

**ASSESSMENT OF SEISMIC VULNERABILITY OF
REINFORCED CONCRETE BUILDINGS:
A CASE STUDY OF SELECTED BUILDINGS CONSTRUCTED IN
ADDIS ABABA**

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Abstract

This thesis is conducted to evaluate seismic vulnerability and investigate nonlinear behaviors of existing reinforced concrete buildings constructed in Addis Ababa. The paper examines four structures using nonlinear dynamic analysis software (IDARC 2D ver. 7.0). Each of these structures will be analyzed for three peak ground accelerations of 0.05g, 0.07g and 0.10g.

The finding of this research is that two of the four buildings will collapse immediately when an earthquake (of intensity 0.05g, 0.07g and 0.10g) occurs. It is also found that the other two buildings resist the seismic loads that will occur during an earthquake and the story force deformation time history responses of these two buildings show stiffness degradation, strength decay and pinching behaviors of reinforced concrete structures.

It was concluded that some of the buildings deform beyond the limit of linearly elastic range and due to the high damage of the main structural elements, the buildings will no longer give service or a total collapse will occur after earthquake. It is recommended to evaluate seismic vulnerability of the existing buildings in Addis Ababa based on nonlinear analysis procedures.

Keywords: Seismic Vulnerability; Nonlinear Dynamic Analysis; Stiffness Degradation; Strength Decay; Pinching

1. Introduction

1.1. Background

EBCS-8 states, the purpose of the code is to ensure that, in the event of earthquakes, human lives are protected, damage is limited and structures important for civil protection remain operational. But the question in most structural engineers' mind is

- a) "Are the buildings constructed in Ethiopia sufficiently designed to withstand intense ground shaking?"
- b) "How do buildings actually perform in the event of an earthquake?"

Although buildings designed according to our code are expected to perform sufficiently well to prevent the loss of life under seismic loads equal to those they were designed for, how they will actually perform in the event of an earthquake is largely unknown. The actual case is that most buildings are expected to deform beyond the limit of linearly elastic behavior when subjected to strong ground shaking and the ensuing dynamic instability can cause excessive structural damage, and even overall collapse.

Seismic vulnerability is the expected degree of damage to a given structure at risk resulting from a given level of seismic hazard. Vulnerability assessment can be observed vulnerability or predicted vulnerability. Observed vulnerability is based on statistics of past earthquake damage. And predicted vulnerability refers to assessment of expected performance of buildings based on engineering computations and design specifications or, if no other method is available, on engineering judgment. Predicted vulnerability is more suitable for area of low and moderate seismicity. Since no major damaging earthquake has occurred in Ethiopia in recent times, vulnerability assessment from observed damage patterns are not available.

Nonlinear dynamic earthquake analysis is the most adequate and comprehensive analysis procedure to evaluate the nonlinear seismic response of structures. The nonlinear response of buildings to the highest level of accuracy can be assessed by making use of non-linear dynamic analysis procedure.

1.2. Objectives

The objective of this thesis work is to:

- a) Evaluate the damage extent of selected buildings constructed in Addis Ababa during an earthquake
- b) Investigate the nonlinear behavior of the case study buildings during the earthquake
- c) To introduce a program for the inelastic damage analysis of reinforced concrete structures.

1.3. Application of results

The results associated with this thesis work pinpoint the risk associated with analysis considering only the linear behavior of reinforced concrete. The results are a wakeup call for structural engineers because buildings need to be designed with the expectation that buildings will undergo some cracking, yielding and damage during an intense earthquake.

1.4. Delimitation and Limitations of the study

Delimitations

- a) Due to the scarcity of building data, only four structures are used to carry out the study
- b) The available damage analysis software, IDARC 2D, is not user friendly especially the input section.
- c) Only ribbed slab system building are considered.
- d) Structural detailing was not checked against the recommendations of EBCS-2 and EBCS-8.

Limitations

- a) Because of the limited amount of time and scope, the evaluation of the buildings is limited to damage analysis
- b) Only three of the case study buildings have architectural data.
- c) The owners of the buildings cannot be made public because of owner's interest.
- d) While performing this research site visit of the buildings is not made.

2. Literature Review

2.1. Seismic Risk

Risk is defined as the expected losses (of lives, persons, injured, property damaged, and economic activity disrupted) due to particular hazard for a given area and reference period. Base on mathematical calculations risk is the product of hazard and vulnerability (Marfai and Njagih, 2002).¹ (Dowrick, 2009) defines seismic risk as the probability that social or economic consequences of earthquakes will equal or exceed specified values at a site, at several sites, or in an area, during a specified exposure time. Risk statements are thus given in quantitative terms.² Dowrick further describes seismic risk as an outcome of seismic hazard as described by the relationship of the form

$$\text{Seismic Risk} = \text{Seismic Hazard} * \text{Vulnerability} * \text{Value}$$

- Where Vulnerability is the amount of damage, induced by a given degree of hazard, and expressed as a fraction of the value of the damaged item under consideration
- Value is the asset at risk that is threatened by the earthquake

Another statement by (Elghazouli, 2009) states that seismic risk is often considered as the convolution of seismic hazard, exposure and vulnerability. Exposure refers to the people, buildings, infrastructure, commercial and industrial facilities located in an area where earthquake effects may be felt; exposure is usually determined by planners and investors, although in some cases avoidance of major geo-hazards may lead to relocation of new infrastructure.³ Figure 2-1 shows a flow chart for the information flow and those involved in the earthquake risk reduction process.

2.2. Seismic Hazard

Seismic hazard is any physical phenomenon (ground shaking, ground failure) associated with an earthquake that may produce adverse effects on human activities. Seismic hazards

¹ M. A. Marfai And J.K. Njagih, 2002, *Vulnerability Analysis And Risk Assessment For Seismic And Flood Hazard In Turialba City, Costa Rica*, p.3

² D. Dowrick, 2009, *Earthquake Resistance Design And Risk Reduction*, pp.1-2

³ A. Elghazouli, 2009, *Seismic Design of Buildings to Eurocode 8*, pp.6

are the potentially damaging effects of earthquake at a particular location, which may include surface rupture, tsunami run-up, liquefaction and landslides, although the most important cause of damage on a global scale is earthquake induced ground shaking (Elghazouli, 2009).¹

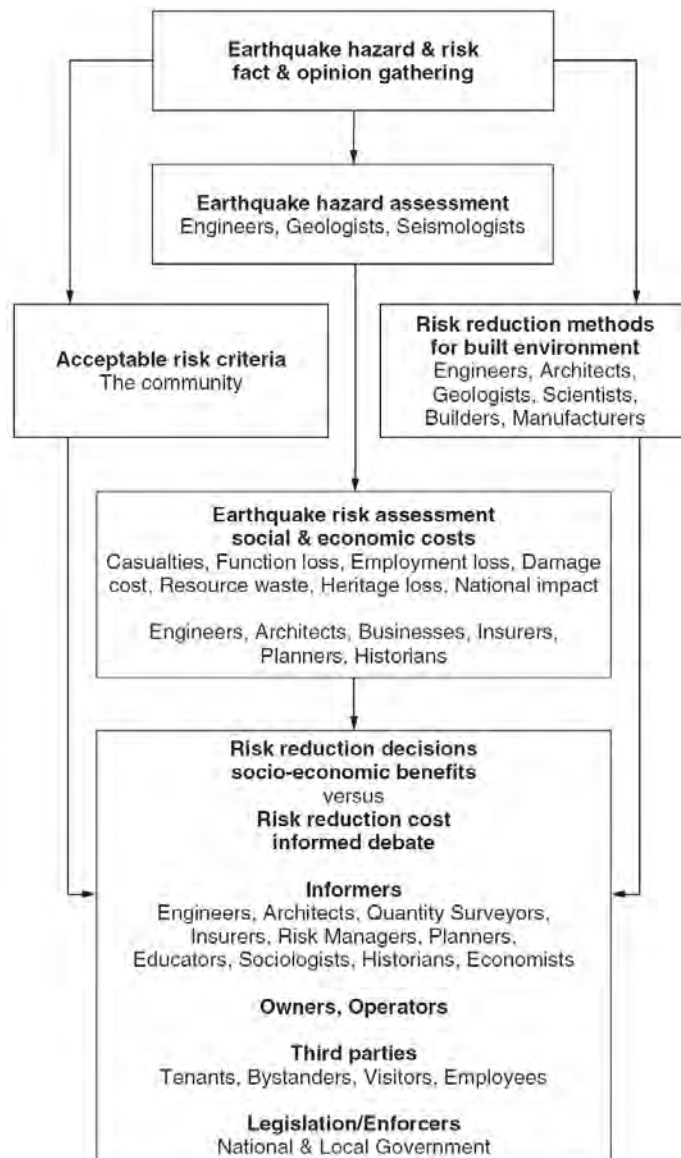


Figure 2-1 Information flow and those involved in the earthquake risk reduction process²

Figure 2.2 shows direct and indirect effects of earthquake described by (Elnashai and Sarno, 2008). Hazards may be either purely descriptive terms or quantitatively evaluated,

¹ A. Elghazouli, 2009, *Seismic Design of Buildings to Eurocode 8*, pp.6

² D. Dowrick, 2009, *Earthquake Resistance Design And Risk Reduction*, Figure 1.1

depending on the needs of situation (Dowrick, 2009).¹ From an earthquake engineer's perspective, hazard can be quantified but not reduced (Elnashai and Sarno, 2008).²

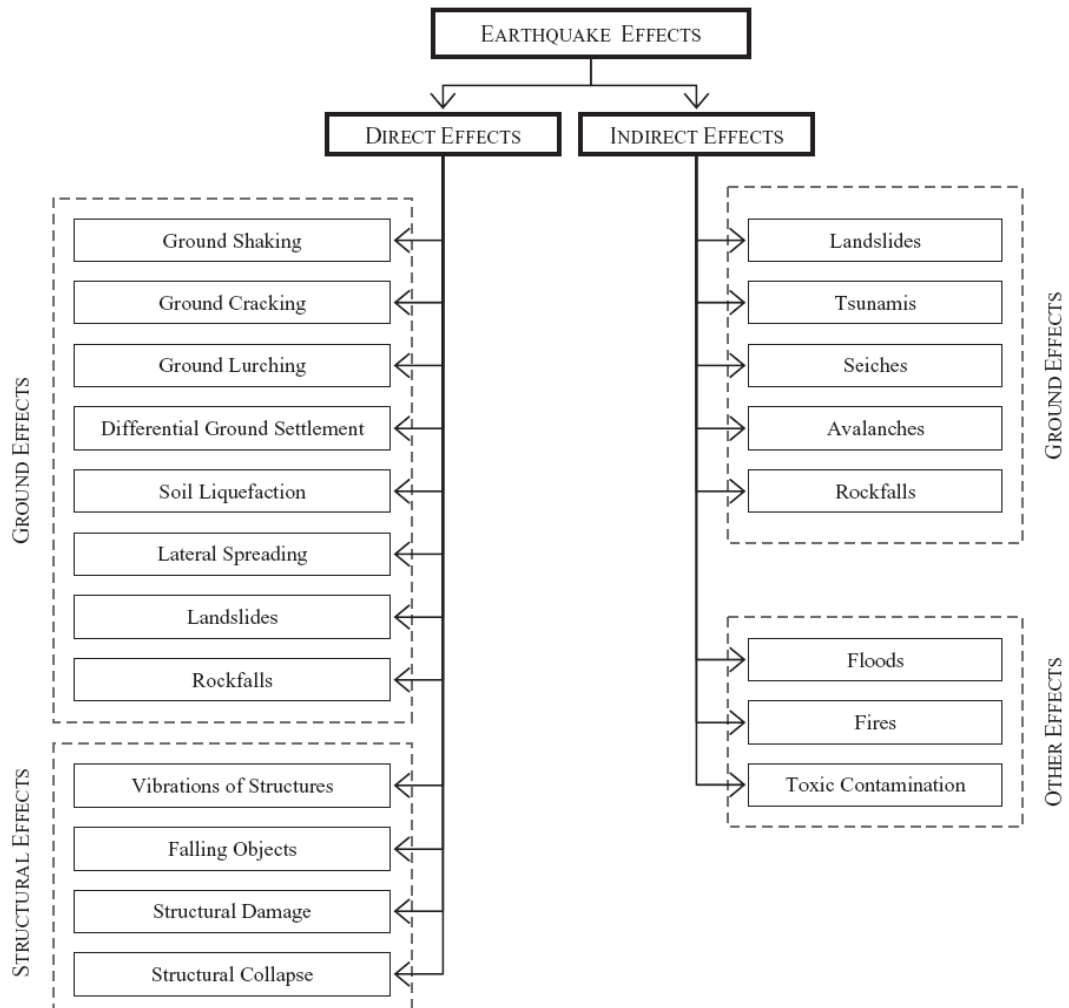


Figure 2-2 Direct and indirect earthquake effects³

Seismic hazard assessments are attempts to forecast the likely future seismic activity rates and strengths, based on knowledge of the past and present, and significant uncertainties arise partly because the processes involved are not fully understood and partly because relevant data are generally scarce and variable in quality (Dowrick, 2009).⁴ The process of assessing seismic hazards starts with the identification of potential seismic sources, which are related to tectonic settings of a certain region. In assessing seismic sources, a plot map

¹ D. Dowrick, 2009, *Earthquake Resistance Design And Risk Reduction*, pp.1

² A. Elnashai And L. Sarno, 2008, *Fundamentals of Earthquake Engineering*, p.32

³ A. Elnashai And L. Sarno, 2008, *Fundamentals of Earthquake Engineering*, Figure 1.22

⁴ D. Dowrick, 2009, *Earthquake Resistance Design And Risk Reduction*, pp.3

showing seismic events, local faults and tectonic events are the main tools (Marfai and Njagih, 2002).¹ According to (Dowrick, 2009), for design or risk assessment purposes the assessment of seismic hazard consists of the following basic steps:²

- a. Definition of the nature and locations of earthquake sources;
- b. Magnitude–frequency relationships for the sources;
- c. Attenuation of ground motion with distance from source;
- d. Determination of ground motions at the site having the required probability of exceedance.

2.3. Seismic Vulnerability

As stated by (Marfai and Njagih, 2002), vulnerability, which is a key topic in risk analysis, is defined as the degree of loss to a given element or set of elements resulting from the occurrence of a natural disaster.³ The same author states, vulnerability to earthquakes in urban areas is largely related to building design and quality since it is these houses, which collapse and injure people. The nature of the construction and the materials used will greatly influence the kind of injuries inflicted.⁴

According to (Elghazouli, 2009), seismic vulnerability is the susceptibility of structures to earthquake effects and is generally defined by the expected degree of damage that would result under different levels of seismic demand; this is the component of the risk equation that can be controlled by engineering design.⁵

Seismic vulnerability of structures varies as a function of construction materials and earthquake action-resistance system employed. However, causes of fatalities and extent of damage depend to a great extent on the type of constructions and the density of population in the area (Elnashai and Sarno, 2008).⁶ Earthquake can cause devastating effects in terms

¹ M.A. Marfai And J. K. Njagih, 2002, *Vulnerability Analysis And Risk Assessment For Seismic And Flood Hazard In Turialba City, Costa Rica*, pp.8

² D. Dowrick, 2009, *Earthquake Resistance Design And Risk Reduction*, pp.3

³ M. A. Marfai And J. K. Njagih, 2002, *Vulnerability Analysis And Risk Assessment For Seismic And Flood Hazard In Turialba City, Costa Rica*, pp.12

⁴ M. A. Marfai And J. K. Njagih, 2002, *Vulnerability Analysis And Risk Assessment For Seismic And Flood Hazard In Turialba City, Costa Rica*, pp.13

⁵ A. Elghazouli, 2009, *Seismic Design of Buildings to Eurocode 8*, pp.6

⁶ A. Elnashai And L. Sarno, 2008, *Fundamentals of Earthquake Engineering*, pp.34

of loss of life and livelihood. The destructive potential of earthquake depends on many factors such as:

- a) Size of an event
- b) Focal depth and Epi-central distance
- c) Topographical conditions
- d) Local geology

Vulnerability can be both evaluated and reduced, by measures of retrofitting. It can also be reduced by other means, such as long-term land use management and education (Elnashai and Sarno, 2008).¹

2.4. *Limit state of Performance*

Table 2.1 Limit state of performance according to EC-8² and ATC-40³

LIMIT STATE	EC-8	ATC-40
Limit State of near Collapse/ Structural Stability Performance Level	Heavily damaged with small residual Strength and stiffness	Substantial damage with significant degradation in stiffness and strength
	vertical elements are capable of sustaining vertical loads	Vertical elements continue to carry their gravity demands
	non structural components have collapsed	Significant risk due to falling hazards may exist
	permanent drift are present	
	structure would not survive another earthquake	building structural system is on the verge of partial or total collapse
Limit State of Significant Damage/ Life Safety Performance Level	Significantly damaged with some residual strength and stiffness	significant damage but some margin against total or partial collapse remains
	vertical elements are capable of sustaining vertical loads	Level of damage is lower than that for the structural Stability level
	non structural components are damaged	Major non structural components have not become dislodged and fallen
	moderate permanent drift are present	
	structure is likely to be uneconomic to repair	Extensive uneconomical structural repair will likely be necessary
Limit State of Damage Limitations/ Immediate Occupancy Performance Level	Slightly damaged with retained structural elements' strength and stiffness	Very limited structural Damage with retained nearly all pre earthquake capacity
	non structural components may show cracking that is repairable economically	Risk of life-threatening injury from structural failure is negligible
	No permanent drifts are present	
	structure does not need any repair measures	

¹ A. Elnashai And L. Sarno, 2008, *Fundamentals of Earthquake Engineering*, pp.33

² Eurocode 8: *Design of structures for earthquake resistance Part 3: Strengthening and repair of buildings*, pp.12

³ *Seismic evaluation and retrofit of concrete buildings, ATC-40*, pp.3-4

2.5. Structural Assessment

EC-8 defines assessment as a quantitative procedure by which it is checked whether an existing undamaged or damaged building can resist the design seismic load combination as specified in the code. It further states that, assessment is made for individual buildings, in order to decide about the need for structural intervention and about the strengthening or repair measures to be implemented.¹

2.5.1. Information for Structural Assessment

2.5.1.1. General information and history

In assessing the earthquake resistance of existing structures, taking also into account the effects of actions in other design situations, the input data shall be collected from available records, relevant information, field investigations and, in most cases, from in-situ and/or laboratory measurements and tests (EC-8).²

2.5.1.2. Required input data

The seismic evaluation of existing concrete buildings depends on data collection as a factual basis. The data collection process includes acquisition of available documents, field observations, field investigations, materials testing, and documentation. While the extent of the data acquisition process will vary from building to building and will depend on the availability of drawings and the level of evaluation being performed, i.e. preliminary evaluation, detailed evaluation and analysis, or preparation of final retrofit construction documents (ATC-40)³. It further states, accurate building information is necessary, in any event, in the following areas:

- a) Building geometry, configuration, and mass (including structural, architectural and mechanical systems)
- b) Elements of the seismic load path, including frames, walls, diaphragms, foundations, and connections

¹ Eurocode 8: Design of structures for earthquake resistance Part 3: Strengthening and repair of buildings, pp.22

² Eurocode 8: Design of structures for earthquake resistance Part 3: Strengthening and repair of buildings, pp.14

³ Seismic evaluation and retrofit of concrete buildings, ATC-40, Section 5-12

- c) Configuration and layout of structural members, including size of members, size of reinforcing, tie spacing, splice locations, and concrete cover
- d) Properties of the materials used in the structural system, such as concrete and steel reinforcing
- e) Anchorage of nonstructural elements

Tables 2.1 through 2.4 summarize the data collection process as it relates to availability of drawings and level of evaluation (ATC-40).

Table 2.2 Information Required For Preliminary Seismic Evaluation When Original Construction drawings Are Available ¹

Item	Required		Comment
	Yes	No	
Structural calculations		X	Helpful but not essential
Site seismicity, geotechnical report		X	Helpful but updated report should be done
Foundation report		X	Helpful but not essential
Prior seismic assessment reports		X	Helpful but not essential
Condition survey of building	X		
Alteration and as built assessment	X		
Walk through dimensioning		X	Unless required by undocumented alterations
Nonstructural walk through	X		Identify falling hazards, weight
Core testing		X	Unless concrete appears substandard
Rebound hammer testing		X	Unless concrete appears substandard
Aggregate Testing		X	
Reinforcement testing		X	
Reinf. Location verification		X	Unless Insufficient Info. On drawings
Nonstructural exploration		X	

Table 2.3 Information Required For Preliminary Seismic Evaluation When Original Construction Drawings are Not Available ²

Item	Required		Comment
	Yes	No	
Structural calculations		X	Could minimize scope of site work
Site seismicity, geotechnical report		X	Could minimize scope of site work
Foundation report		X	Could minimize scope of site work
Prior seismic assessment reports		X	Could minimize scope of site work
Condition survey of building	X		
Alteration and as built assessment	X		
Walk through dimensioning	X		Sufficient to define primary elements
Nonstructural walk through	X		Identify falling hazards, weight
Core testing	X		Minimum 2 per floor, 8 per building
Rebound hammer testing		X	Could be helpful, especially if concrete appears Substandard
Aggregate Testing	X		Several cores
Reinforcement testing		X	
Reinf. Location verification		X	Could be helpful
Nonstructural exploration		X	

¹ Seismic evaluation and retrofit of concrete buildings, ATC-40, Table 5.1

² Seismic evaluation and retrofit of concrete buildings, ATC-40, Table 5.2

Table 2.4 Information Required For Detailed Seismic Evaluation When Original Construction Drawings are Available ¹

Item	Required		Comment
	Yes	No	
Structural calculations		X	Could be helpful
Site seismicity, geotechnical report		X	Helpful but not essential
Foundation report		X	Helpful but not essential
Prior seismic assessment reports		X	Helpful but not essential
Condition survey of building	X		
Alteration and as built assessment	X		
Walk through dimensioning		X	Spot checking is appropriate
Nonstructural walk through	X		Identify falling hazards, weight
Core testing	X		Minimum 2 per floor, 8 per building
Rebound hammer testing	X		Minimum 8 per floor, 16 per building
Aggregate Testing	X		Each core
Reinforcement testing		X	Optional
Reinf. Location verification	X		Pachometer@ 10% of critical locations, visual @ 2 locations
Nonstructural exploration	X		Verify anchorage and bracing conditions for components sensitive to building performance

Table 2.5 Information Required For Detailed Seismic Evaluation When Original Construction Drawings are Not Available ²

Item	Required		Comment
	Yes	No	
Structural calculations		X	Could be helpful
Site seismicity, geotechnical report		X	Helpful but not essential
Foundation report		X	Helpful but not essential
Prior seismic assessment reports		X	Helpful but not essential
Condition survey of building	X		
Alteration and as built assessment	X		
Walk through dimensioning	X		Must be done very thoroughly, particularly if structure will be retrofitted
Nonstructural walk through	X		Identify falling hazards, weight
Core testing	X		Minimum 2 per floor, 8 per building
Rebound hammer testing	X		Minimum 8 per floor, 16 per building
Aggregate Testing	X		Each core
Reinforcement testing	X		2 per type
Reinf. Location verification	X		Pachometer for all critical elements, visual on 25%
Nonstructural exploration	X		Verify anchorage and bracing conditions for components sensitive to building performance

Similarly, EC-8 recommends the following inputs as requirement for structural assessment.

