‘C’, ‘>2.6’, and on the ten story model ‘DCH’ sub-soil class ‘C’, ‘>3.37’.
- The ductility demand imposed on the shear frames by the El-Centro ground motion ranged between ‘1.62’, ‘DCL’ Soil-class ‘C’, to ‘5.25’, ‘DCH’ Soil-class ‘A’, for the five story shear frames and from ‘2.08’, ‘DCM’ Soil-class ‘C’, to ‘5.31’, ‘DCH’ Soil-class ‘A’, for the ten story shear frames. The demand shows increase when going from the models designed for sub-soil class ‘C’ to ‘A’ for both the five and ten story shears frames.
- The ductility demand imposed on the shear frames by the El-Centro ground motion was found to be well with in the limits of ‘DCM’ and ‘DCH’. Exception to this are the shear frames designed for ‘DCL’. A ductility demand in excess of 2 was observed on the five story shear frame on sub-soil class ‘A’ and in all sub-soil conditions on the ten story shear frame. Taking into account that the peak ground acceleration of the ground motion, 0.319g, under consideration and the bed rock acceleration ratio, 0.1g, used for the design of the frames, this shows the disadvantage of designing for ‘DCL’ in high seismic cases.
- The influence of geometrical non-linearity was observed pronounced on the longer period, ten story models (T_1 = 2.5 sec), than on the shorter period, five story models (T_1 = 1.3 sec), which can be examined from the overlapping and separation of the respective %base shear yield strength Vs ductility demand curves developed with and without the inclusion of P-Δ effects.
- The increase on ductility demand due to geometrical non-linearity was observed to be as high as ‘113%’ (from ‘4.81’ to ‘10.24’, model ‘DCH’ sub-soil class ‘A’ with % base shear yield strength = ‘52.02 %’) for the longer period model while for the case of the shorter period model it was noted to peak only ‘24%’ (from ‘2.6’to ‘3.21’, model DCH sub-soil class ‘C’ with %base shear yield strength = ‘55.82 %’).
- Making reference to Table 4.2, the values highlighted with a red fill, are considered unrealistic and such exaggerated ductility demand values resulted as a consequence of formation of hinges at the bases of all the 1st story columns causing loss of stability - a typical hinge formation mechanism demonstrating

such phenomenon is presented on Figure 4.5. Such values were also observed to be associated with dramatic increases without significance lateral load increment. The grey filled values on the other hand, are considered difficult to meet on real practice.



Points B and C correspond to points on the Force-Deformation curve of Fig. 3.2 and the other performance level between B and C is (IO) - Immediate occupancy

Fig 4.5 Hinge Pattern (5 story model 'DCM' sub-soil class 'C')

- As can be observed from the relationship developed for the case study building, a ductility demand value of '9.74' was imposed on the building at about '78.33%' bases shear yield strength in the X-direction. This may be due to lack of proper detailing in addition to inadequate framing in this direction. But in the Y-direction at the same value of %base shear yield strength the ductility level was observed to be slightly above '4'. Even this level of ductility demand for a 'DCM' design is above the limiting value which is between ‘2’ and ‘4’ for a %base shear yield strength value of ‘75%’.

- The investigation revealed that in all the beams and columns of the case study building, the critical region, potential location of the plastic hinge formation, was not defined and the spacing of hoops with in this region does not meet the recommendations of the special provisions. Regarding the detailing for the columns - the minimum diameter of hoops, maximum distance between main bars, minimum cross sectional dimension, extension of the length of the critical region to an additional length of $0.5*l_{cr}$ in the lower two stories, provision of at least one intermediate bar on all the sides of the column were all failed in all the cross sections of the columns. Further more, the minimum amount of main bars was not meet on sections ‘C-C’ and ‘D-D’.
- The adjustments made on the case study building resulted in a significant decrease on the strength and ductility demand relationship especially on the X-direction. The highly exaggerated ductility demand observed at ‘78.33%’ base shear yield strength was observed to decrease to a value slightly above ‘4’ for the X-direction and for the Y-direction, it was determined to be below ‘4’. Such a considerably improved ductile behavior was attained with only limited amount of increase, almost no increase, on the overall cost of the structure but following the special provisions for structural elements recommended by EBCS 8, 1995. Fig. 4.3 shows a plot of %base shear yield strength against ductility demand of both the original and improved structure.
- Another important observation was made by considering the static pushover curves of both the original and modified structures. Fig. 4.4 shows these pushover curves. It can clearly be observed that there is quite an obvious improvement in the capacity of the building to resist a higher level of base shear. From the plot, a roof displacement of ‘76.1mm’ was reached at a base shear value of ‘781.57 kN’ by the original structure while for the modified structure; the roof displacement was recorded as ‘77.91mm’ at a base shear value of ‘1235.97 kN’ (58% increase as compared to the original structure). At these two particular points, the bases of all the ground floor columns of both structures were hinged.