- a) Identification of the structural system and of its compliance with the regularity criteria in 4.2.3 of EN 1998-1. The information should be collected either from on site investigation or from original design drawings, if available. In this latter case,

¹ Seismic evaluation and retrofit of concrete buildings, ATC-40, Table 5.3

² Seismic evaluation and retrofit of concrete buildings, ATC-40, Table 5.4

information on possible structural changes since construction should also be collected.

- b) Identification of the type of building foundations.
- c) Identification of the ground conditions
- d) Information about the overall dimensions and cross-sectional properties of the building elements and the mechanical properties and condition of constituent materials.
- e) Information about identifiable material defects and inadequate detailing.
- f) Information on the seismic design criteria used for the initial design, including the value of the force reduction factor, if applicable.
- g) Description of the present and/or the planned use of the building (with identification of its importance category).
- h) Re-assessment of variable loads considering the use of the building.
- i) Information about the type and extent of previous and present structural damages, if any, including earlier repair measures.¹

2.6. Nonlinear Dynamic Analysis

Nonlinear dynamic earthquake analysis is the most adequate and comprehensive analysis procedure to evaluate the nonlinear seismic response of structures (Sam Lee, 2008).² Another statement by (Paolo Bazzurro, 2009) states that nonlinear dynamic earthquake analysis is today the current state-of-the-art methodology for predicting building response to earthquake ground motion.³

According to (Wilson, 2002) the major advantage of using the forces obtained from a dynamic analysis as the basis for a structural design is that the vertical distribution of forces may be significantly different from the forces obtained from an equivalent static load analysis. Consequently, the use of dynamic analysis will produce structural designs that are more earthquake resistant than structures designed using static loads.⁴

¹ Eurocode 8: Design of structures for earthquake resistance Part 3: Strengthening and repair of buildings, pp.14-15

² S. Lee, 2008, Nonlinear Dynamic Earthquake Analysis of Skyscrapers by ABAQUS, pp.1

³ P. Bazzurro, 2009, Nonlinear Dynamic Analysis—a Step Advance in Assessing the Vulnerability of Buildings to Earthquake Ground Motion, pp.4

⁴ E. L. Wilson, 2002, Three-Dimensional Static and Dynamic Analysis of Structures, pp.17-1

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 behavior of the structure is assumed to be linear elastic. As nonlinear behavior occurs, the incremental stiffness is modified for the next time or load increment. Hence, the response of the nonlinear system is approximated by the response of a sequential series of linear systems having varying stiffnesses (Anderson, 2000).¹

2.7. Modelling for Nonlinear Dynamic Analysis

Nonlinear time-history analyses are a very powerful tool, provided they are supported by proper approximations and modelling. The analysis is inherently complex and may be very time consuming (Pecker, 2007).² Many engineers use large values for element properties when modeling rigid parts of structures. This can cause large errors in the results for static and dynamic analysis problems. In the case of nonlinear analysis the practice of using unrealistically large numbers can cause slow convergence and result in long computer execution times (Wilson, 2002).³

Mathematical modeling of a structure, for the purpose of structural analysis, can be done in a variety of different ways depending on the method of analysis adopted. In a general sense, modeling involves the representation of a structure by elements to which physical and material characteristics are assigned and the appropriate loading is applied or transmitted. There are three stages of modeling that an analyst has to go through prior to performing a dynamic analysis: (Saatcioglu and Humar, 2002)⁴

- a) structural modeling
- b) member modeling and
- c) hysteretic modeling (for nonlinear analysis)

¹ J. C. Anderson, 2000 *Seismic Design Handbook, Dynamic Response of Structures*. pp.231

² A. Pecker, 2007 *Advanced Earthquake Engineering Analysis*, pp.64

³ E. L. Wilson, 2002, *Three-Dimensional Static and Dynamic Analysis of Structures*, pp.7-3

⁴ M. Saatcioglu and J. Humar (2002), *Dynamic analysis of buildings for earthquake resistant design. Proposed Earthquake Design Requirements of the NBCC*, pp 344

2.7.1. Structural modeling

Structural models are idealizations of the prototype and are intended to simulate the response characteristics of systems. Three levels of modelling (see figure 2-3) are generally used for earthquake response analysis, substitution model, stick model and detailed model (Elnashai and Sarno, 2008).¹

Substitute models are idealized as an equivalent single degree of freedom system or 'substitute system'. Four parameters are needed to define the substitute system: effective mass, effective height, effective stiffness and effective damping. The height defines the location of the equivalent or effective mass of the substitute system.² Stick models consist of multi degree of freedom systems in which each element idealizes a number of members of the prototype structure. In multi - storey building frames, each storey is modelled by a single line of finite elements representing the deformational characteristics of all columns and their interaction with beams. For three - dimensional models, the stick element relates the shear forces along two horizontal orthogonal directions and the storey torque to the corresponding inter - storey translations and rotations, respectively.

Detailed models include general finite elements idealizations in which structures are discretized into a large number of elements with section analysis or spatial elements in 2D or 3D. Such a modelling approach allows representation of details of the geometry of the members, and enables the description of the history of stresses and strains at fibers along the length or across the section dimensions. In the detailed modelling approach, beams and columns of frames are represented by flexural elements, braces by truss elements, and shear and core walls by 2D elements, such as plates and shells (Elnashai and Sarno, 2008).³

(Saatcioglu and Humar, 2002) illustrate the modeling of common forms of buildings structures with line elements (see Figure 2-4). In frame and frame-wall interactive systems the vertical elements such as columns and flexure-dominant structural walls are

¹ A. Elnashai And L. Sarno, 2008, *Fundamentals of Earthquake Engineering*, pp.191

² A. Elnashai And L. Sarno, 2008, *Fundamentals of Earthquake Engineering*, pp.191

³ A. Elnashai And L. Sarno, 2008, *Fundamentals of Earthquake Engineering*, pp.192

represented by vertical line elements, and the horizontal elements such as beams, and slab diaphragms are represented by horizontal line elements as shown in figure 2-4a and figure 2-4e. Sometimes shear panels, like shear walls and in-fill panels, as well as bracing elements are modeled by diagonal struts and ties as shown in figure 2-4b, 2-4c and 2-4d.¹

2.7.2. Member Modeling

Force-deformation characteristics of individual members, essential for structural analysis are specified through member models. In a linear dynamic analysis, members are modeled by elastic line elements. In a nonlinear analysis, each member is idealized as an elastic line element with inelastic springs at the ends. The springs account for potential plastic hinges at member ends (See Figure 2-5). This model was developed by (Giberson, 1967) and it is known as the “single-component model”. The main disadvantage of the single-component model is the assumption that fixes the location of the point of contra flexure for the purpose of inelastic deformation computations. The single-component model is therefore regarded as an approximate and yet reasonably accurate member model (Saatcioglu and Humar, 2002).²

The other inelastic model (See Figure 2-6) proposed by (Clough and et.al, 1966) is known as the “two component model” (Reinhorn and et.al, 2009). The elastic member accounted for the strain hardening characteristics of the reinforcing steel, while the elastic perfectly plastic member for the yielding of the reinforcement (Reinhorn and et.al, 2009)³. This dual-component model, however, is applicable to structural members that exhibit elastoplastic behavior without any stiffness degradation (Saatcioglu and Humar, 2002)⁴.

¹ M. Saatcioglu and J. Humar (2002), *Dynamic analysis of buildings for earthquake resistant design. Proposed Earthquake Design Requirements of the NBCC*, pp 344

² M. Saatcioglu and J. Humar (2002), *Dynamic analysis of buildings for earthquake resistant design. Proposed Earthquake Design Requirements of the NBCC*, pp 346

³ A.M. Reinhorn et.al, 2009, IDARC2D Version 7.0:A Program for the Inelastic Damage Analysis of Structures, pp.12

⁴ M. Saatcioglu and J. Humar (2002), *Dynamic analysis of buildings for earthquake resistant design. Proposed Earthquake Design Requirements of the NBCC*, pp 346

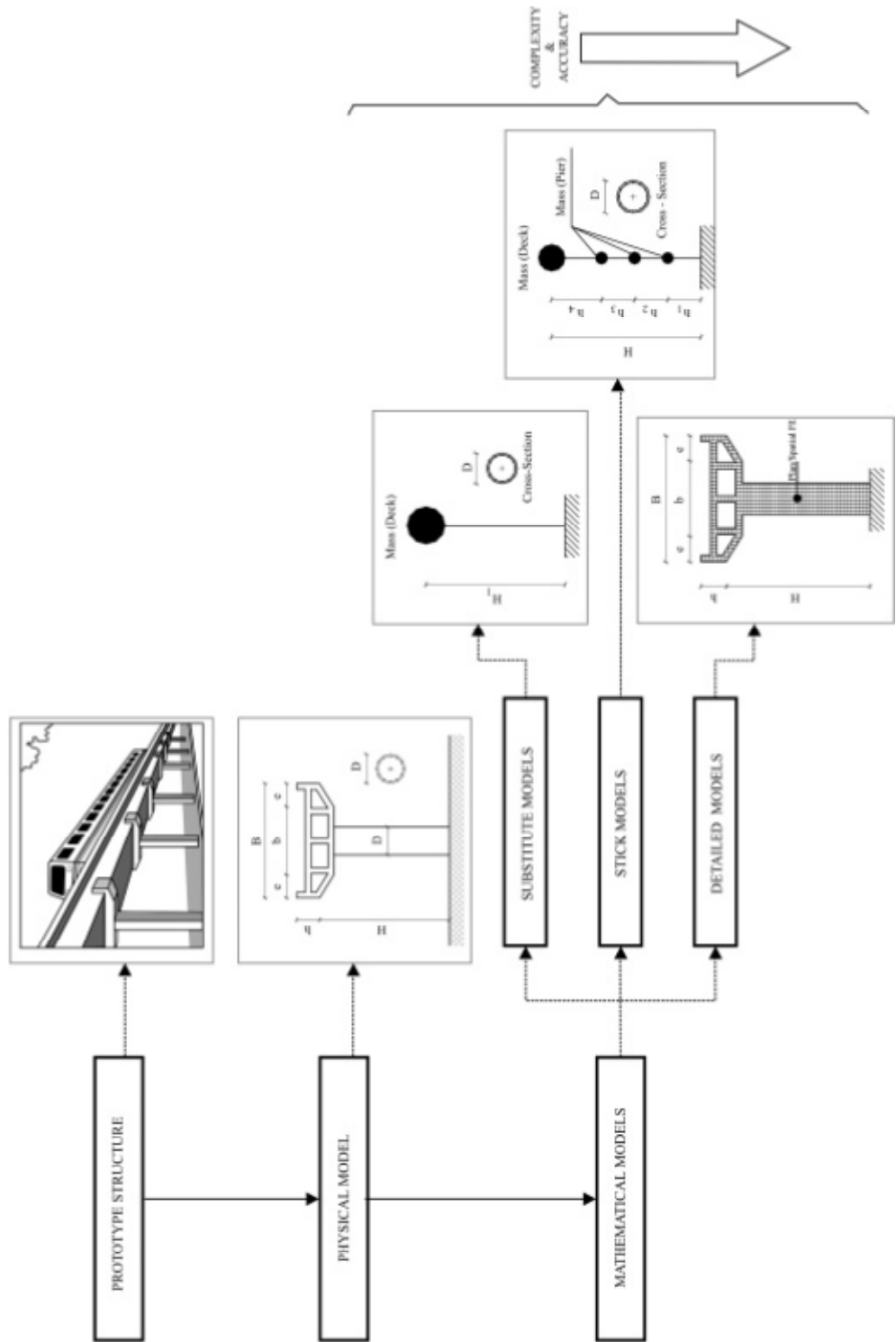


Figure 2-3 Levels of structural modelling for earthquake response analysis¹

¹ A. Elnashai And L. Sarno, 2008, *Fundamentals of Earthquake Engineering*, Figure 4.3

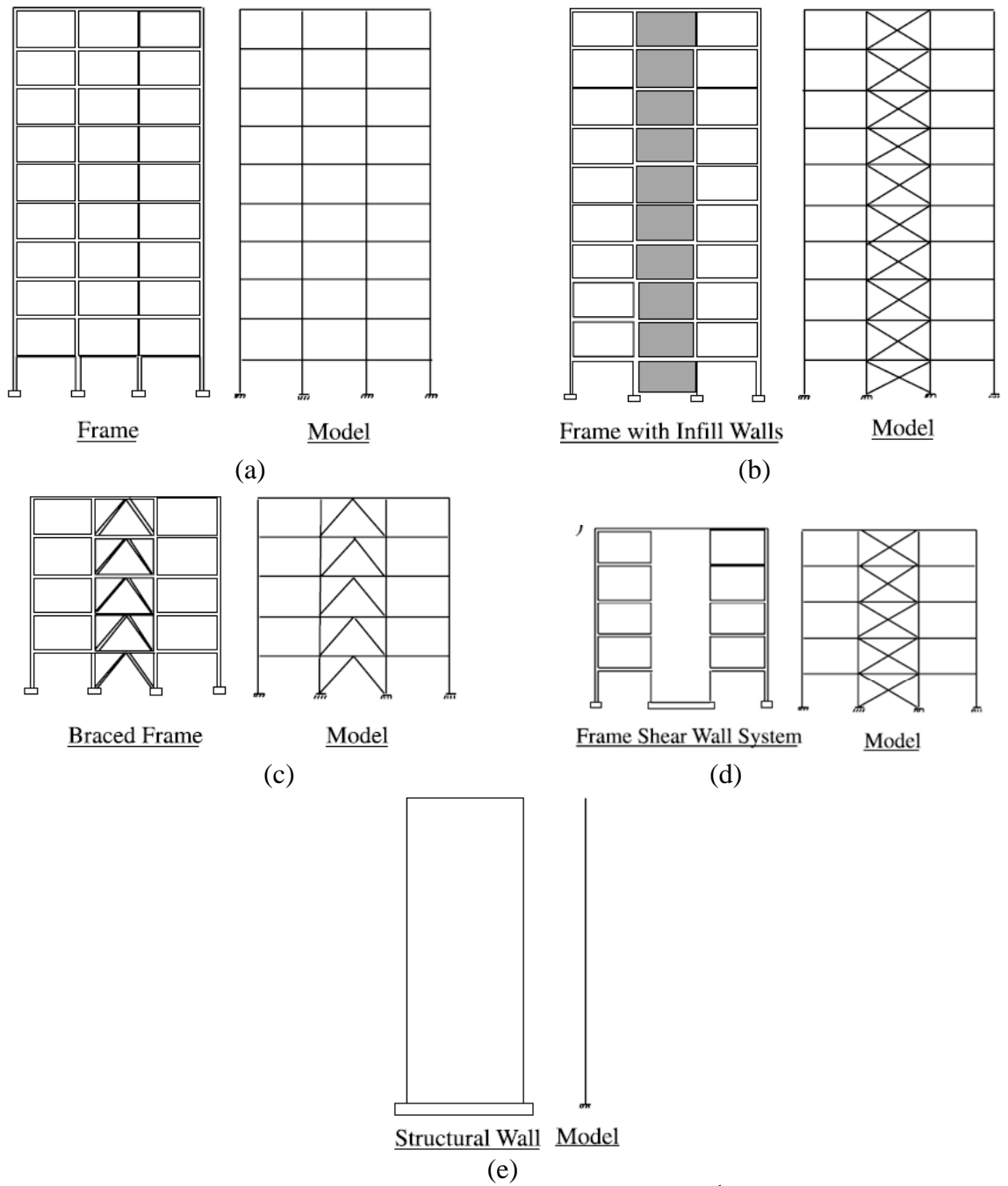


Figure 2-4 Structural Model Examples¹

¹ M. Saatcioglu and J. Humar(2002), *Dynamic analysis of buildings for earthquake resistant design. Proposed Earthquake Design Requirements of the NBCC, Figure 7*

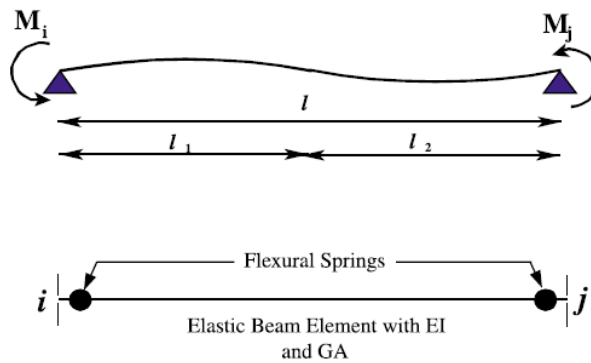


Figure 2-5 Single-component model¹

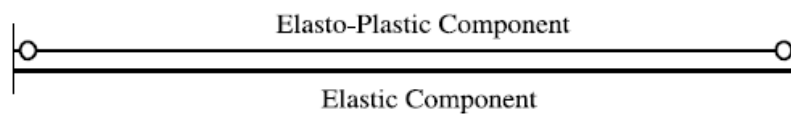


Figure 2-6 Dual-component model²

2.7.3. Hysteretic Modeling

Modeling the hysteretic behavior of structural elements is one of the core aspects of a nonlinear structural analysis program (Reinhorn and et.al, 2009)³. Dynamic inelastic response history analysis requires realistic conceptual models that can simulate strength, stiffness, and energy-dissipation characteristics of members (Saatcioglu and Humar, 2002)⁴. Hysteretic models can be broadly classified into two types – Polygonal Hysteretic models (PHM) and Smooth Hysteretic Models (SHM) (Sivaselvan and Reinhorn, 1999).⁵ The PHM refers to models based on piecewise linear behavior. Such models are most often motivated by actual behavioral stages of an element or structure, such as initial or elastic behavior, cracking, yielding, stiffness and strength degrading stages, crack and gap closures etc. On the other hand, the SHM refers to models with continuous change of

¹ M. Saatcioglu and J. Humar(2002), *Dynamic analysis of buildings for earthquake resistant design. Proposed Earthquake Design Requirements of the NBCC, Figure 8*

² M. Saatcioglu and J. Humar(2002), *Dynamic analysis of buildings for earthquake resistant design. Proposed Earthquake Design Requirements of the NBCC, Figure 9*

³ A.M. Reinhorn et.al, 2009, *IDARC2D Version 7.0:A Program for the Inelastic Damage Analysis of Structures,pp.61*

⁴ M. Saatcioglu and J. Humar, 2000, *Dynamic analysis of buildings for earthquake resistant design. Proposed Earthquake Design Requirements of the NBCC, pp 350*

⁵ M..V Sivaselvan and A.M. Reinhorn,(1999), *Hysteretic Models for Cyclic Behavior of Deteriorating Inelastic Structures , pp 2*

stiffness due to yielding, but sharp changes due to unloading and deteriorating behavior (Reinhorn and et.al, 2009).¹

2.7.3.1. Stiffness Degradation

According to (Saatcioglu and Humar, 2002), most hysteretic models consist of a *primary curve* (backbone curve), which can be computed analytically by well established procedures of mechanics, and a set of empirical rules that define the branches of loading, unloading, and reloading under reversed cyclic loading. The primary curve provides an envelope of hysteresis loops of force–deformation relationships. Therefore, it provides a convenient means of defining the strength boundary for modeling purposes. Hysteretic rules within the strength boundary (primary curve) simulate the actual member behavior, often as observed during tests.²

Structural steel elements exhibit elastoplastic behavior, with unloading and reloading branches of hysteresis loops parallel to the initial elastic branch. The post-yield slope of the primary curve may reflect steel strain hardening with a post-yield rigidity approximately equal to 0.5–5% of the elastic rigidity (See Figure 2-7).³

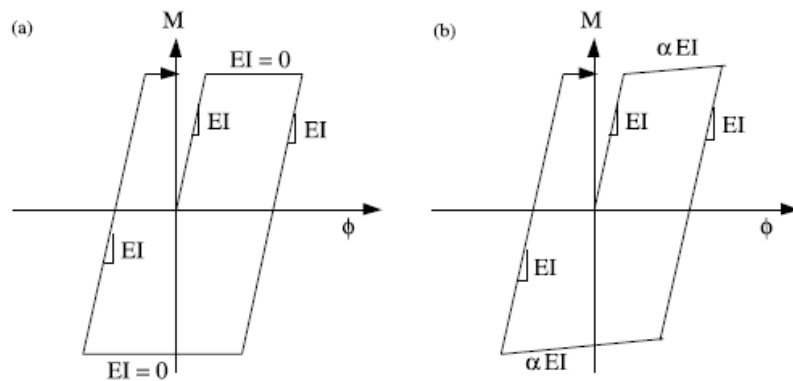


Figure 2-7 Elastoplastic hysteretic models for steel structures:
(a) Zero post-yield rigidity; (b) with strain hardening

¹A.M. Reinhorn et.al, 2009, IDARC2D Version 7.0:A Program for the Inelastic Damage Analysis of Structures, pp.61

² M. Saatcioglu and J. Humar, 2000, *Dynamic analysis of buildings for earthquake resistant design. Proposed Earthquake Design Requirements of the NBCC*, pp 350

³ M. Saatcioglu and J. Humar, 2000, *Dynamic analysis of buildings for earthquake resistant design. Proposed Earthquake Design Requirements of the NBCC*, pp 351

Unlike structural steel elements, reinforced concrete develops stiffness degradation under inelastic deformation reversals. The degradation of stiffness takes place during reloading and unloading as evidenced by reductions in the slopes of corresponding force–deformation hysteresis loops. The degradation in stiffness increases at a varying rate, usually as a function of the number of cycles and the level of peak deformations attained. A simple hysteretic model (See Figure 2-8) was developed by Clough (1966), incorporating stiffness degradation into a perfectly elastoplastic response (Saatcioglu and Humar, 2002)¹.

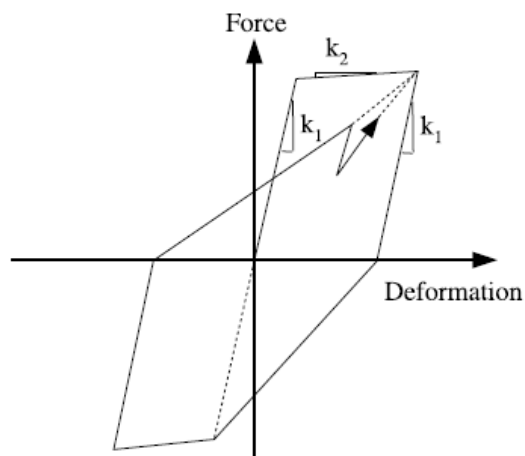


Figure 2-8 Stiffness degrading model by Clough (1966).

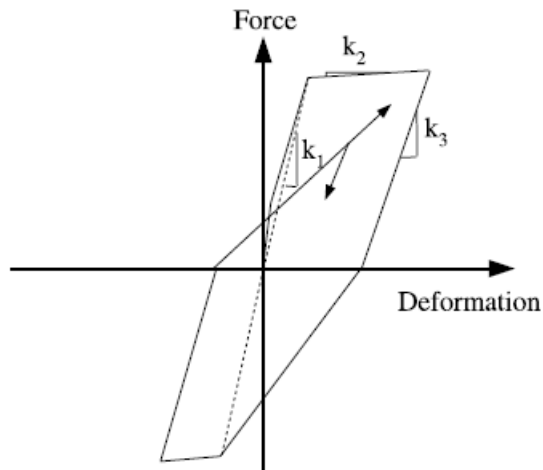


Figure 2-9 Stiffness degrading model by Takeda et al. (1970).

An improved degrading stiffness model (See Figure 2-9) was developed by Takeda et al. (1970) with a trilinear primary curve (Saatcioglu and Humar, 2002). The reloading points

¹ M. Saatcioglu and J. Humar, 2000, *Dynamic analysis of buildings for earthquake resistant design. Proposed Earthquake Design Requirements of the NBCC*, pp 352

in the model are aimed at the response point at previous maximum deformations. Unloading slopes are reduced as a function of the previous maximum deformation; hence the stiffness degradation is introduced in a more refined manner as compared to the Clough model.¹

2.7.3.2. Strength Decay

An important feature of hysteretic response is *strength decay*. Structural members exhibit progressive loss of strength under relatively high levels of inelastic deformation cycles. Figure 2-10 illustrates a strength decay obtained in a concrete column under reversed cyclic loading by (Ozcebe and Saatcioglu, 1989). The degree of strength decay depends on many parameters, including the governing deformation mode, concrete confinement, shear strength, loading history, and level of axial load. Early and rapid strength decay can have a very significant effect on structural response and, if not considered in hysteretic modeling, can lead to significant errors in dynamic analysis.²

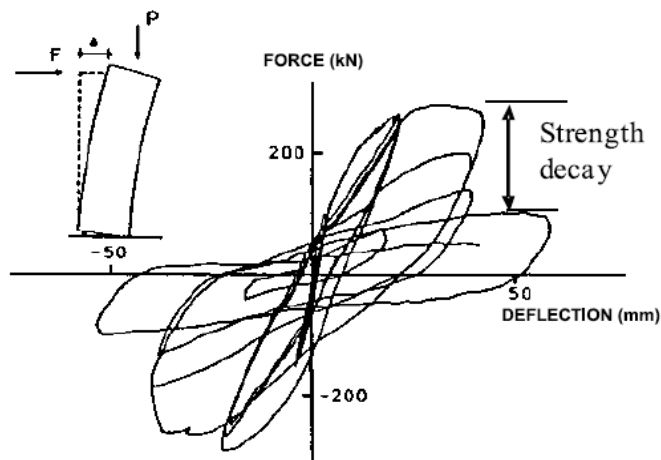


Figure 2-10 Strength decay in a column (Ozcebe and Saatcioglu 1989).

2.7.3.3. Slip or Pinching

Slip or pinching occurs as a result of crack closure, bolt slip, etc (Sivaselvan and Reinhorn, 1999).³ Hysteresis loops of both steel and reinforced concrete members

¹ M. Saatcioglu and J. Humar, 2000, *Dynamic analysis of buildings for earthquake resistant design. Proposed Earthquake Design Requirements of the NBCC*, pp 352

² M. Saatcioglu and J. Humar, 2000, *Dynamic analysis of buildings for earthquake resistant design. Proposed Earthquake Design Requirements of the NBCC*, pp 352

³ M..V Sivaselvan and A.M. Reinhorn, (1999), *Hysteretic Models for Cyclic Behavior of Deteriorating Inelastic Structures*, pp 14

generally show a marked change in slope during reloading. In steel members this may be attributed to the slippage of a bolt or a rivet until the force is completely reversed. The change in slope in concrete elements is associated with opening and closing of cracks under cyclic loading (Saatcioglu and Humar, 2002) ¹. Strength decay and pinching of hysteresis loops were modeled by Takayanagi and Schnobrich (1976), who modified the Takeda et al. model to incorporate features that are prevalent in shear and bar slip, as illustrated in Figure 2-11 (Saatcioglu and Humar, 2002) ².

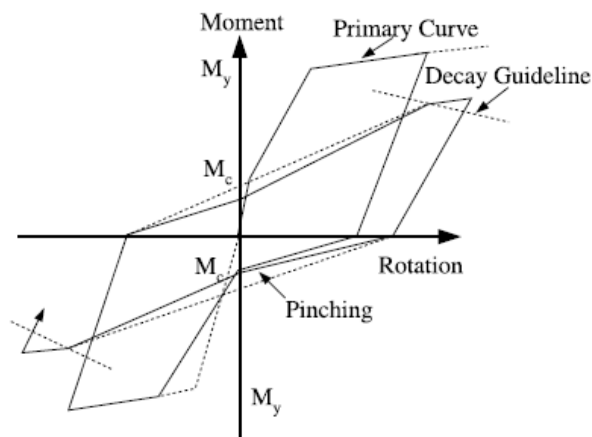


Figure 2-11 Hysteretic model incorporating pinching and strength decay (Takayanagi and Schnobrich 1976).

2.8. IDARC2D (A Program for the Inelastic Damage Analysis of Structures)

The computer program IDARC was introduced in 1987 as a two-dimensional analysis program to study the non-linear response of multistory reinforced concrete. The program was developed assuming that floor diaphragms behave as rigid horizontal links, therefore, only one horizontal degree of freedom is required per floor. This approach greatly reduces the total computational effort. Therefore, the building is modeled as a series of plane frames linked by a rigid horizontal diaphragm. Each frame is in the same vertical plane, and no torsional effects are considered. Since the floors are considered infinitely rigid, identical frames can be simply lumped together, and the stiffness contributions of each

¹ M. Saatcioglu and J. Humar, 2000, *Dynamic analysis of buildings for earthquake resistant design. Proposed Earthquake Design Requirements of the NBCC*, pp 352

² M. Saatcioglu and J. Humar, 2000, *Dynamic analysis of buildings for earthquake resistant design. Proposed Earthquake Design Requirements of the NBCC*, pp 353

typical frame factored by the number of duplicate equal frames. Input is only required for each of the typical frames (Reinhorn et.al, 2009)¹.

Version 7.0 of IDARC includes the following types of structural elements (Reinhorn and et.al, 2009)²:

- a) Column elements
- b) Beam elements
- c) Deep beam and column elements
- d) Rocking column elements
- e) Shear wall elements
- f) Edge column elements
- g) Transverse beam elements
- h) Rotational spring elements
- i) Visco-elastic damper elements
- j) Friction damper elements
- k) Hysteretic damper elements
- l) Infill panel elements
- m) Element moment releases

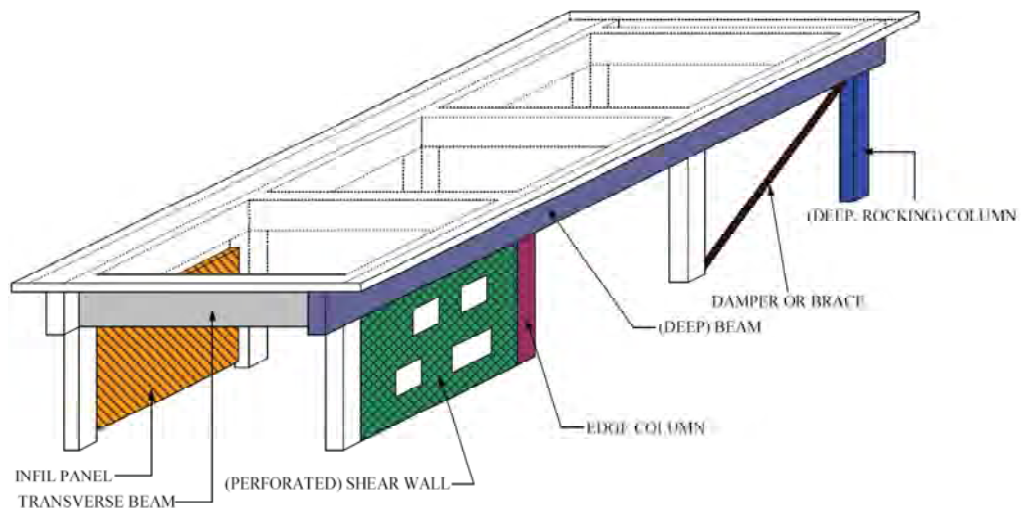


Figure 2-12 Structural Model definition by (Reinhorn et.al, 2009)³

¹A.M. Reinhorn and et.al, 2009, IDARC2D Version 7.0:A Program for the Inelastic Damage Analysis of Structures,pp.3

²A.M. Reinhorn and et.al, 2009, IDARC2D Version 7.0:A Program for the Inelastic Damage Analysis of Structures,pp.11

³A.M. Reinhorn and et.al, 2009, IDARC2D Version 7.0:A Program for the Inelastic Damage Analysis of Structures, Figure 3.1

The primary modelling technique employed in the computer program IDARC is the representation of the overall behavior of components in terms of macro models. Each component of the structural system is discretized into a series of macro elements: beam-column elements whose inelastic behavior is characterized by a primarily flexural response, shear-wall elements that may deform inelastically in the plane of the diaphragm, and inelastically rotational springs that may be attached at any node. Special-purpose elements such as viscoelastic braces and other friction devices for hysteretic energy dissipation were also developed. The reliability of the macroelements is enhanced through the introduction of distributed flexibility models which account for the effects of spread plasticity (Reinhorn and Kunnath). Nonlinear material behavior is specified by means of a generic hysteretic force-deformation model that incorporates stiffness degradation, strength deterioration and pinching or bond-slip effects. ¹

The program evaluates the nonlinear response of the structure under the following four possible analysis options (Reinhorn et.al, 2009) ²:

- a) Incremental Nonlinear Static Monotonic Analysis
- b) Incremental Nonlinear Static-Adaptive (Pushover) Analysis
- c) Nonlinear Quasi-static Analysis
- d) Eigenvalue Analysis
- e) Nonlinear Dynamic Analysis

The user may select any of the four options for the analysis, or a combination of a nonlinear static analysis with any of the three other analysis options. In this thesis work, nonlinear dynamic analysis option is selected because the nonlinear response of buildings to the highest level of accuracy can be assessed by making use of non-linear dynamic analysis procedure.

The final response quantities are expressed in terms of damage indices that provide engineers with a qualitative interpretation of the analysis. Important researches have been carried out to develop damage indicators to qualify the response of structures. The current

¹A.M. Reinhorn and S.Kunnath, 1994, IDARC2D:A Program for the Inelastic Damage Analysis of Reinforced Concrete Structures,pp.29

²A.M. Reinhorn and et.al, 2009, IDARC2D Version 7.0:A Program for the Inelastic Damage Analysis of Structures,pp.3

release of IDARC incorporates three models for damage indices: (i) a modified Park & Ang model (Park et al., 1984; Kunnath et al.1992b), introduced in the previous releases of the program, (ii) a new fatigue based damage model introduced by Reinhorn and Valles (1995), based on Bracci (1990), and (iii) an overall damage qualification based on the variation of the fundamental period of the structure.

The Park & Ang and the fatigue based damage model can be used to calculate different damage indices: element, story (subassembly), and overall building damage. However, for the story and overall damage indices the ultimate inter-story deformation or top story displacement are required, as well as the corresponding story yield shear force or base shear yield force level. To determine an estimate of the story and overall damage indices, weighting factors were introduced based on the energy absorption in the different structural elements or entire stories substructure.

The program IDARC includes the option to determine the response of the structure at instants during the analysis. Several types of response snapshots can be specified:

- a) Displacement profile.
- b) Element stress ratios.
- c) Structural collapse state.
- d) Damage indices.
- e) Dynamic characteristics (eigenvalue analysis).

Response snapshots can be requested by the user during pushover, quasi-static or dynamic analysis. Default snapshots will be reported, if requested by the user, for the first crack, yield or failure observed in any column, beam or shear wall in the structure during the analysis.

2.9. Seismic Hazard Level of Addis Ababa

According to EBCS-8:1995, Addis Ababa is located in zone '2' on the seismic hazard map of Ethiopia. Due to the recent area expansion of the Addis Ababa, some areas like Akaki, which is categorized as zone '3', are included in the city. In EBCS-8:1995 peak ground acceleration of 0.05g and 0.07g are given for zone '2' and zone '3' respectively. But codes

like (UBC-97, 1997)¹ and other researches done by (Kebede and Asfaw, 1996)² and (Radius, 1999)³ give higher peak ground accelerations. This significant difference of peak ground accelerations brings questions regarding the validity of the peak ground accelerations given by EBCS-8:1995. (Kassegne, 2006)⁴ proposed a more conservative zoning than the EBCS-8:1995 be adopted that increases the peak ground acceleration to at least 0.10g – 0.12g.

¹International Conference of Building Officials (ICBO), 1997, Uniform Building Code.

²F. Kebede and L. Asfaw, 1996, "Seismic Hazard Assessment for Ethiopia and the Neighboring Countries", *Sinet – Ethiopian Journal of Science*, 19(1)

³IDNDR RADIUS Project, 1999, Addis Ababa Case Study, Final Report, Prepared by Addis Ababa RADIUS Group.

⁴S.K Kassegne , 2006, ' Proposed Considerations for Revision of EBCS-8:1995 for Conservative Seismic Zoning and Stringent Requirements for Torsionally Irregular Buildings, *pp.6*

3. Methods and Procedures

3.1. Assumptions

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

- a. It is assumed that the case study buildings are constructed according to the original drawings provided by the designer.
- b. It is assumed that the case study buildings are designed for bedrock acceleration ($\alpha_o = 0.05$).
- c. It is assumed that the structures are designed according to EBCS and EBCS8, but the structural detailing was not checked against the recommendations of EBCS-2 and EBCS-8.

3.2. Procedures

- a. Collect information on the selected case study buildings.
- b. Evaluate if the collected data is sufficient enough to carry out seismic evaluation of the buildings.
- c. Generate artificial accelerogram that gives response spectrum compatible with EBCS 8 design spectrum.
- d. Model the buildings using IDARC analysis software.
- e. Analyze the IDARC 2D building model x-direction frames for bed rock acceleration, $\alpha_o = 0.05$.
- f. Analyze the IDARC 2D building model y-direction frames for bed rock acceleration, $\alpha_o = 0.05$.
- g. Evaluate the damage indices analysis results of IDARC 2D.
- h. Present the analysis result of selected elements at first crack, first yield and at collapse, if any.
- i. Present the time history of stories and damage state of frames graphically.
- j. Re-analyze each model for bedrock acceleration, $\alpha_o = 0.07$ and repeat steps e to i.
- k. Re-analyze each model for bedrock acceleration, $\alpha_o = 0.1$ and repeat steps e to i.

3.3. Case Studies

There are four case study structures considered in this thesis work. The buildings are located in Addis Ababa city. The four case study buildings will be analyzed for two design earthquake with peak ground acceleration 0.05g and 0.07g. A conservative peak ground acceleration of 0.10g is also used in the analysis to evaluate the performance of the case study buildings for seismic hazard which is higher than the design earthquake.

The information required according to Table 2.2 is not fully available for some of the case studies. The assumption that the buildings are constructed according to the original design will cover up information like condition survey of building, alteration and as built assessment and non structural walk through. The available information besides the structural drawings of the four case studies is shown in Table 3.1. The structural drawings of the buildings are included in the appendix.

Table 3.1 Available information of case studies

Item	CASE STUDIES			
	01	02	03	04
Structural calculations	No	Yes	Yes	Yes
Site seismicity, geotechnical report	No	No	No	No
Foundation report	No	Yes	No	No
Prior seismic assessment reports	No	No	No	No
Condition survey of building	Yes	No	No	No
Alteration and as built assessment	No	No	No	No
Walk through dimensioning	No	No	No	No
Nonstructural walk through	No	No	No	No
Core testing	No	No	No	No
Rebound hammer testing	No	No	No	No
Aggregate Testing	No	No	No	No
Reinforcement testing	No	No	No	No
Reinf. Location verification	No	No	No	No
Nonstructural exploration	No	No	No	No

3.4. Artificial Earthquake

Ground motions used in seismic response of structure include various characteristics depending on fault mechanism of earthquake, wave propagation, and amplification of soil type. It is a difficult task to quantitatively examine all affecting factors. In seismic design, design response spectrum generally represents its characteristics. Therefore, in this study, an artificial accelerogram that gives compatible response spectrum with the EBCS-8-1995

design spectrum was developed and used for analysis. The artificial acceleration is shown in Figure 3.1 and the response spectrum developed is compared with the EBCS8-8-1995 design spectrum on Figure 3.2.

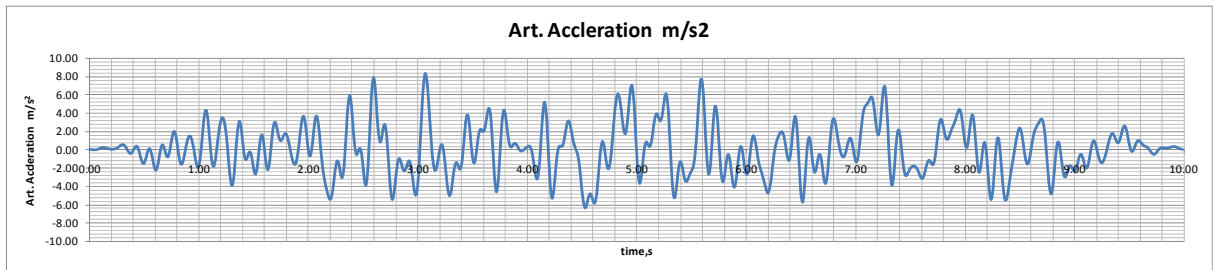


Figure 3-1 Artificially generated wave data

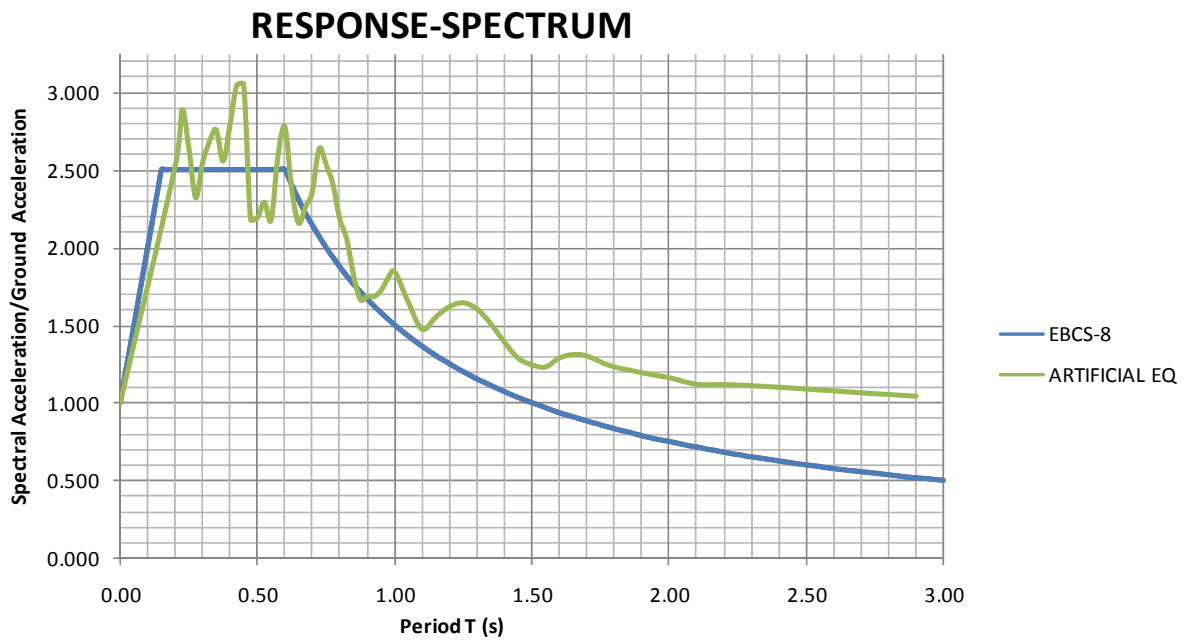


Figure 3-2 Response spectrum Comparison

4. Results

4.1. Building Mass for Seismic

The weight used for seismic analysis in the IDARC 2D is slightly higher as compared to the ETABS analysis result. The result is summarized in table 4.1.

Table 4.1 Seismic mass of IDARC 2D and ETABS

CASE STUDIES	WEIGHT USED FOR SEISMIC ANALYSIS (KN)			
	IDARC 2D-X FRAME	IDARC 2D-Y FRAME	ETABS 3D-EQX	ETABS 3D-EQY
01	21261.43	21261.43	20850.00	20850.00
02	24927.83	24927.83	24386.36	24386.36
03	10448.31	10455.91	10150.88	10150.88
04	11660.57	11660.57	11450.06	11450.06

4.2. Damaged Index of Frames (Bed rock acceleration, $\alpha_o = 0.05, 0.07&0.10$)

The damage index of the building at first crack and first yield of columns and beams and also at collapse of the building, if any, is reported in the IDAR 2D analysis result and summarized in Table 4.2 - Table 4.5.