5. Conclusions and Recommendations

- Considering the results shown on Table 4.1, the permitted reductions on the design base shear when designing for ductility classes ‘DCM’ and ‘DCH’ were found to impose ductility demands with in the ductility limits of ‘DCM’ and ‘DCH’ except for the ten story ‘DCH’ models.
- Following the conclusion made on the preceding point, no correction shall be applied on the design base shear values recommended by EBCS8, 1995 for the ductility class ‘DCM’ and ‘DCH’ for the five story shear frame and ductility class ‘DCM’ for the ten story shear frame in all sub-soil conditions. A further investigation should be carried out on the ten story shear frames designed for ‘DCH’. Taking into consideration this complicated relationship between these two parameters, caution shall be taken when designing buildings to ‘DCM’ and ‘DCH’ at least to fulfill detailing requirements recommended by the code.
- It was observed, from the relationship between strength and ductility developed for the five and ten story shear frames, that undermining the design base shear dramatically increases the ductility demand. Taking this into consideration, the author recommends that the design base shear must be calculated with a good level of accuracy to avoid undesirable story ductility demands.
- For the five story shear frames, material non-linearity was observed to have by far a significant effect on ductility demand as compared to geometrical non-linearity. While on the ten story shear frames, geometric non-linearity also has an important role to play. The author suggests that greater care shall be given for detailing of structures designed for ‘DCM’ and ‘DCH’, especially around potential locations of plastic hinge formations on structural members and inclusion of P-Δ effects shall not be avoided for longer period structures.
- EC8, 2004 restricts designing for ‘DCL’ only to low seismic cases. By carrying out a detail study on this issue under many ground motions on different soil conditions, the author suggests that this shall be adapted to EBCS 8, 1995 taking into consideration the preceding point.
- Comparing the special detailing provisions for beams and columns, EBCS 8, 1995 suggests a more strict provision on the definition of the critical region length, the maximum spacing of hoops with in the critical region and minimum width of cross section than EC8, 2004.

- Considering the improvement, lower imposed ductility demand and improved static pushover curve, observed on the case study building, the author strongly suggests that the special detailing provisions recommended in the codes shall be followed without any compromise.
- Finally the author suggests that such a study shall further be conducted considering different fundamental period of vibration, dumping ratio and ground motion on concrete and steel MDOF systems to address these basic seismic design parameters and enforce its implementation in the seismic design practice of Ethiopia.

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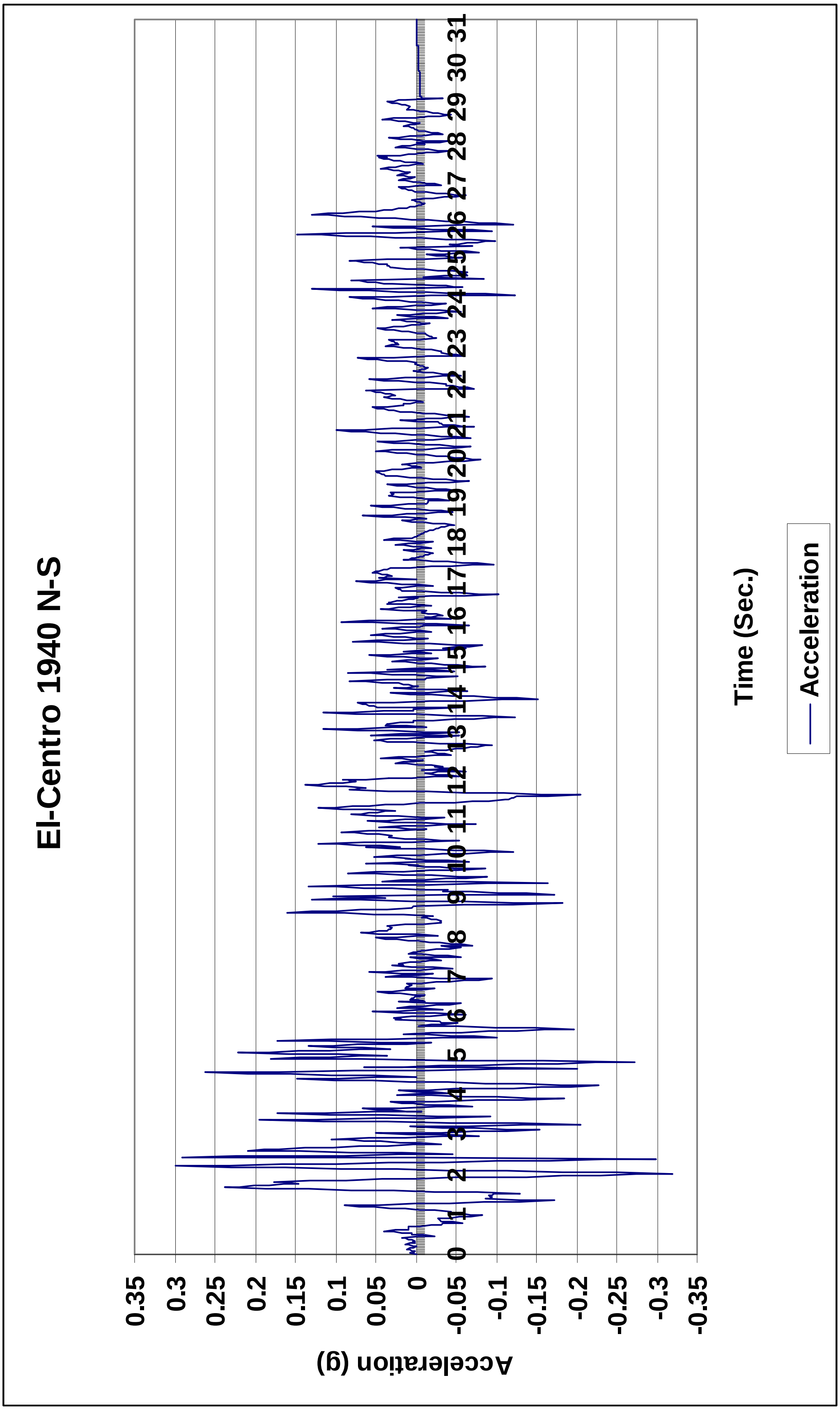
APPENDIX: El-Centro, 1940 Ground Motion



DECLARATION

I, the undersigned declare that this thesis is my work and that all sources of material used for this thesis have been duly acknowledged.

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DATE OF SUBMISSION: NOVEMBER 2009