The after earthquake degree of damage, damage state for service, usability condition and the expected appearance of each case study are shown in Table 4.6 - Table 4.17.

It is observed that

- a) Case Study-01 fails for both X and Y direction frame analysis at an early stage of the earthquake
- b) Case study-02 and Case Study-03 perform well for both X and Y direction frame analysis.
- c) Case study-04 performs well for X direction frame analysis but fails for Y direction frame analysis.

Table 4.2 Case Study 01 – Damage index

CASE STUDY 01	ELEMENT	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s), $\alpha_0 = 0.05$	Damage Index	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s), $\alpha_0 = 0.07$	Damage Index	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s), $\alpha_0 = 0.10$	Damage Index
X-FRAME	COLUMN (CRACK)	0.005	0.128	0.005	0.128	0.005	0.128
	BEAM (CRACK)	0.005	0.128	0.005	0.128	0.005	0.128
	COLUMN (YIELD)	0.01	2.11	0.01	2.11	0.01	2.11
	BEAM (YIELD)	-	2.11	-	2.11	-	2.11
	COLUMN (AFTER EQ)	0.02	2.11	0.02	2.11	0.02	2.11
	BEAM (AFTER EQ)	0.02	2.11	0.02	2.11	0.02	2.11
Y-FRAME	COLUMN (CRACK)	0.005	0.057	0.005	0.057	0.005	0.057
	BEAM (CRACK)	0.005	0.057	0.005	0.057	0.005	0.057
	COLUMN (YIELD)	0.01	0.167	0.01	0.167	0.01	0.167
	BEAM (YIELD)	0.01	0.167	0.01	0.167	0.01	0.167
	COLUMN (AFTER EQ)	0.02	1.2	0.02	1.2	0.02	1.2
	BEAM (AFTER EQ)	0.02	1.2	0.02	1.2	0.02	1.2

Table 4.3 Case Study 02 – Damage index

CASE STUDY 02	ELEMENT	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s), $\alpha_0 = 0.05$	Damage Index	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s), $\alpha_0 = 0.07$	Damage Index	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s), $\alpha_0 = 0.10$	Damage Index
X-FRAME	COLUMN (CRACK)	8.815	0.051	6.125	0.002	2.72	0.002
	BEAM (CRACK)	6.735	0	3.085	0	2.08	0
	COLUMN (YIELD)	-	0.033	-	0.021	-	0.065
	BEAM (YIELD)	7.99	0.033	6.89	0.021	6.745	0.065
	COLUMN (AFTER EQ)	-	0.053	-	0.064	-	0.079
	BEAM (AFTER EQ)	-	0.053	-	0.064	-	0.079
Y-FRAME	COLUMN (CRACK)	9.025	0.031	3.34	0.003	2.655	0.002
	BEAM (CRACK)	3.075	0	2.965	0	1.895	0
	COLUMN (YIELD)	-	0.015	-	0.025	-	0.016
	BEAM (YIELD)	6.85	0.015	6.75	0.025	3.09	0.016
	COLUMN (AFTER EQ)	-	0.032	-	0.044	-	0.068
	BEAM (AFTER EQ)	-	0.032	-	0.044	-	0.068

Table 4.4 Case Study 03 – Damage index

CASE STUDY 03	ELEMENT	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s), $\alpha_0 = 0.05$	Damage Index	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s), $\alpha_0 = 0.07$	Damage Index	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s), $\alpha_0 = 0.10$	Damage Index
X-FRAME	COLUMN (CRACK)	5.255	0.020	4.635	0.018	3.995	0.018
	BEAM (CRACK)	3.915	0	1.475	0	1.29	0.016
	COLUMN (YIELD)	-	0.013	-	0.018	7.575	0.092
	BEAM (YIELD)	4.75	0.013	4.63	0.018	5.445	0.058
	COLUMN (AFTER EQ)	-	0.028	-	0.048	-	0.093
	BEAM (AFTER EQ)	-	0.028	-	0.048	-	0.093
Y-FRAME	COLUMN (CRACK)	5.245	0.019	4.64	0.018	3.995	0.018
	BEAM (CRACK)	3.93	0	3.38	0	1.275	0
	COLUMN (YIELD)	-	0.017	-	0.017	-	0.016
	BEAM (YIELD)	5.22	0.017	4.62	0.017	3.96	0.016
	COLUMN (AFTER EQ)	-	0.033	-	0.052	-	0.089
	BEAM (AFTER EQ)	-	0.033	-	0.052	-	0.089

Table 4.5 Case Study 04 – Damage index

CASE STUDY 04	ELEMENT	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s), $\alpha_0 = 0.05$	Damage Index	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s), $\alpha_0 = 0.07$	Damage Index	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s), $\alpha_0 = 0.10$	Damage Index
X-FRAME	COLUMN (CRACK)	4.87	0.024	3.605	0.004	2.525	0.01
	BEAM (CRACK)	1.325	0	1.28	0	0.945	0
	COLUMN (YIELD)	-	0.017	-	0.018	7.545	0.125
	BEAM (YIELD)	4.68	0.017	4.06	0.018	3.995	0.022
	COLUMN (AFTER EQ)	-	0.053	-	0.089	-	0.134
	BEAM (AFTER EQ)	-	0.053	-	0.089	-	0.134
Y-FRAME	COLUMN (CRACK)	0.005	0.275	0.005	0.275	0.005	0.275
	BEAM (CRACK)	0.005	0.275	0.005	0.275	0.005	0.275
	COLUMN (YIELD)	0.005	0.275	0.005	0.275	0.005	0.275
	BEAM (YIELD)	0.005	0.275	0.005	0.275	0.005	0.275
	COLUMN (AFTER EQ)	0.01	1.2	0.01	1.2	0.01	1.2
	BEAM (AFTER EQ)	0.01	1.2	0.01	1.2	0.01	1.2

Table 4.6 Case Study 01 – Damage analysis Park & Ang $\alpha_0 = 0.05$

CASE STUDY 01	ELEMENT	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s), $\alpha_0 = 0.05$	Damage Index	Degree of Damage	Damage State (Service)	Usability	Appearance
X-FRAME	COLUMN (CRACK)	0.005	0.128	Slightly	Serviceable	Usable	Undeformed/uncracked
	BEAM (CRACK)	0.005	0.128	Slightly	Serviceable	Usable	Undeformed/uncracked
	COLUMN (YIELD)	0.01	2.11	Collapse	Collapse	Unusable	Loss of shear/axial capacity
	BEAM (YIELD)	-	2.11	Collapse	Collapse	Unusable	Loss of shear/axial capacity
	COLUMN (AFTER EQ)	0.02	2.11	Collapse	Collapse	Unusable	Loss of shear/axial capacity
	BEAM (AFTER EQ)	0.02	2.11	Collapse	Collapse	Unusable	Loss of shear/axial capacity
Y-FRAME	COLUMN (CRACK)	0.005	0.057	Slightly	Serviceable	Usable	Undeformed/uncracked
	BEAM (CRACK)	0.005	0.057	Slightly	Serviceable	Usable	Undeformed/uncracked
	COLUMN (YIELD)	0.01	0.167	Slightly	Serviceable	Usable	Undeformed/uncracked
	BEAM (YIELD)	0.01	0.167	Slightly	Serviceable	Usable	Undeformed/uncracked
	COLUMN (AFTER EQ)	0.02	1.2	Collapse	Collapse	Unusable	Loss of shear/axial capacity
	BEAM (AFTER EQ)	0.02	1.2	Collapse	Collapse	Unusable	Loss of shear/axial capacity

Table 4.7 Case Study 01 – Damage analysis Park & Ang $\alpha_0 = 0.07$

CASE STUDY 01	ELEMENT	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s), $\alpha_0 = 0.07$	Damage Index	Degree of Damage	Damage State (Service)	Usability	Appearance
X-FRAME	COLUMN (CRACK)	0.005	0.128	Slightly	Serviceable	Usable	Undeformed/uncracked
	BEAM (CRACK)	0.005	0.128	Slightly	Serviceable	Usable	Undeformed/uncracked
	COLUMN (YIELD)	0.01	2.11	Collapse	Collapse	Unusable	Loss of shear/axial capacity
	BEAM (YIELD)	-	2.11	Collapse	Collapse	Unusable	Loss of shear/axial capacity
	COLUMN (AFTER EQ)	0.02	2.11	Collapse	Collapse	Unusable	Loss of shear/axial capacity
	BEAM (AFTER EQ)	0.02	2.11	Collapse	Collapse	Unusable	Loss of shear/axial capacity
Y-FRAME	COLUMN (CRACK)	0.005	0.057	Slightly	Serviceable	Usable	Undeformed/uncracked
	BEAM (CRACK)	0.005	0.057	Slightly	Serviceable	Usable	Undeformed/uncracked
	COLUMN (YIELD)	0.01	0.167	Slightly	Serviceable	Usable	Undeformed/uncracked
	BEAM (YIELD)	0.01	0.167	Slightly	Serviceable	Usable	Undeformed/uncracked
	COLUMN (AFTER EQ)	0.02	1.2	Collapse	Collapse	Unusable	Loss of shear/axial capacity
	BEAM (AFTER EQ)	0.02	1.2	Collapse	Collapse	Unusable	Loss of shear/axial capacity

Table 4.8 Case Study 01 – Damage analysis Park & Ang $\alpha_0 = 0.10$

CASE STUDY 01	ELEMENT	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s), $\alpha_0 = 0.10$	Damage Index	Degree of Damage	Damage State (Service)	Usability	Appearance
X-FRAME	COLUMN (CRACK)	0.005	0.128	Slightly	Serviceable	Usable	Undeformed/uncracked
	BEAM (CRACK)	0.005	0.128	Slightly	Serviceable	Usable	Undeformed/uncracked
	COLUMN (YIELD)	0.01	2.11	Collapse	Collapse	Unusable	Loss of shear/axial capacity
	BEAM (YIELD)	-	2.11	Collapse	Collapse	Unusable	Loss of shear/axial capacity
	COLUMN (AFTER EQ)	0.02	2.11	Collapse	Collapse	Unusable	Loss of shear/axial capacity
	BEAM (AFTER EQ)	0.02	2.11	Collapse	Collapse	Unusable	Loss of shear/axial capacity
Y-FRAME	COLUMN (CRACK)	0.005	0.057	Slightly	Serviceable	Usable	Undeformed/uncracked
	BEAM (CRACK)	0.005	0.057	Slightly	Serviceable	Usable	Undeformed/uncracked
	COLUMN (YIELD)	0.01	0.167	Slightly	Serviceable	Usable	Undeformed/uncracked
	BEAM (YIELD)	0.01	0.167	Slightly	Serviceable	Usable	Undeformed/uncracked
	COLUMN (AFTER EQ)	0.02	1.2	Collapse	Collapse	Unusable	Loss of shear/axial capacity
	BEAM (AFTER EQ)	0.02	1.2	Collapse	Collapse	Unusable	Loss of shear/axial capacity

Table 4.9 Case Study 02 – Damage analysis Park & Ang $\alpha_0 = 0.05$

CASE STUDY 02	ELEMENT	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s), $\alpha_0 = 0.05$	Damage Index	Degree of Damage	Damage State (Service)	Usability	Appearance
X-FRAME	COLUMN (CRACK)	8.815	0.051	Slightly	Serviceable	Usable	Undeformed/uncracked
	BEAM (CRACK)	6.735	0	None	Undamaged	Usable	Undeformed/uncracked
	COLUMN (YIELD)	-	0.033	Slightly	Serviceable	Usable	Undeformed/uncracked
	BEAM (YIELD)	7.99	0.033	Slightly	Serviceable	Usable	Undeformed/uncracked
	COLUMN (AFTER EQ)	-	0.053	Slightly	Serviceable	Usable	Undeformed/uncracked
	BEAM (AFTER EQ)	-	0.053	Slightly	Serviceable	Usable	Undeformed/uncracked
Y-FRAME	COLUMN (CRACK)	9.025	0.031	Slightly	Serviceable	Usable	Undeformed/uncracked
	BEAM (CRACK)	3.075	0	None	Undamaged	Usable	Undeformed/uncracked
	COLUMN (YIELD)	-	0.015	Slightly	Serviceable	Usable	Undeformed/uncracked
	BEAM (YIELD)	6.85	0.015	Slightly	Serviceable	Usable	Undeformed/uncracked
	COLUMN (AFTER EQ)	-	0.032	Slightly	Serviceable	Usable	Undeformed/uncracked
	BEAM (AFTER EQ)	-	0.032	Slightly	Serviceable	Usable	Undeformed/uncracked

Table 4.10 Case Study 02 – Damage analysis Park & Ang $\alpha_0 = 0.07$

CASE STUDY 02	ELEMENT	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s), $\alpha_0 = 0.07$	Damage Index	Degree of Damage	Damage State (Service)	Usability	Appearance
X-FRAME	COLUMN (CRACK)	6.125	0.002	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (CRACK)	3.085	0	None	Undamaged	Usable	Undeformed/ uncracked
	COLUMN (YIELD)	-	0.021	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (YIELD)	6.89	0.021	Slightly	Serviceable	Usable	Undeformed/ uncracked
	COLUMN (AFTER EQ)	-	0.064	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (AFTER EQ)	-	0.064	Slightly	Serviceable	Usable	Undeformed/ uncracked
Y-FRAME	COLUMN (CRACK)	3.34	0.003	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (CRACK)	2.965	0	None	Undamaged	Usable	Undeformed/ uncracked
	COLUMN (YIELD)	-	0.025	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (YIELD)	6.75	0.025	Slightly	Serviceable	Usable	Undeformed/ uncracked
	COLUMN (AFTER EQ)	-	0.044	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (AFTER EQ)	-	0.044	Slightly	Serviceable	Usable	Undeformed/ uncracked

Table 4.11 Case Study 02 – Damage analysis Park & Ang $\alpha_0 = 0.10$

CASE STUDY 02	ELEMENT	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s), $\alpha_0 = 0.10$	Damage Index	Degree of Damage	Damage State (Service)	Usability	Appearance
X-FRAME	COLUMN (CRACK)	2.72	0.002	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (CRACK)	2.08	0	None	Undamaged	Usable	Undeformed/ uncracked
	COLUMN (YIELD)	-	0.065	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (YIELD)	6.745	0.065	Slightly	Serviceable	Usable	Undeformed/ uncracked
	COLUMN (AFTER EQ)	-	0.079	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (AFTER EQ)	-	0.079	Slightly	Serviceable	Usable	Undeformed/ uncracked
Y-FRAME	COLUMN (CRACK)	2.655	0.002	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (CRACK)	1.895	0	None	Undamaged	Usable	Undeformed/ uncracked
	COLUMN (YIELD)	-	0.016	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (YIELD)	3.09	0.016	Slightly	Serviceable	Usable	Undeformed/ uncracked
	COLUMN (AFTER EQ)	-	0.068	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (AFTER EQ)	-	0.068	Slightly	Serviceable	Usable	Undeformed/ uncracked

Table 4.12 Case Study 03 – Damage analysis Park & Ang $\alpha_o = 0.05$

CASE STUDY 03	ELEMENT	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s), $\alpha_o = 0.05$	Damage Index	Degree of Damage	Damage State (Service)	Usability	Appearance
X-FRAME	COLUMN (CRACK)	5.255	0.020	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (CRACK)	3.915	0	None	Undamaged	Usable	Undeformed/ uncracked
	COLUMN (YIELD)	-	0.013	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (YIELD)	4.75	0.013	Slightly	Serviceable	Usable	Undeformed/ uncracked
	COLUMN (AFTER EQ)	-	0.028	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (AFTER EQ)	-	0.028	Slightly	Serviceable	Usable	Undeformed/ uncracked
Y-FRAME	COLUMN (CRACK)	5.245	0.019	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (CRACK)	3.93	0	None	Undamaged	Usable	Undeformed/ uncracked
	COLUMN (YIELD)	-	0.017	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (YIELD)	5.22	0.017	Slightly	Serviceable	Usable	Undeformed/ uncracked
	COLUMN (AFTER EQ)	-	0.033	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (AFTER EQ)	-	0.033	Slightly	Serviceable	Usable	Undeformed/ uncracked

Table 4.13 Case Study 03 – Damage analysis Park & Ang $\alpha_o = 0.07$

CASE STUDY 03	ELEMENT	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s), $\alpha_o = 0.07$	Damage Index	Degree of Damage	Damage State (Service)	Usability	Appearance
X-FRAME	COLUMN (CRACK)	4.635	0.018	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (CRACK)	1.475	0	None	Undamaged	Usable	Undeformed/ uncracked
	COLUMN (YIELD)	-	0.018	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (YIELD)	4.63	0.018	Slightly	Serviceable	Usable	Undeformed/ uncracked
	COLUMN (AFTER EQ)	-	0.048	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (AFTER EQ)	-	0.048	Slightly	Serviceable	Usable	Undeformed/ uncracked
Y-FRAME	COLUMN (CRACK)	4.64	0.018	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (CRACK)	3.38	0	None	Undamaged	Usable	Undeformed/ uncracked
	COLUMN (YIELD)	-	0.017	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (YIELD)	4.62	0.017	Slightly	Serviceable	Usable	Undeformed/ uncracked
	COLUMN (AFTER EQ)	-	0.052	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (AFTER EQ)	-	0.052	Slightly	Serviceable	Usable	Undeformed/ uncracked

Table 4.14 Case Study 03 – Damage analysis Park & Ang $\alpha_o = 0.10$

CASE STUDY 03	ELEMENT	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s) , $\alpha_o = 0.10$	Damage Index	Degree of Damage	Damage State (Service)	Usability	Appearance
X-FRAME	COLUMN (CRACK)	3.995	0.018	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (CRACK)	1.29	0.016	Slightly	Serviceable	Usable	Undeformed/ uncracked
	COLUMN (YIELD)	7.575	0.092	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (YIELD)	5.445	0.058	Slightly	Serviceable	Usable	Undeformed/ uncracked
	COLUMN (AFTER EQ)	-	0.093	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (AFTER EQ)	-	0.093	Slightly	Serviceable	Usable	Undeformed/ uncracked
Y-FRAME	COLUMN (CRACK)	3.995	0.018	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (CRACK)	1.275	0	None	Undamaged	Usable	Undeformed/ uncracked
	COLUMN (YIELD)	-	0.016	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (YIELD)	3.96	0.016	Slightly	Serviceable	Usable	Undeformed/ uncracked
	COLUMN (AFTER EQ)	-	0.089	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (AFTER EQ)	-	0.089	Slightly	Serviceable	Usable	Undeformed/ uncracked

Table 4.15 Case Study 04 – Damage analysis Park & Ang $\alpha_o = 0.05$

CASE STUDY 04	ELEMENT	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s) , $\alpha_o = 0.05$	Damage Index	Degree of Damage	Damage State (Service)	Usability	Appearance
X-FRAME	COLUMN(CRACK)	4.87	0.024	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (CRACK)	1.325	0	None	Undamaged	Usable	Undeformed/ uncracked
	COLUMN (YIELD)	-	0.017	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (YIELD)	4.68	0.017	Slightly	Serviceable	Usable	Undeformed/ uncracked
	COLUMN (AFTER EQ)	-	0.053	Slightly	Serviceable	Usable	Undeformed/ uncracked
	BEAM (AFTER EQ)	-	0.053	Slightly	Serviceable	Usable	Undeformed/ uncracked
Y-FRAME	COLUMN (CRACK)	0.005	0.275	Minor	Serviceable	Usable	Moderate to Severe Cracking
	BEAM (CRACK)	0.005	0.275	Minor	Serviceable	Usable	Moderate to Severe Cracking
	COLUMN (YIELD)	0.005	0.275	Minor	Serviceable	Usable	Moderate to Severe Cracking
	BEAM (YIELD)	0.005	0.275	Minor	Serviceable	Usable	Moderate to Severe Cracking
	COLUMN (AFTER EQ)	0.01	1.2	Collapse	Collapse	Unusable	Loss of shear/ axial capacity
	BEAM (AFTER EQ)	0.01	1.2	Collapse	Collapse	Unusable	Loss of shear/ axial capacity

Table 4.16 Case Study 04 – Damage analysis Park & Ang $\alpha_0 = 0.07$

CASE STUDY 04	ELEMENT	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s), $\alpha_0 = 0.07$	Damage Index	Degree of Damage	Damage State (Service)	Usability	Appearance
X-FRAME	COLUMN(CRACK)	3.605	0.004	Slightly	Serviceable	Usable	Undeformed/uncracked
	BEAM (CRACK)	1.28	0	None	Undamaged	Usable	Undeformed/uncracked
	COLUMN (YIELD)	-	0.018	Slightly	Serviceable	Usable	Undeformed/uncracked
	BEAM (YIELD)	4.06	0.018	Slightly	Serviceable	Usable	Undeformed/uncracked
	COLUMN (AFTER EQ)	-	0.089	Slightly	Serviceable	Usable	Undeformed/uncracked
	BEAM (AFTER EQ)	-	0.089	Slightly	Serviceable	Usable	Undeformed/uncracked
Y-FRAME	COLUMN (CRACK)	0.005	0.275	Minor	Serviceable	Usable	Moderate to Severe Cracking
	BEAM (CRACK)	0.005	0.275	Minor	Serviceable	Usable	Moderate to Severe Cracking
	COLUMN (YIELD)	0.005	0.275	Minor	Serviceable	Usable	Moderate to Severe Cracking
	BEAM (YIELD)	0.005	0.275	Minor	Serviceable	Usable	Moderate to Severe Cracking
	COLUMN (AFTER EQ)	0.01	1.2	Collapse	Collapse	Unusable	Loss of shear/axial capacity
	BEAM (AFTER EQ)	0.01	1.2	Collapse	Collapse	Unusable	Loss of shear/axial capacity

Table 4.17 Case Study 04 – Damage analysis Park & Ang $\alpha_0 = 0.10$

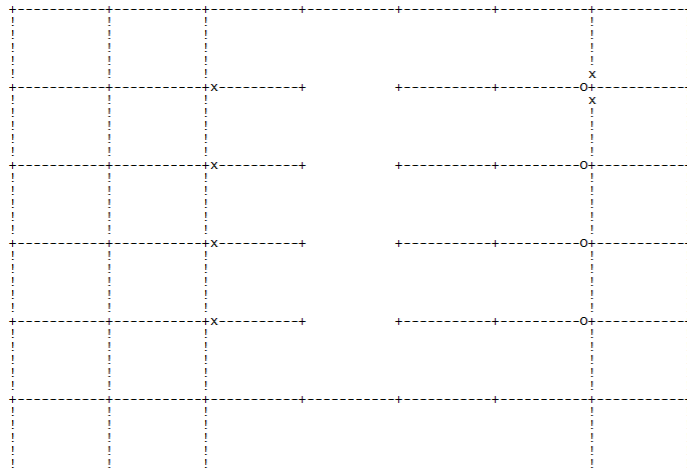
CASE STUDY 04	ELEMENT	TIME OF FIRST CRACK OR YIELD OR COLLAPSE(s), $\alpha_0 = 0.10$	Damage Index	Degree of Damage	Damage State (Service)	Usability	Appearance
X-FRAME	COLUMN(CRACK)	2.525	0.01	Slightly	Serviceable	Usable	Undeformed/uncracked
	BEAM (CRACK)	0.945	0	None	Undamaged	Usable	Undeformed/uncracked
	COLUMN (YIELD)	7.545	0.125	Slightly	Serviceable	Usable	Undeformed/uncracked
	BEAM (YIELD)	3.995	0.022	Slightly	Serviceable	Usable	Undeformed/uncracked
	COLUMN (AFTER EQ)	-	0.134	Slightly	Serviceable	Usable	Undeformed/uncracked
	BEAM (AFTER EQ)	-	0.134	Slightly	Serviceable	Usable	Undeformed/uncracked
Y-FRAME	COLUMN (CRACK)	0.005	0.275	Minor	Serviceable	Usable	Moderate to Severe Cracking
	BEAM (CRACK)	0.005	0.275	Minor	Serviceable	Usable	Moderate to Severe Cracking
	COLUMN (YIELD)	0.005	0.275	Minor	Serviceable	Usable	Moderate to Severe Cracking
	BEAM (YIELD)	0.005	0.275	Minor	Serviceable	Usable	Moderate to Severe Cracking
	COLUMN (AFTER EQ)	0.01	1.2	Collapse	Collapse	Unusable	Loss of shear/axial capacity
	BEAM (AFTER EQ)	0.01	1.2	Collapse	Collapse	Unusable	Loss of shear/axial capacity

4.3. Damage state and Time History Results

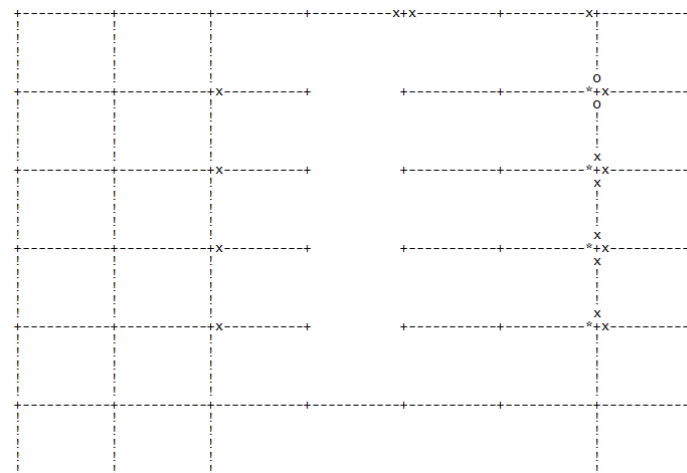
Time history of selected stories and damage state of building frames are shown on Figure 4.2 through 4.7.

- = BEAM	X = CRACKING(FOR CONCRETE)
! = COLUMN	INITIAL YIELD(FOR STEEL)
W = SHEAR WALL	O = PLASTIC HINGE DEVELOPED
I = EDGE COLUMN	* = LOCAL FAUILURE(EXCEED CRITERIA)
	FOR EDGE COLS: C: COMPRESSION
	T: TENSION
	O: TENSILE YIELD
	X = INITIAL SHEAR CRACK OR YIELD
	\$ = SHEAR FAILURE

Figure 4-1 Damage state notations



(a) First Beam Yield and First Column crack at $t = 0.005s$ - Frame 06



(b) First Column yields at $t = 0.01s$ - Frame 06

Figure 4-2 Case study 1X ($\alpha_o = 0.05$)

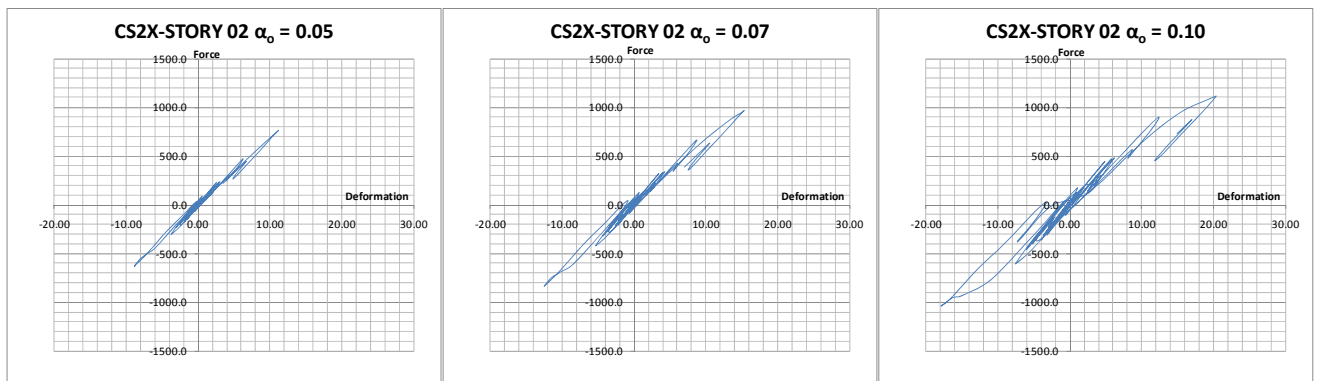
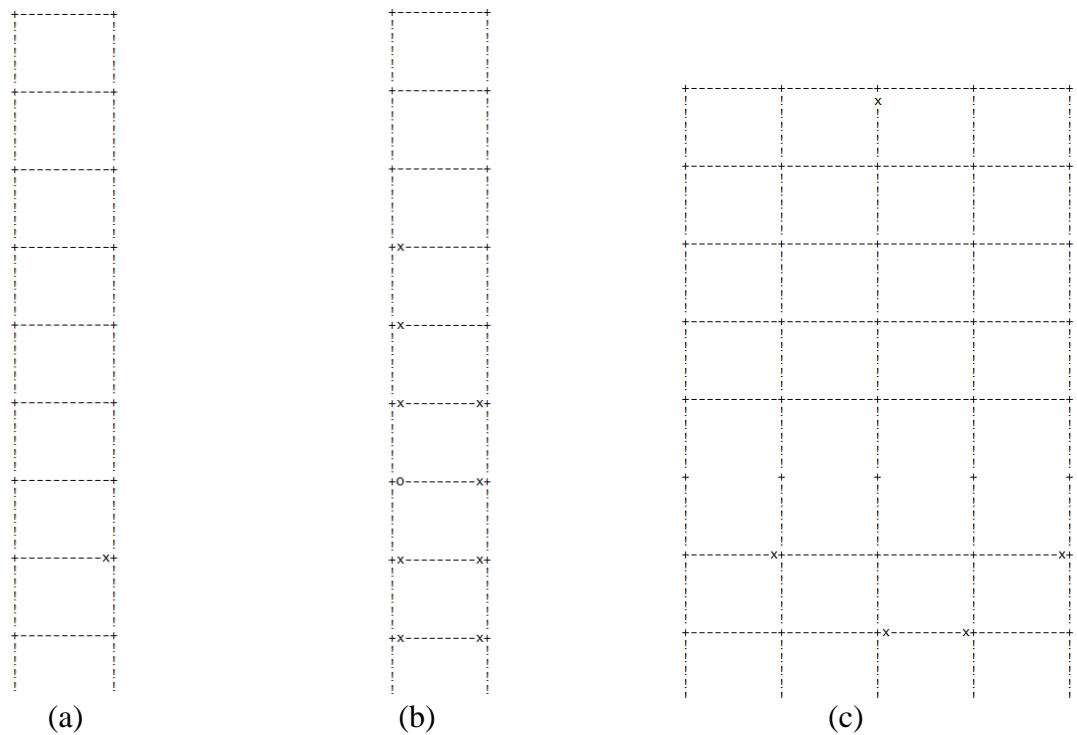
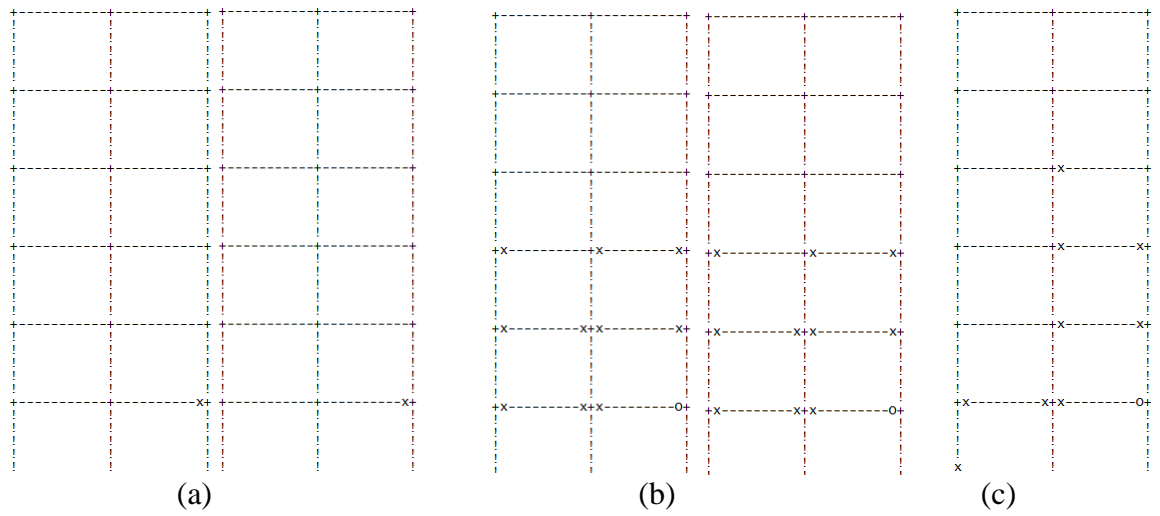


Figure 4-3 Time History for Stories – Case Study 2X-Frame



- (a) First Beam cracks at $t = 6.375s$ - Frame 02
- (b) First Beam yields at $t = 7.990s$ - Frame 02
- (c) First Column cracks at $t = 8.815s$ - Frame 05

Figure 4-4 Case study 2X ($\alpha_o = 0.05$)



(a) First Beam cracks at $t = 3.930s$ - Frame 01&08

(b) First Beam yields at $t = 5.220s$ - Frame 01&08

(c) First Column cracks at $t = 5.245s$ - Frame 05

Figure 4-5 Case study 3Y ($\alpha_o = 0.05$)

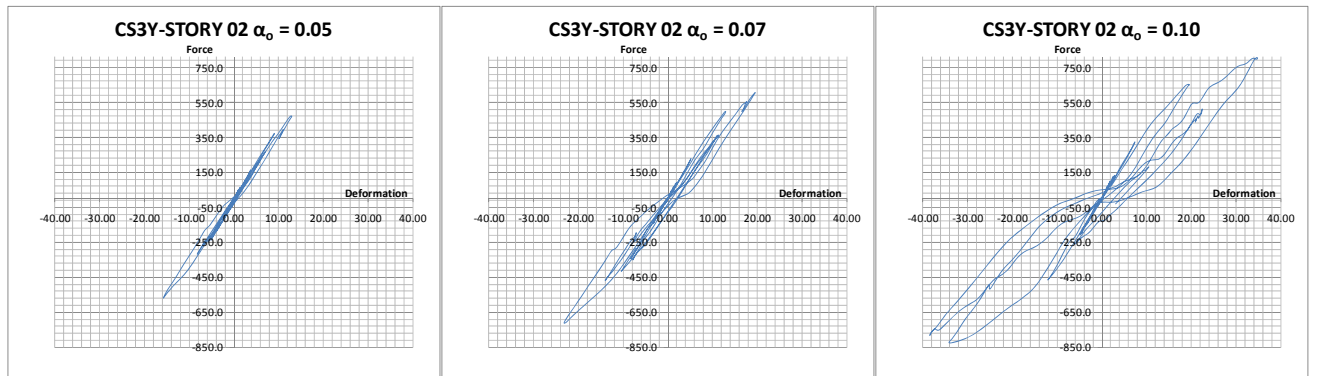


Figure 4-6 Time History for Stories – Case Study 3Y- Frame

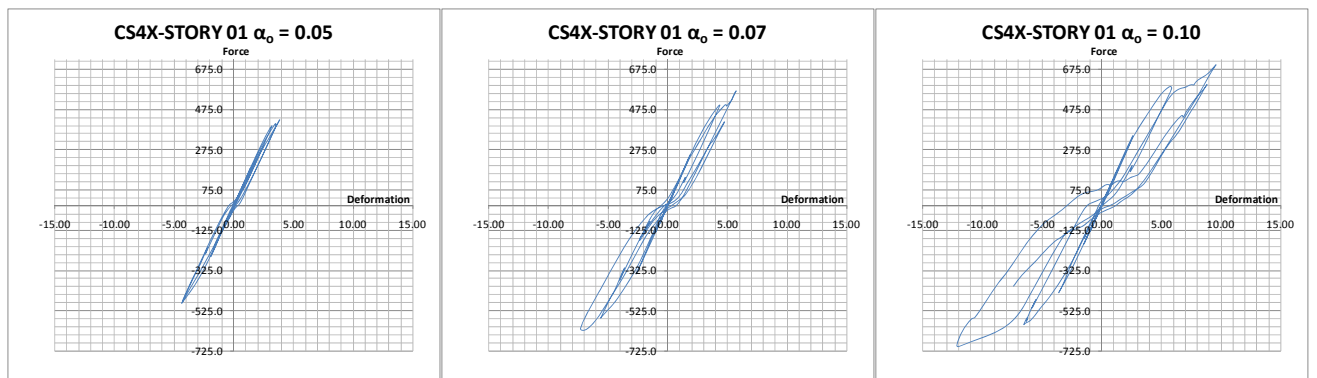


Figure 4-7 Time History for Stories – Case Study 4X- Frame

5. Discussions

Case study 01

- a) Fails at the beginning of the earthquake which indicates loss of structural capacity without dissipating the input energy generated by the earthquake.
- b) All stories show linear relationship of force-deformation response.
- c) Stiffness degradation, strength decay and pinching are not observed in any of the force-deformation response.
- d) Similar force-deformation relationship is observed for different bed rock acceleration ratio.

Case study 02

- a) Performs well for all three bed rock acceleration ratio which indicates the building frames are capable of dissipating input energy much higher than the design input.
- b) For both direction frames, the stiffness degradation, strength decay and pinching behaviors in the force-deformation response increase from bottom story to top story.
- c) For both direction frames, the stiffness degradation, strength decay and pinching behaviors in the force-deformation response increase with increasing bed rock acceleration ratio.

Case study 03

- a) Performs well for all three bed rock acceleration ratio which indicates the building frames are capable of dissipating input energy much higher than the design input.
- b) For both direction frames, the stiffness degradation, strength decay and pinching behaviors in the force-deformation response increase from bottom story to top story.
- c) For both direction frames, the stiffness degradation, strength decay and pinching behaviors in the force-deformation response increase with increasing bed rock acceleration ratio.

Case study 04

- a) For X frames, the stiffness degradation, strength decay and pinching behaviors in the force-deformation response increase from bottom story to top story.
- b) For X frames, the stiffness degradation, strength decay and pinching behaviors in the force-deformation response increase with increasing bed rock acceleration ratio.
- c) Y frames fail at the beginning of the earthquake which indicates loss of structural capacity without dissipating the input energy generated by the earthquake.
- d) For Y frames, stiffness degradation, strength decay and pinching are not observed in any of the force-deformation response.
- e) For Y frames, similar force-deformation relationship is observed for different bed rock acceleration ratio.

6. Conclusions and Recommendations

6.1. Conclusions

Seismic vulnerability of selected buildings constructed in Addis Ababa was assessed based on three different bed rock acceleration ratios. Nonlinear dynamic analysis was performed on four different reinforced concrete structures using IDARC 2d software.

From the results,

- It is observed that nonlinear dynamic analysis procedure captures the nonlinear characteristics of reinforced concrete buildings.
- Considering the results shown for case study 01 and 04, it is observed that the buildings deform beyond the limit of linearly elastic range and due to the high damage of the main structural elements, the buildings will no longer give service or a total collapse will occur after earthquake.
- Case study 04 fails for Y direction analysis because the frames in this direction are weak. The X direction frame performs well and shows visible nonlinear behaviors.
- Considering the results shown for case study 02 and 03, it is observed that the structures are capable of withstanding earthquake with high bedrock acceleration ratio greater than the design value.
- The force deformation time history of case study 02 and 03 shows significant stiffness degradation and strength decay because of the cracking and yielding of their structural members. Pinching behaviors are also observed which are caused by the closing and opening of cracks of reversal loading.
- Even though the IDARC 2D software is helpful in capturing the nonlinear behavior of the case study buildings, it lacks considering torsional effects caused by non symmetric buildings.

6.2. Recommendations

- It is recommended that nonlinear dynamic analysis procedures shall be used for seismic evaluation of existing buildings.

- Out of the four case study buildings considered in this study, 50% are found to be insufficient to resist the earthquake load induced on them. Therefore the author strongly suggests the evaluation of existing buildings constructed in Addis Ababa because if an earthquake of the expected intensity occurs, the result may be catastrophic.
- It is observed that the input process of IDARC 2D needs too many lines in text format. And also gathering the output is time taking. Therefore, further developments need to be done on the pre-processor and post-processor to save time and avoid input errors.
- It is also recommended that the Ethiopian building code standards include provisions that address strengthening of buildings.
- Finally the author suggests that such a study shall further be conducted on large number of buildings located in different sub cities of Addis Ababa, in order to get a detailed and realistic damage map of the city.

References

1. A. Elghazouli. (2009) *Seismic Design of Buildings to Eurocode 8*, Spon Press, London and New York.
2. A. Elnashai and L. Sarno. (2008) *Fundamentals of Earthquake Engineering*, John Wiley & Sons, Ltd., United Kingdom.
3. A. Pecker. (2007) *Advanced Earthquake Engineering Analysis*, Springer Wien NewYork., USA.
4. A.M. Reinhorn et.al. (2009) *IDARC2D Version 7.0: A Program for the Inelastic Damage Analysis of Structures*, University at Buffalo, USA.
5. D. Dowrick. (2009) *Earthquake Resistance Design and Risk Reduction*, 2nd edn, John Wiley & Sons, Ltd., United Kingdom.
6. *Ethiopian Building Code Standard: EBCS-8 Design of Structures for Earthquake Resistance*. (1995) Ministry of Works and urban Development, Addis Ababa.
7. Eurocode8. (2003) *Design of structures for earthquake resistance Part 3: Strengthening and repair of buildings*, European Committee for Standardization, European Standard.
8. *European Standard Eurocode 8: Design of Structures for Earthquake Resistance – Part 1: General Rules, Seismic Actions and Rules for Buildings*. (2003) European Committee for Standardization, Brussels.
9. E. L. Wilson. (2002) *Three-Dimensional Static and Dynamic Analysis of Structures*, Computers and Structures, Inc., USA.
10. IDNDR RADIUS Project. (1999) *Addis Ababa Case Study, Final Report*. Addis Ababa RADIUS Group, Ethiopia.
11. *International Conference of Building Officials. (1997) Uniform Building Code*, Whittier, CA.
12. J.C. Anderson. (2000) *Seismic Design Handbook, Dynamic Response of Structures*, University of Southern California, USA.
13. M.A Marfai and J.K. Njagih. (2002) *Vulnerability Analysis And Risk Assessment For Seismic And Flood Hazard In Turrialba City, Costa Rica*, International Institute for Geo-information Sciences and Earth Observation (ITC), Enscheda Netherlands.

14. M. Saatcioglu and J. Humar. (2002) Dynamic analysis of buildings for earthquake resistant design. Proposed Earthquake Design Requirements of the NBCC, Canada.
15. M.V Sivaselvan and A.M. Reinhorn. (1999) Hysteretic Models for Cyclic Behavior of Deteriorating Inelastic Structures University at Buffalo, USA.
16. P. Bazzurro. (2009) Nonlinear Dynamic Analysis—a Step Advance in Assessing the Vulnerability of Buildings to Earthquake Ground Motion.
17. S.K Kassegne. (2006) Proposed Considerations for Revision of EBCS-8:1995 for Conservative Seismic Zoning and Stringent Requirements for Torsionally Irregular Buildings, EACE's Zede Research Journal.
18. S. Lee. (2008) Nonlinear Dynamic Earthquake Analysis of Skyscrapers by ABAQUS, CTBUH 8th World Congress.

Appendices

Appendix A - Program Verification

A comprehensive study of lightly reinforced frame structures was the subject of numerous investigations at the State University of New York at Buffalo (Bracci, 1992), and at Cornell University (El-Altar, 1990). A 1:3 scaled model was constructed, tested, retrofitted and re-tested using simulated earthquake motion generated by the shaking table at SUNY/Buffalo. The model reflects a “slice” of a long structure with three-bay frames in the transverse direction. The “slice” has two parallel lightly reinforced frames as indicated by the model representation in the plan view in Figure A.1. Essential geometrical data and reinforcement details are also shown in the figure. Attained concrete strength were 4000 psi, 3000 psi and 3500 psi at the first, second and third story levels respectively, with an elastic modulus of 2700 ksi, 2300 ksi and 2530 ksi, respectively. The steel had an average yielding strength of 65 ksi after annealing with modulus of elasticity of approximately 29000 ksi. Additional details about the structure and the testing can be found in Bracci (1992).

The model was tested by a sequence of ground (table) motions reflecting a low level earthquake (PGA=0.05g), a moderate earthquake (PGA=0.20g) and a severe earthquake (PGA=0.30g). The ground motion was obtained by scaling the acceleration time history of Taft (1952) N21E component. Only two sets of results are presented here.

The main purpose of this study was to investigate the effectiveness of using identified component properties from separate sub-assembly tests in predicting the dynamic response of the total structure. The data set used for in this example is presented in Appendix C. Only the second run at a measured peak acceleration of 0.22g is included, since the basic data is the same for both runs, with the exception of the initial stiffness and the input ground motion. As indicated, the data was derived entirely from the results of separate interior and exterior beam-column sub-assembly tests which provided information on yield strength and hysteretic behavior. No attempt was made to fit the observed shaking table response.

The comparisons of displacements for the top story during the mild and moderate earthquakes are shown in Figure A.2 and A.3. IDARC predictions show good agreement for both peak values and the total response history. The comparison includes predictions by DRAIN-2D and SARCF. More data on observed behavior in terms of deformations, stresses and damage mechanisms are reported in Bracci (1992).

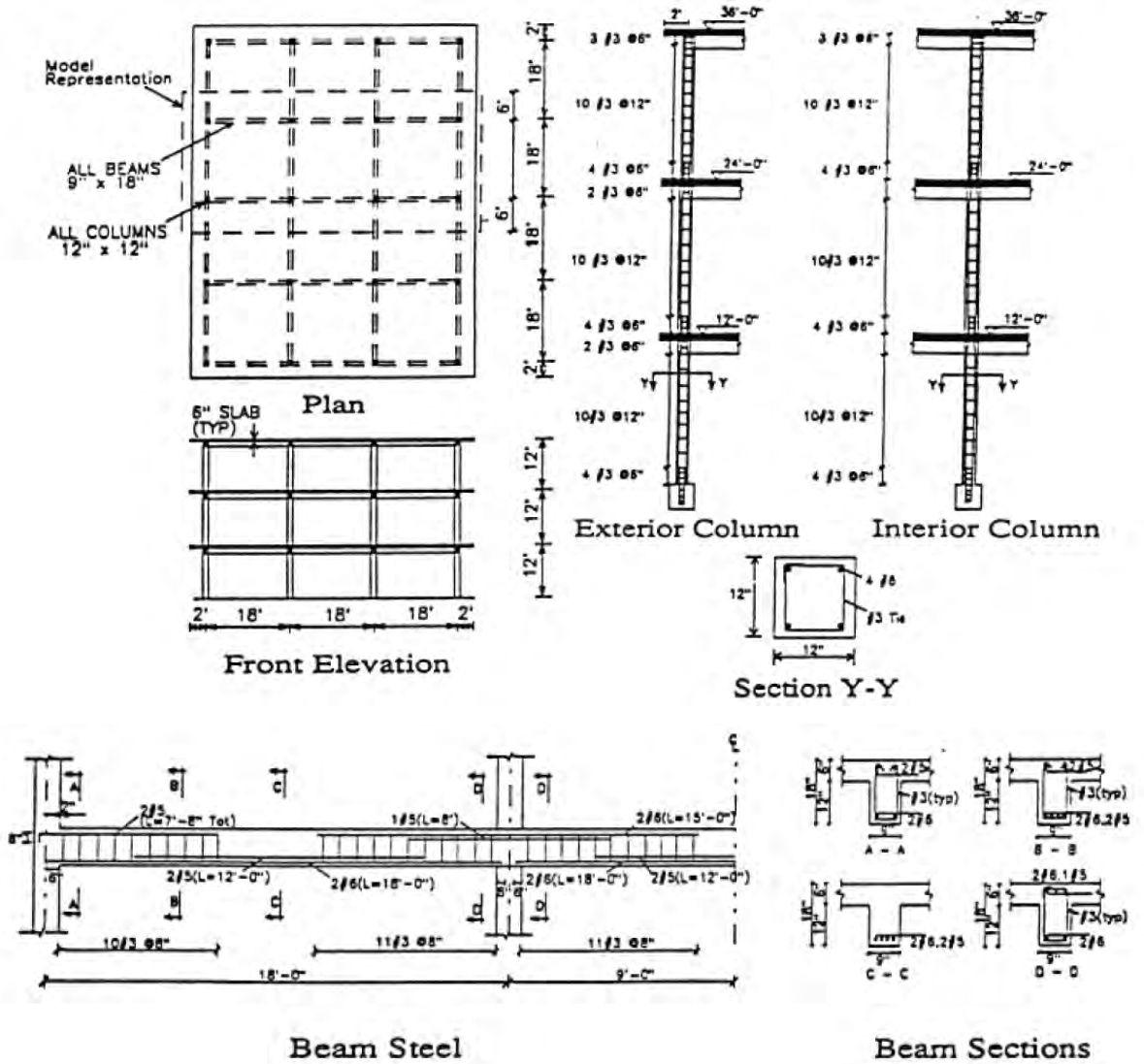


Figure A. 1

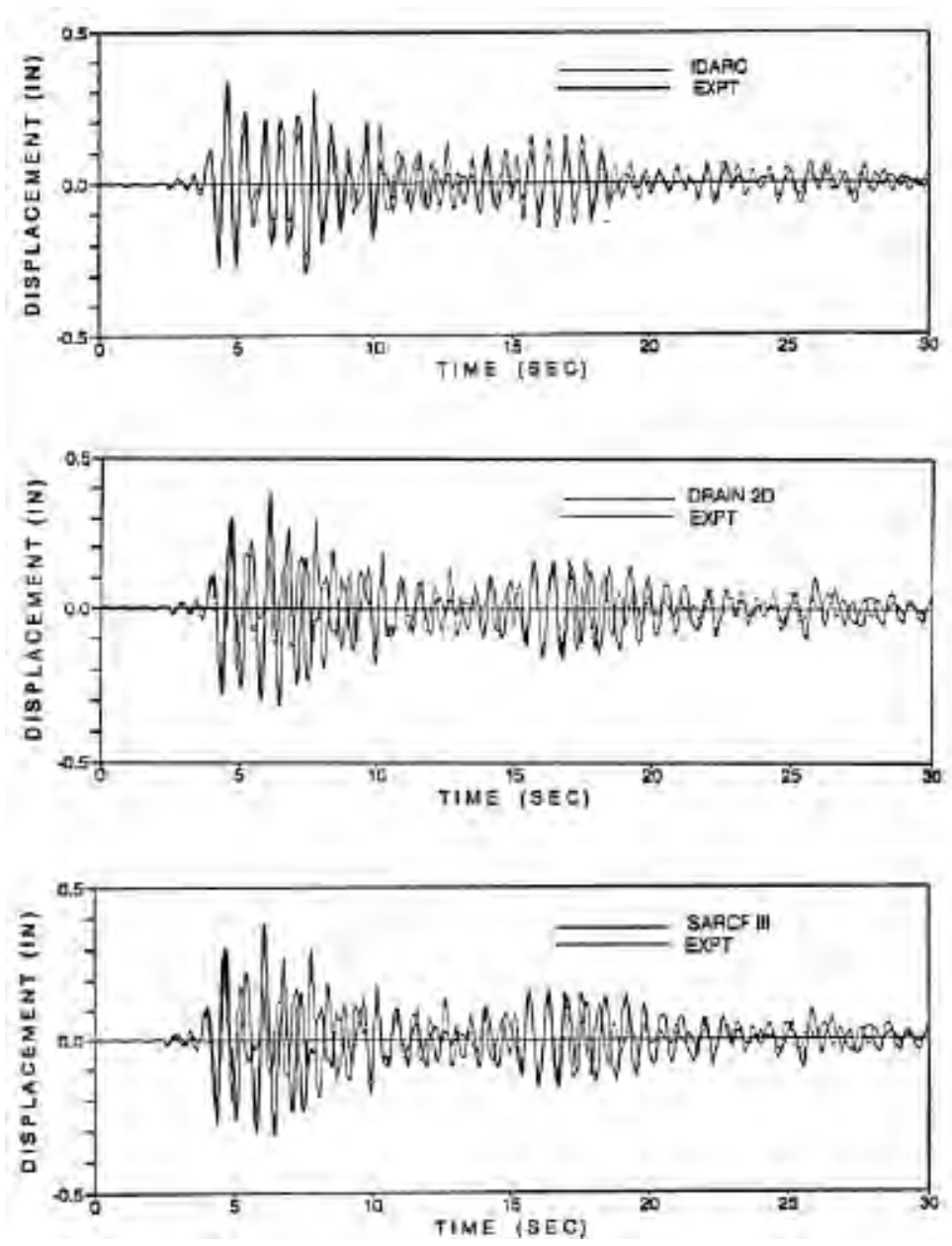


Figure A. 2 Comparison with other programs – low intensity (0.05g)

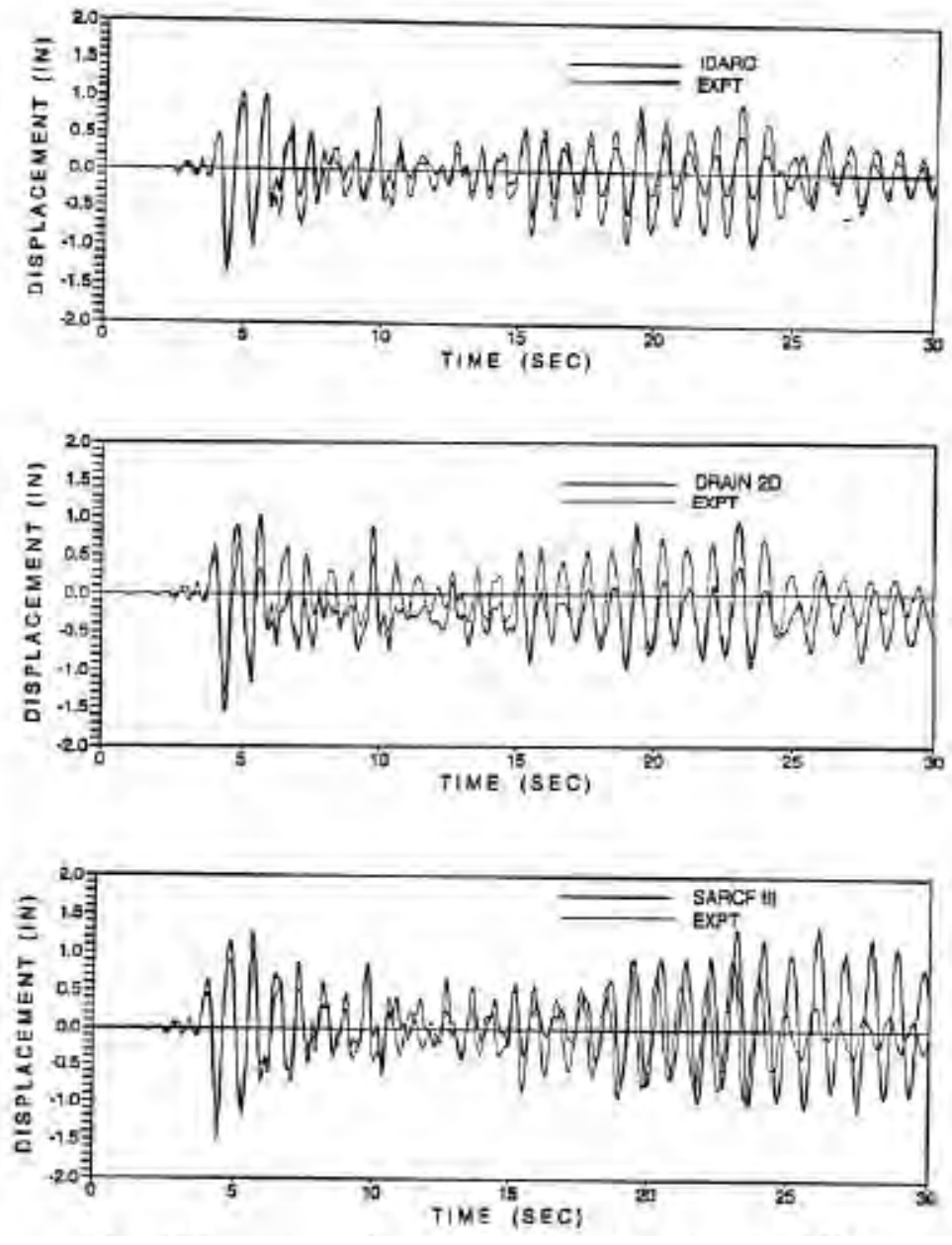


Figure A. 3 Comparison with other programs – moderate intensity (0.22g)

Appendix B - Modified Park & Ang Damage Model

The Park & Ang damage model (Park et al., 1984) was incorporated in IDARC since the original release of the program (Park et al., 1987). Furthermore, the Park & Ang damage model is also an integral part of the three parameter hysteretic model since the rate of strength degradation is directly related to the parameter β described below (Park et al., 1987).

The original Park & Ang damage index for a structural element is defined as:

$$DI_{P\&A} = \frac{\delta_m}{\delta_u} + \frac{\beta}{\delta_u P_y} \int dE_h \quad (\text{B.1})$$

Where δ_m is the maximum experienced deformation; δ_u is the ultimate deformation of the element; P_y is the yield strength of the element; $\int dE_h$ is the cumulative hysteretic energy absorbed by the element during the response history; and β is a model constant parameter. A value of 0.1 for the parameter β has been suggested for nominal strength deterioration (Park et al., 1987). The Park & Ang damage model accounts for damage due to maximum inelastic excursions, as well as damage due to the history of deformations. Both components of damage are linearly combined.

Three damage indices are computed using this damage model:

1. Element damage index: column, beams or shear wall elements.
2. Story damage index: vertical and horizontal components and total story damage.
3. Overall building damage.

Equation B.1 is the basis for the modified damage index developed for this program, although some considerations need to be taken into account as shown below.

$$DI_{P\&A} = \frac{\delta_m - \delta_y}{\delta_u - \delta_y} + \frac{\beta}{\delta_u P_y} E_h \quad (\text{B.2})$$

Where E_h is a total cumulative hysteretic energy: $\int dE_h$. Direct application of the damage model to a structural element, a story, or the overall building requires the determination of the corresponding overall element, story, or building ultimate deformations. Since the inelastic behavior is confined to plastic zones near the ends of some members, the relation between element, story or top story deformations, with the local plastic rotations is difficult to establish. For the element end section damage, the following modifications to the original model were introduced in IDARC version 3.0

(Kunnath et al., 1992b):

$$DI_{component} = \frac{\theta_m - \theta_r}{\theta_u - \theta_r} + \frac{\beta}{M_y \theta_u} E_h \quad (B.3)$$

Where θ_m is the maximum rotation attained during the loading history; θ_u is the ultimate rotation capacity of the section; θ_r is the recoverable rotation when unloading; M_y is the yield moment; and E_h is the cumulative dissipated energy in the section. The overall element damage is defined as the biggest damage index of the end sections.

Two additional damage indices are defined as follows: (i) story and (ii) overall damage indices which are computed using weighting factors based on the hysteretic energy dissipated at component and story levels respectively:

(i) Story Damage

$$DI_{story} = \sum (\lambda_i)_{component} (DI_i)_{component}$$

$$\text{where } (\lambda_i)_{component} = \left(\frac{E_i}{\sum E_i} \right)_{component} \quad (B.4)$$

(ii) Overall Damage

$$DI_{overall} = \sum (\lambda_i)_{story} (DI_i)_{story} \quad (B.5)$$

Where

$$(\lambda_i)_{story} = \left(\frac{E_i}{\sum E_i} \right)_{story}$$

Where λ_i are the energy weighting factors; and E_i are the cumulative energy absorbed by the component or story “i”.

The Park & Ang damage model has been calibrated using a database of observed structural damage of nine reinforced concrete buildings (Park et al., 1986). Table B.1 presents the calibrated damage index with the degree of observed damage in the structure.

Table B.1 Interpretation of overall damage index (Park et al., 1986)

LIMIT STATE DAMAGE INDEX	DEGREE OF DAMAGE	DAMAGE (SERVICE) STATE	USABILITY	APPEARANCE
(1)	(2)	(3)	(4)	(5)
0.00	None	Undamaged	Usable	Undeformed / uncracked
0.20-0.30	Slight	Serviceable		Moderate to severe cracking
0.50-0.60	Minor Moderate	Repairable	Temporarily Unusable	Spalling of concrete cover
> 1.0	Severe Collapse	Unrepairable Collapse	Unusable	Buckled bars, exposed core Loss of shear/axial capacity

Appendix C - Input for Program Verification

Analysis of 1:3 Scale Three Story Model 0.2g

Control Data

3,1,0,0,0,0,0,1,1

Element types

6,1,0,0,0,0,0,0,0

Element data

12,9,0,0,0,0,0,0,0

Unit system

1

Floor elevations

45.0, 93.0, 141.0

Number of duplicate frames

2

No of column lines

4

Nodal weights

1, 1, 3.375, 3.375, 3.375, 3.375

2, 1, 3.375, 3.375, 3.375, 3.375

3, 1, 3.375, 3.375, 3.375, 3.375

Env generation option

1

Hysteretic Control

2

1, 1, 10.0, 0.01, 0.08, 1.0, 0

2, 1, 15.0, 0.01, 0.08, 1.0, 0

Column input option

1

Column data

1

1, 10, 20, 50, 48.0,3.0,3.0

1, 45400.0, 843.0, 10.0, 18.0, 0.00200, 0.006, 0.88
10.0, 18.0, 0.00200, 0.006, 0.88

1, 45400.0, 843.0, 10.0, 18.0, 0.00200, 0.006, 0.88
10.0, 18.0, 0.00200, 0.006, 0.88

1

2, 10, 20, 50, 48.0,3.0,3.0

1, 45400.0, 843.0, 10.0, 22.0, 0.00200, 0.006, 0.88
10.0, 22.0, 0.00200, 0.006, 0.88

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10.0, 22.0, 0.00200, 0.006, 0.88

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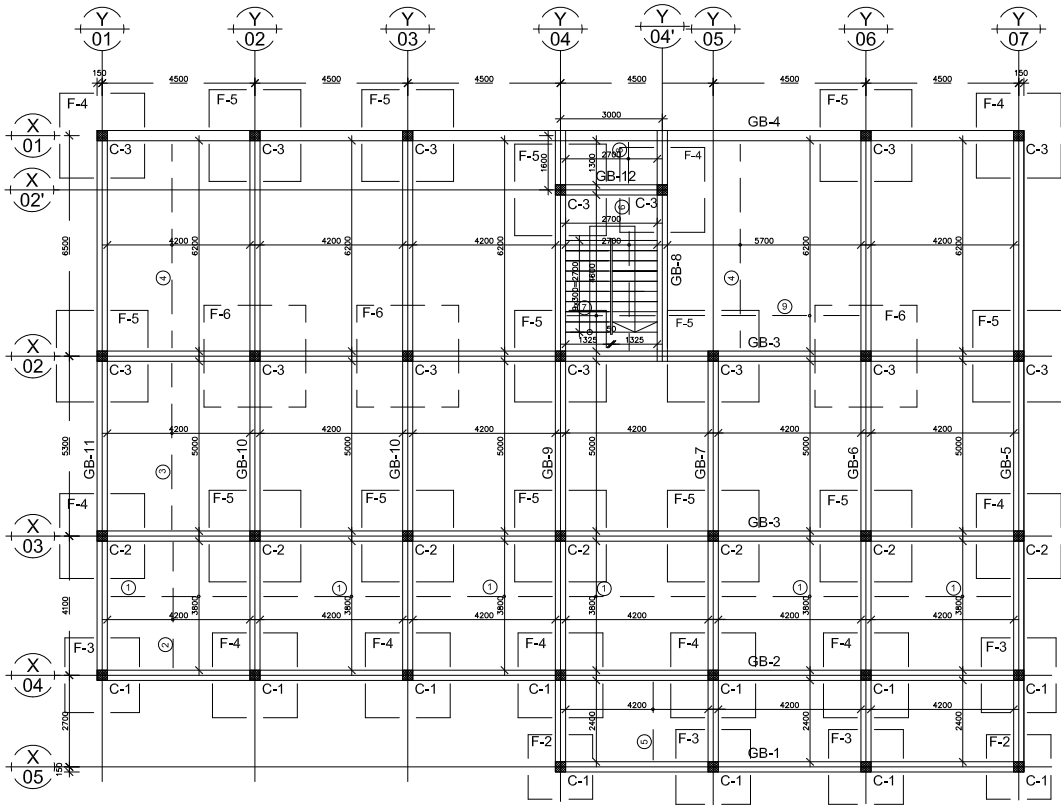
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 10.0, 22.0, 0.003, 0.006, 0.87
 1
 4, 10, 20, 50, 48.0,3.0,3.0
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 14.0, 29.0, 0.003, 0.006, 0.87
 1, 45900.0, 900.0, 14.0, 29.0, 0.003, 0.006, 0.87
 14.0, 29.0, 0.003, 0.006, 0.87
 1
 5, 10, 20, 50, 45.0,0.0,3.0
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 12.0, 28.0, 0.003, 0.008, 0.88
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 12.0, 28.0, 0.003, 0.008, 0.88
 1
 6, 10, 20, 50, 45.0,0.0,3.0
 1, 45200.0, 960.0, 16.0, 38.0, 0.003, 0.008, 0.88
 16.0, 38.0, 0.003, 0.008, 0.88
 1, 45200.0, 960.0, 16.0, 38.0, 0.003, 0.008, 0.88
 16.0, 38.0, 0.003, 0.008, 0.88
 Beam input type
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 Beam data
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 30.0, 70.0, 0.001, 0.01, 1.71
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 3,2,1,3,2,3
 4,1,1,4,2,3
 5,3,1,1,1,2
 6,4,1,2,1,2
 7,4,1,3,1,2
 8,3,1,4,1,2
 9,5,1,1,0,1
 10,6,1,2,0,1
 11,6,1,3,0,1
 12,5,1,4,0,1
 Beam connectivity
 1,1,3,1,1,2
 2,1,3,1,2,3
 3,1,3,1,3,4

4,1,2,1,1,2
5,1,2,1,2,3
6,1,2,1,3,4
7,1,1,1,1,2
8,1,1,1,2,3
9,1,1,1,3,4
Type of Analysis
3
Static loads
0,0,0,0
Dynamic Analysis Control Data
0.2, 0.0, 0.001, 30.0, 1.2, 3
Wave data
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TAFT - EARTHQUAKE
wave05.dat
SNAPSHOT CONTROL DATA
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0,0,0,0,0
Output options
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JELAS.PRN
Hys output
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Appendix D - Case Study Structural Drawings



GROUND FLOOR PLAN AT ±0.00

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DEPARTMENT OF CIVIL ENGINEERING

PROJECT:
ASSESSMENT OF SEISMIC VULNERABILITY
OF BUILDINGS; A CASE STUDY OF SELECTED
BUILDINGS CONSTRUCTED IN ADDIS ABABA

LOCATION:
ADDIS ABABA, ETHIOPIA

Rev./No. Date Changes OR Remark By

NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...

CASE STUDY: **Case Study # 1: Six Story-Building Ribbed Slab System with One Way Framing**

DRAWING NUMBER: **ST-01**

DRAWING TITLE: **GROUND FLOOR REINFORCEMENT AND FOUNDATION LAYOUT**

PROJECT STATUS:

FINAL DESIGN CONSTRUCTION UNDER SERVICE

DESIGNED BY:

DISCLOSED

DATE: 05-MAR-2010

Checked by:

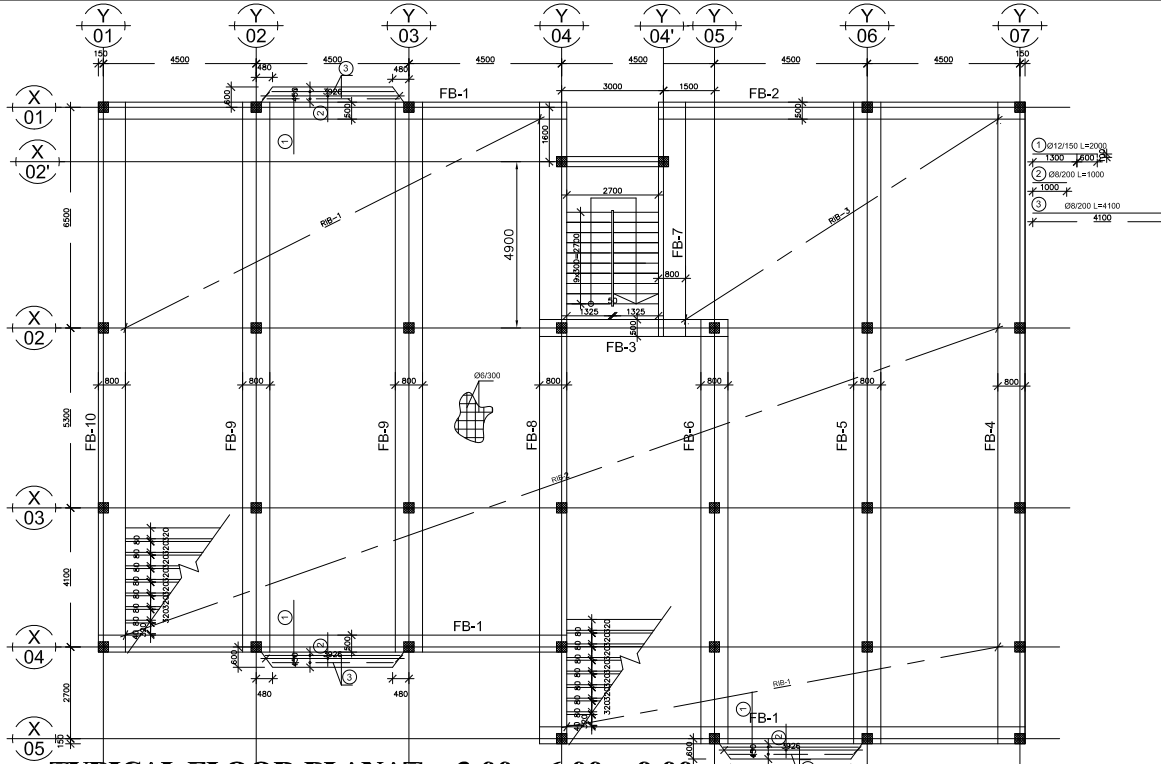
Daniel Getachew

ADVISOR

Adil Zekaria, Dr.-Ing

DATE: JAN-2011

REVISION SHEET #:



TYPICAL FLOOR PLAN AT +3.00, +6.00, +9.00

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

PROJECT:
ASSESSMENT OF SEISMIC VULNERABILITY
OF BUILDINGS; A CASE STUDY OF SELECTED
BUILDINGS CONSTRUCTED IN ADDIS ABABA

LOCATION:
ADDIS ABABA, ETHIOPIA

Rev./No. Date Changes OR Remark By

NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...

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DRAWING NUMBER: **ST-02**

DRAWING TITLE: **TYPICAL FLOOR PLAN AT +3.00, +6.00, +9.00**

PROJECT STATUS:

FINAL DESIGN CONSTRUCTION UNDER SERVICE

DESIGNED BY:

DISCLOSED

DATE: 05-MAR-2010

Checked by:

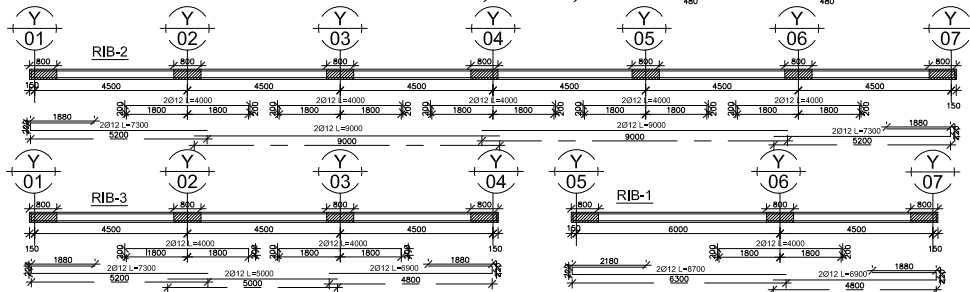
Daniel Getachew

ADVISOR

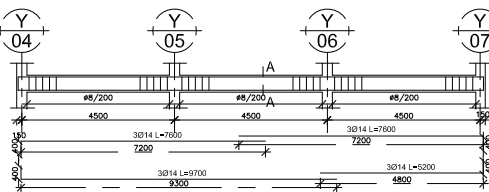
Adil Zekaria, Dr.-Ing

DATE: JAN-2011

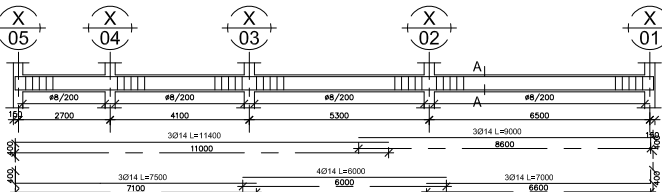
REVISION SHEET #:



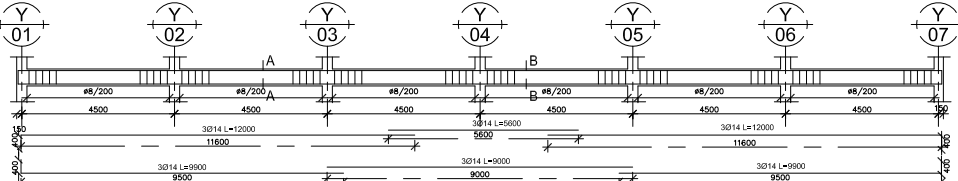
GB-1 Ground floor beam Y4-Y5-Y6-Y7 on axis X5



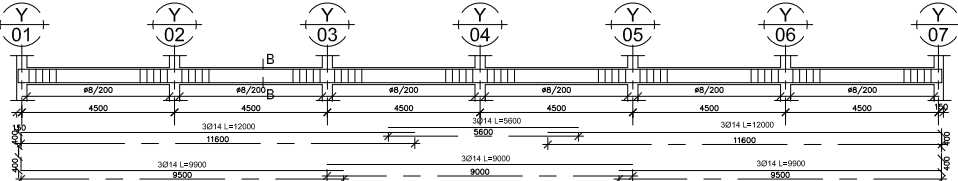
GB-5 Ground floor beam X5-X4-X3-X2-X1 on axis Y7



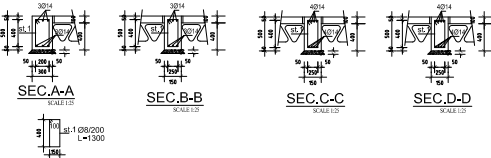
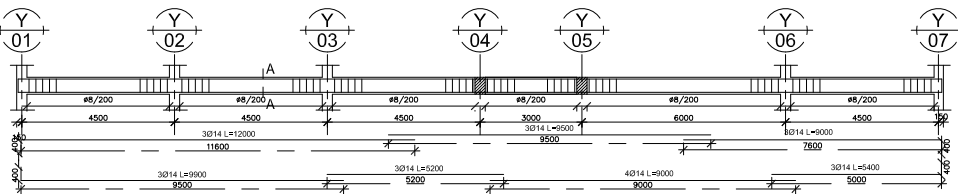
GB-2 Ground floor beam Y1-Y2-Y3-Y4-Y5-Y6-Y7 on axis X4



GB-3 Ground floor beam Y1-Y2-Y3-Y4-Y5-Y6-Y7 on axes X3 & X2



GB-4 Ground floor beam Y1-Y2-Y3-Y4-Y5-Y6-Y7 on axes X3 & X2



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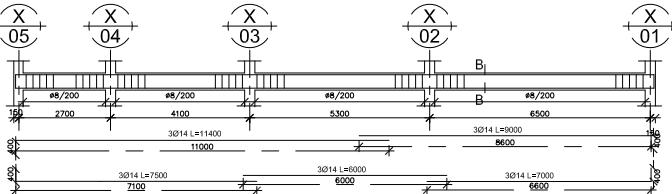
PROJECT:
ASSESSMENT OF SEISMIC VULNERABILITY
OF BUILDINGS; A CASE STUDY OF SELECTED
BUILDINGS CONSTRUCTED IN ADDIS ABABA

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ADDIS ABABA, ETHIOPIA

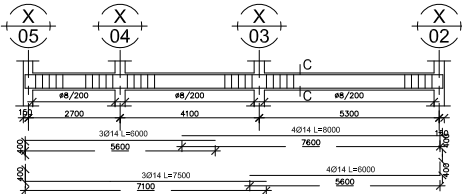
Rev./No.	Date	Changes OR Remark	By
NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...			
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DESIGNED BY: DISCLOSED		Checked by: Daniel Getachew	
DATE: 05-MAR-2019		ADVISOR: Adil Zekaria, Dr.-Ing	
REVISION SHEET #:		DATE: JAN-2011	
A		A	

GROUND FLOOR BEAMS
SCALE: 1/50

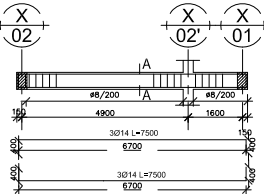
GB-6 Ground floor beam X5-X4-X3-X2-X1 on axis Y6



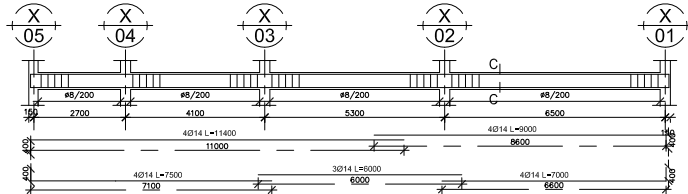
GB-7 Ground floor beam X1-X2-X3-X4-X5 on axis Y5



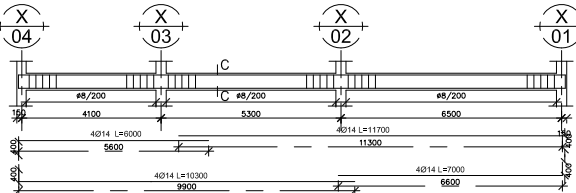
GB-8 Ground floor beam X2-X2-X1 on axis Y4



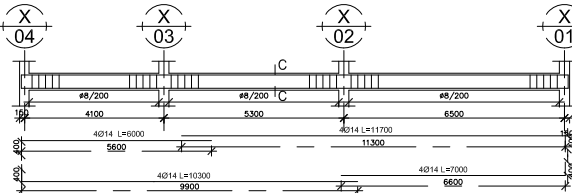
GB-9 Ground floor beam X5-X4-X3-X2-X1 on axis Y4



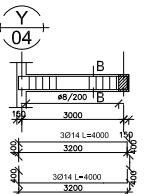
GB-10 Ground floor beam X4-X3-X2-X1 on axis Y2 & Y3



GB-11 Ground floor beam X4-X3-X2-X1 on axis Y1



GB-12 Ground floor beam Y4



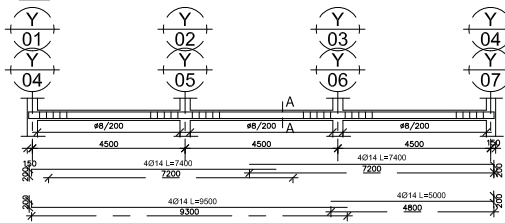
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DEPARTMENT OF CIVIL ENGINEERING

PROJECT:
ASSESSMENT OF SEISMIC VULNERABILITY
OF BUILDINGS; A CASE STUDY OF SELECTED
BUILDINGS CONSTRUCTED IN ADDIS ABABA

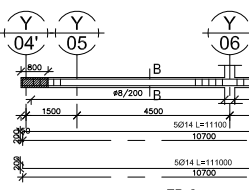
LOCATION:
ADDIS ABABA, ETHIOPIA

Rev./No.	Date	Changes OR Remark	By
NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...			
CASE STUDY: Case Study # 1: Six Story-Building Ribbed Slab System with One Way Framing			
DRAWING NUMBER: ST-04		PROJECT STATUS:	
DRAWING TITLE: GROUND FLOOR BEAM DETAILS (2)		<input type="radio"/> FINAL DESIGN <input type="radio"/> CONSTRUCTION UNDER SERVICE	
DESIGNED BY: DISCLOSED		Checked by: Daniel Getachew	
DATE: 05-MAR-2019		ADVISOR: Adil Zekaria, Dr.-Ing	
REVISION SHEET #:		DATE: JAN-2011	
A		A	

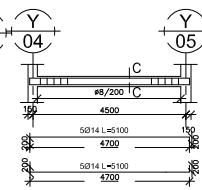
FB-1 Floor beam Y1-Y2-Y3-Y4 & Y4-Y5-Y6-Y7 on axes X1,X4 & X5



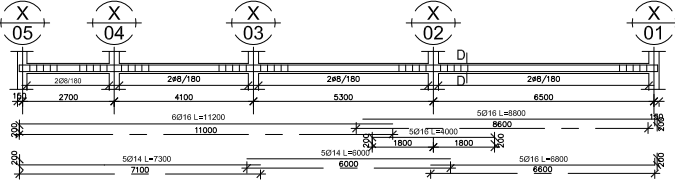
FB-2 Floor beam Y4-Y5-Y6-Y7 on axis X1



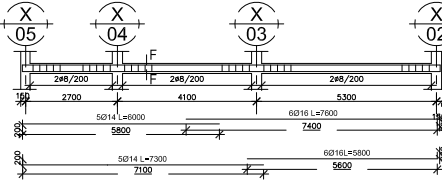
FB-3 Floor beam Y4-Y5 on axis X5



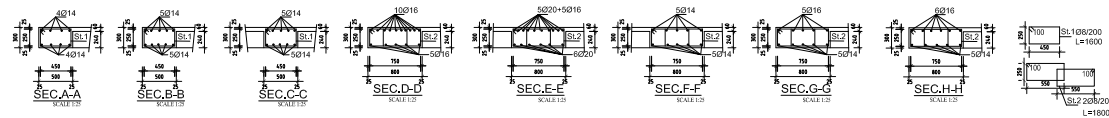
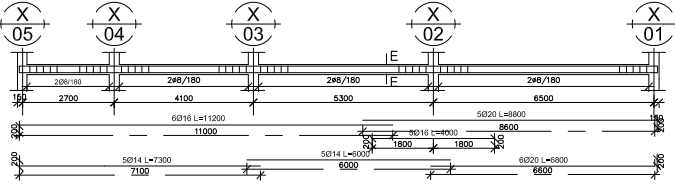
FB-4 Floor beam X5-X4-X3-X2-X1 on axis Y7



FB-6 Floor beam X5-X4-X3-X2 on axis Y5



FB-5 Floor beam X5-X4-X3-X2-X1 on axis Y6



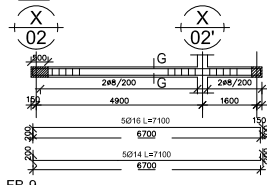
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SCHOOL OF GRADUATE STUDIES
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DEPARTMENT OF CIVIL ENGINEERING

PROJECT:
ASSESSMENT OF SEISMIC VULNERABILITY
OF BUILDINGS; A CASE STUDY OF SELECTED
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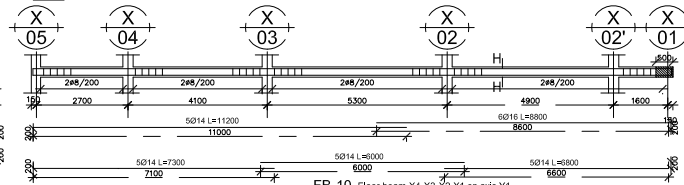
LOCATION:
ADDIS ABABA, ETHIOPIA

Rev./No.	Date	Changes OR Remark	By
NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...			
CASE STUDY: Case Study # 1: Six Story-Building Ribbed Slab System with One Way Framing			
DRAWING NUMBER: ST-05		DRAWING TITLE: TYPICAL FLOOR BEAM DETAILS (1)	
PROJECT STATUS: FINAL DESIGN: <input type="radio"/> CONSTRUCTION: <input type="radio"/> UNDER SERVICE: <input checked="" type="radio"/>		Checked by: Daniel Getachew ADVISOR	
DESIGNED BY: DISCLOSED		ADVISOR: Adil Zekaria, Dr.-Ing	
DATE: 05-MAR-2010	REVISION SHEET #:	DATE: JAN-2011	REVISION SHEET #:
	A		A

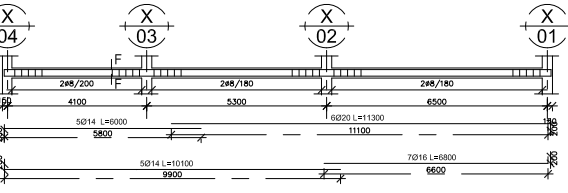
FB-7 Floor beam X2-X1 on axis Y4'



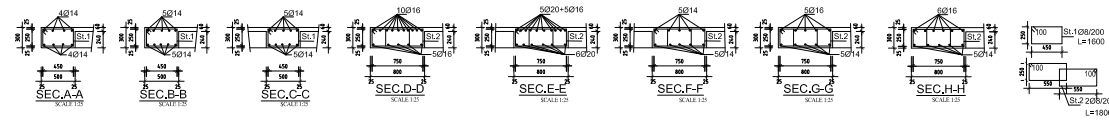
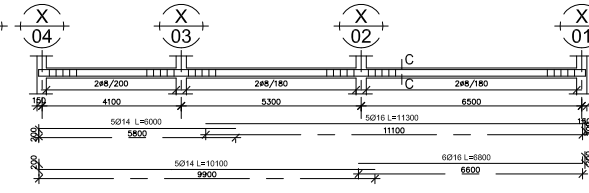
FB-8 Floor beam X5-X4-X3-X2-X1 on axis Y4



FB-9



FB-10 Floor beam X4-X3-X2-X1 on axis Y1



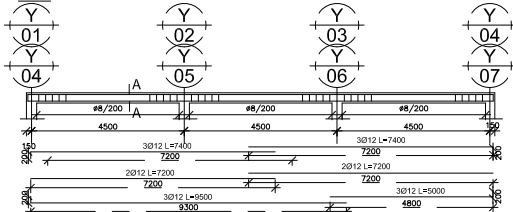
ADDIS ABABA UNIVERSITY
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INSTITUTION OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING

PROJECT:
ASSESSMENT OF SEISMIC VULNERABILITY
OF BUILDINGS; A CASE STUDY OF SELECTED
BUILDINGS CONSTRUCTED IN ADDIS ABABA

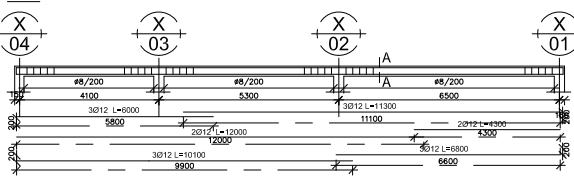
LOCATION:
ADDIS ABABA, ETHIOPIA

Rev./No.	Date	Changes OR Remark	By
NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...			
CASE STUDY: Case Study # 1: Six Story-Building Ribbed Slab System with One Way Framing			
DRAWING NUMBER: ST-06		DRAWING TITLE: TYPICAL FLOOR BEAM DETAILS (2)	
PROJECT STATUS: FINAL DESIGN: <input type="radio"/> CONSTRUCTION: <input type="radio"/> UNDER SERVICE: <input checked="" type="radio"/>		Checked by: Daniel Getachew ADVISOR	
DESIGNED BY: DISCLOSED		ADVISOR: Adil Zekaria, Dr.-Ing	
DATE: 05-MAR-2010	REVISION SHEET #:	DATE: JAN-2011	REVISION SHEET #:
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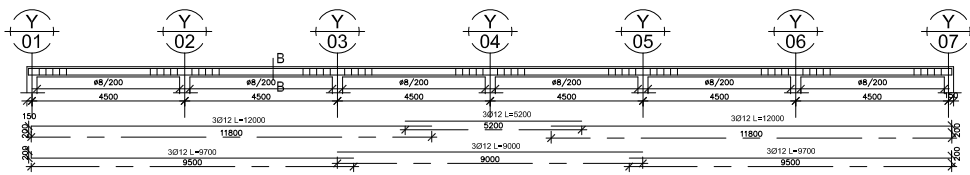
RB-1 Roof beam Y1-Y2-Y3-Y4 & Y4-Y5-Y6-Y7 on axes X4 & X5



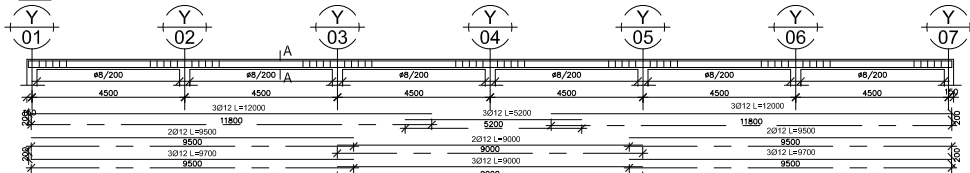
RB-6 Roof beam X4-X3-X2-X1 on axis Y1



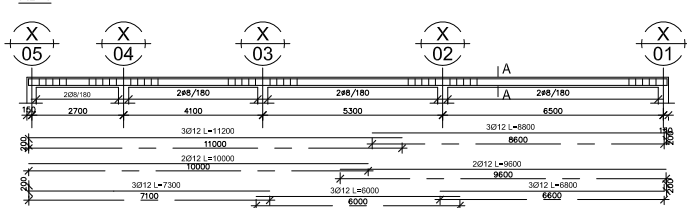
RB-2 Roof beam Y1-Y2-Y3-Y4-Y5-Y6-Y7 on axis X2



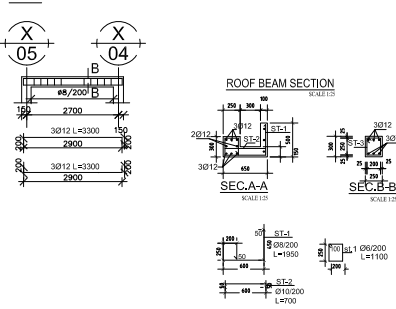
RB-3 Roof beam Y1-Y2-Y3-Y4-Y5-Y6-Y7 on axis X1



RB-4 Floor beam X5-X4-X3-X2-X1 on axis Y7



RB-5 Roof beam X5-X4 on axis Y4

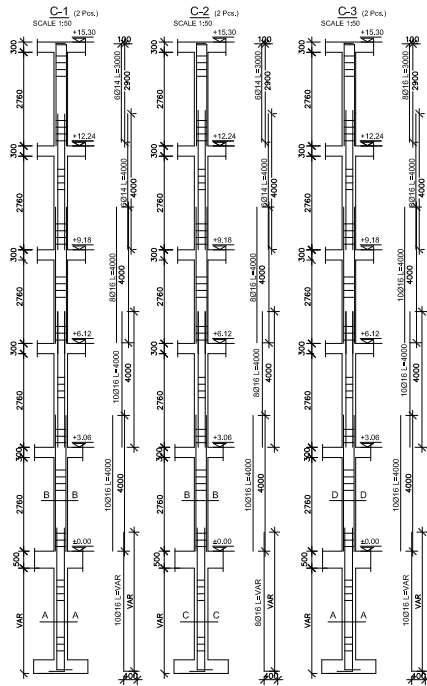


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DEPARTMENT OF CIVIL ENGINEERING

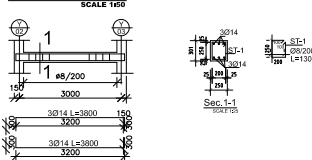
ASSESSMENT OF SEISMIC VULNERABILITY
OF BUILDINGS: A CASE STUDY OF SELECTED
BUILDINGS CONSTRUCTED IN ADDIS ABABA

ADDIS ABABA, ETHIOPIA

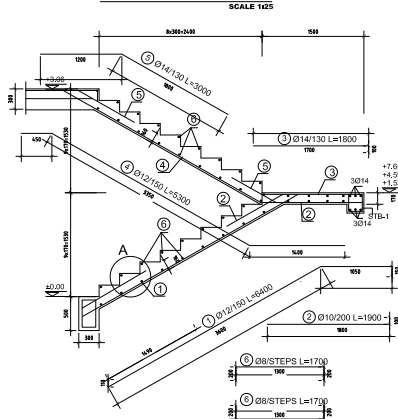
Rev./No.	Date	Changes OR Remark	By
NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...			
CASE STUDY: Case Study # 1: Six Story-Building Ribbed Slab System with One Way Framing			
DRAWING NUMBER: ST-07		DRAWING TITLE: ROOF FLOOR BEAM DETAILS	
PROJECT STATUS: FINAL DESIGN: <input type="radio"/> CONSTRUCTION: <input type="radio"/> UNDER SERVICE: <input checked="" type="radio"/>		Checked by: Daniel Getachew ADVISOR	
DESIGNED BY: DISCLOSED		DATE: ADIL ZEKARIA, DR.-ING	
DATE: 05-MAR-2010	REVISION SHEET #: A	DATE: JAN-2011	REVISION SHEET #: A



STAIR CASE BEAM-1 SCALE 1/20



STAIR CASE SECTION SCALE 1/25

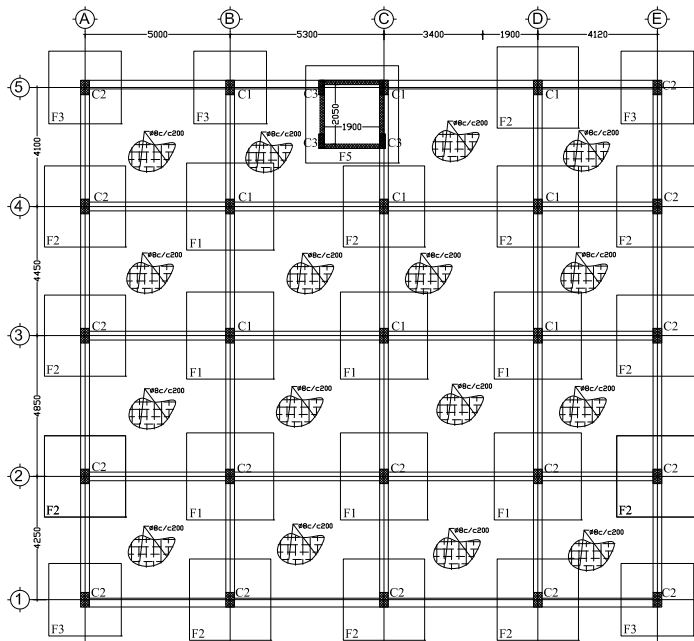


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ADDIS ABABA, ETHIOPIA

Rev./No.	Date	Changes OR Remark	By
NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...			
CASE STUDY: Case Study # 1: Six Story-Building Ribbed Slab System with One Way Framing			
DRAWING NUMBER: ST-08		DRAWING TITLE: COLUMN AND STAIR DETAILS	
PROJECT STATUS: FINAL DESIGN: <input type="radio"/> CONSTRUCTION: <input type="radio"/> UNDER SERVICE: <input checked="" type="radio"/>		Checked by: Daniel Getachew ADVISOR	
DESIGNED BY: DISCLOSED		DATE: ADIL ZEKARIA, DR.-ING	
DATE: 05-MAR-2010	REVISION SHEET #: A	DATE: JAN-2011	REVISION SHEET #: A



Rev./No.	Date	Changes OR Remark	By

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BUILDING TYPE:
**Case Study # 2: Eight Story-Building
Ribbed Slab Floor System**

DRAWING NUMBER: **ST-01**
DRAWING TITLE:
FOUNDATION AND BASEMENT PLAN

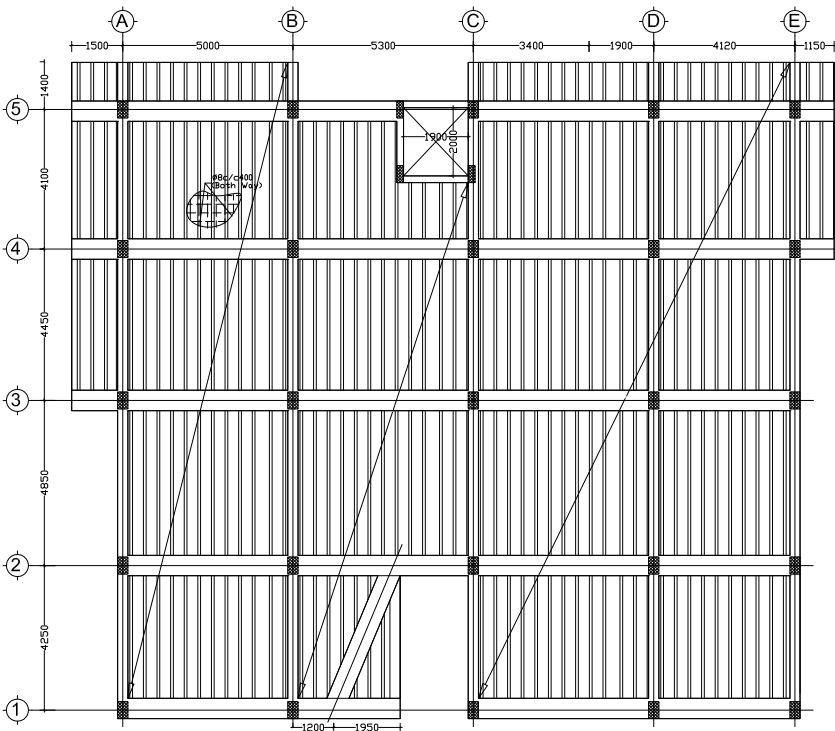
PROJECT STATUS:
 FINAL DESIGN
 CONSTRUCTION
 UNDER SERVICE

Checked by:
Daniel Getachew
ADVISOR

DESIGNED BY:
DISCLOSED
DATE: **05-MAR-2010**

ADVISOR:
Adil Zekaria, Dr.-Ing
DATE: **JAN-2011**

FOUNDATION AND BASEMENT PLAN AT -2.90 & -4.70



Rev./No.	Date	Changes OR Remark	By

NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...

BUILDING TYPE:
**Case Study # 2: Eight Story-Building
Ribbed Slab Floor System**

DRAWING NUMBER: **ST-02**
DRAWING TITLE:
FOUNDATION AND BASEMENT PLAN

PROJECT STATUS:
 FINAL DESIGN
 CONSTRUCTION
 UNDER SERVICE

Checked by:
Daniel Getachew
ADVISOR

DESIGNED BY:
DISCLOSED
DATE: **05-MAR-2010**

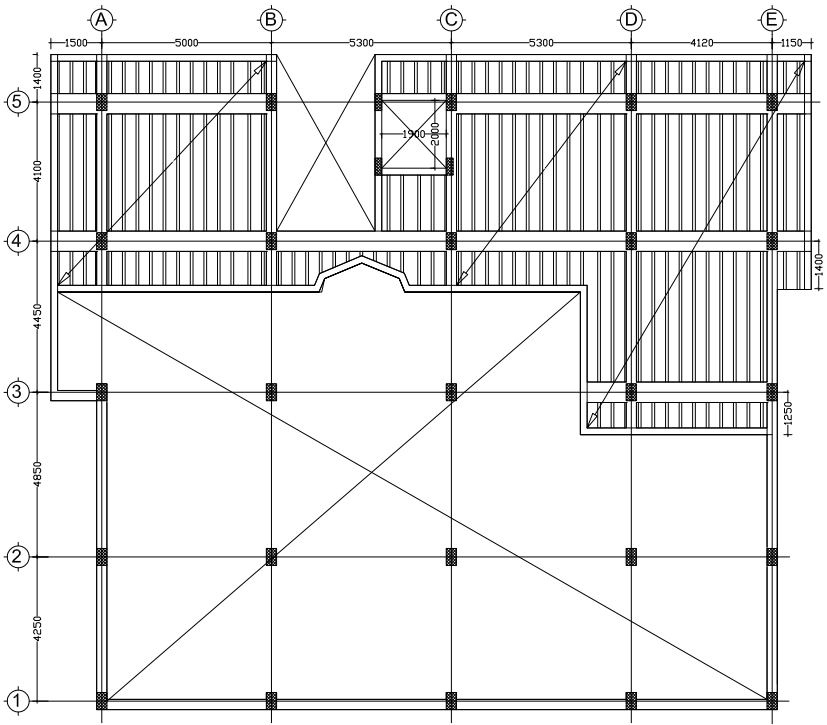
ADVISOR:
Adil Zekaria, Dr.-Ing
DATE: **JAN-2011**

GROUND FLOOR PLAN ±0.00

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PROJECT:
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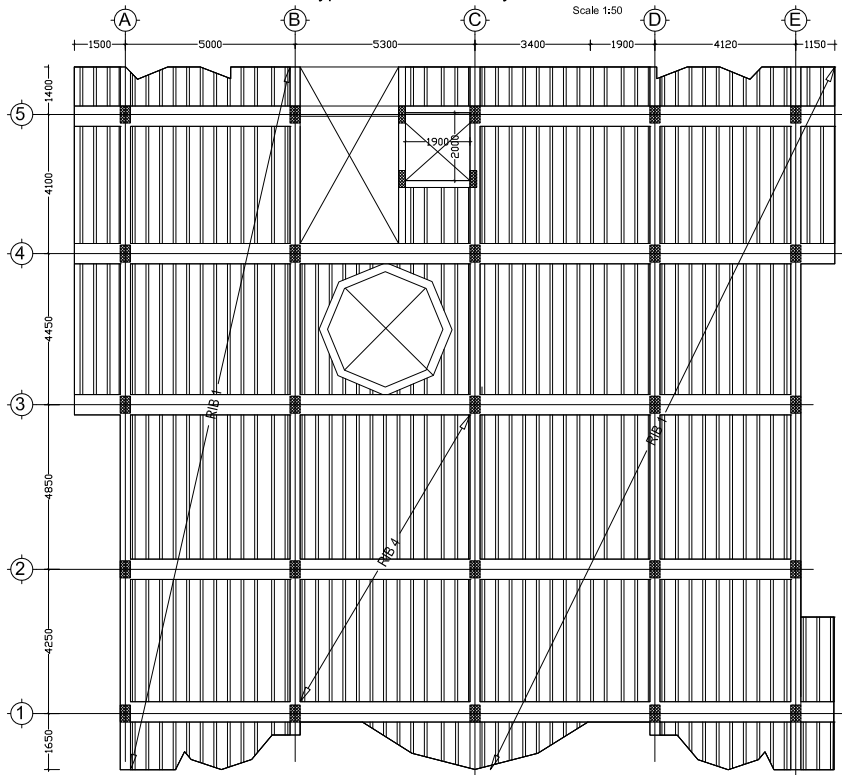
LOCATION:
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MEZZANINE FLOOR PLAN +3.00

Rev./No.	Date	Changes OR Remark	By
NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...			
BUILDING TYPE: Case Study # 2: Eight Story-Building Ribbed Slab Floor System			
DRAWING NUMBER: ST-03		DRAWING TITLE: MEZZANINE FLOOR PLAN	
PROJECT STATUS: <input type="radio"/> FINAL DESIGN <input type="radio"/> CONSTRUCTION UNDER SERVICE <input checked="" type="radio"/> DISCLOSED		Checked by: Daniel Getachew ADVISOR	
DESIGNED BY: DISCLOSED		ADVISOR: Adil Zekaria, Dr.-Ing	
DATE: 05-MAR-2010	REVISION SHEET #: A	DATE: JAN-2011	REVISION SHEET #: A

Typical Floor Beam Layout Plan



TYPICAL FLOOR(1ST TO 4TH) PLAN +6.00,+9.00,+12.00,15.00

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PROJECT:
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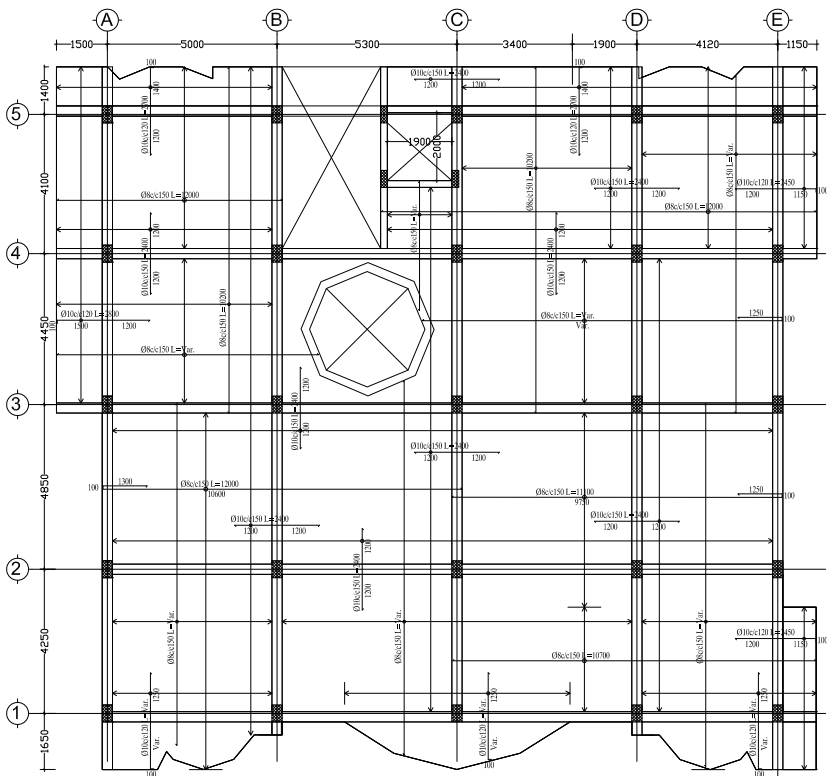
LOCATION:
ADDIS ABABA, ETHIOPIA

Rev./No.	Date	Changes OR Remark	By
NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...			
BUILDING TYPE: Case Study # 2: Eight Story-Building Ribbed Slab Floor System			
DRAWING NUMBER: ST-04		DRAWING TITLE: TYPICAL FLOOR(1ST TO 4TH) PLAN	
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DESIGNED BY: DISCLOSED		ADVISOR: Adil Zekaria, Dr.-Ing	
DATE: 05-MAR-2010	REVISION SHEET #: A	DATE: JAN-2011	REVISION SHEET #: A

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INSTITUTION OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING

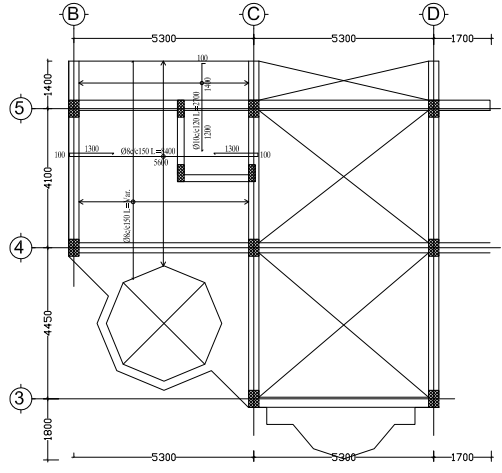
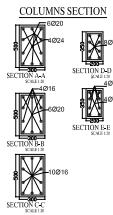
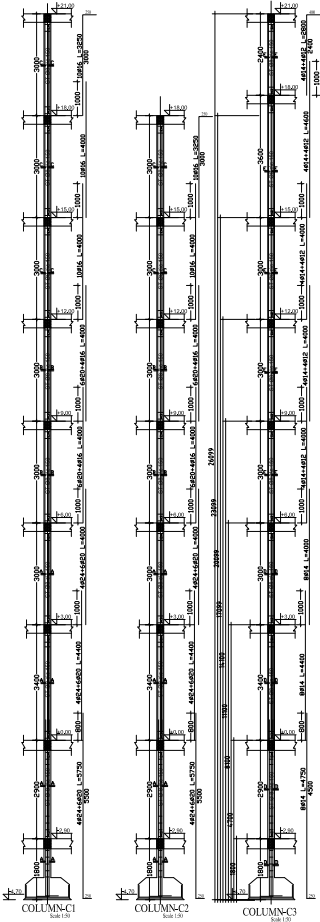
PROJECT:
ASSESSMENT OF SEISMIC VULNERABILITY
OF BUILDINGS: A CASE STUDY OF SELECTED
BUILDINGS CONSTRUCTED IN ADDIS ABABA

LOCATION:
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ROOF FLOOR PLAN +18.00

Rev./No.	Date	Changes OR Remark	By
NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...			
BUILDING TYPE: Case Study # 2: Eight Story-Building Ribbed Slab Floor System			
DRAWING NUMBER: ST-05		DRAWING TITLE: ROOF FLOOR PLAN	
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DESIGNED BY: DISCLOSED		ADVISOR: Adil Zekaria, Dr.-Ing	
DATE: 05-MAR-2010	REVISION SHEET #:	DATE: JAN-2011	REVISION SHEET #:
	A		A



LIFT SHAFT ROOF FLOOR PLAN +21.00

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DEPARTMENT OF CIVIL ENGINEERING

PROJECT:
ASSESSMENT OF SEISMIC VULNERABILITY
OF BUILDINGS: A CASE STUDY OF SELECTED
BUILDINGS CONSTRUCTED IN ADDIS ABABA

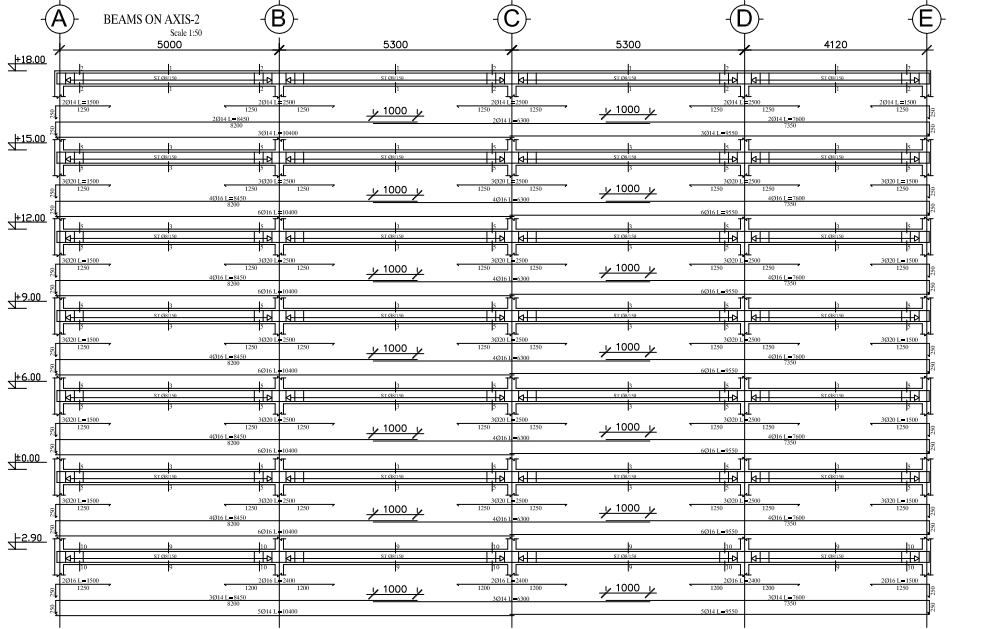
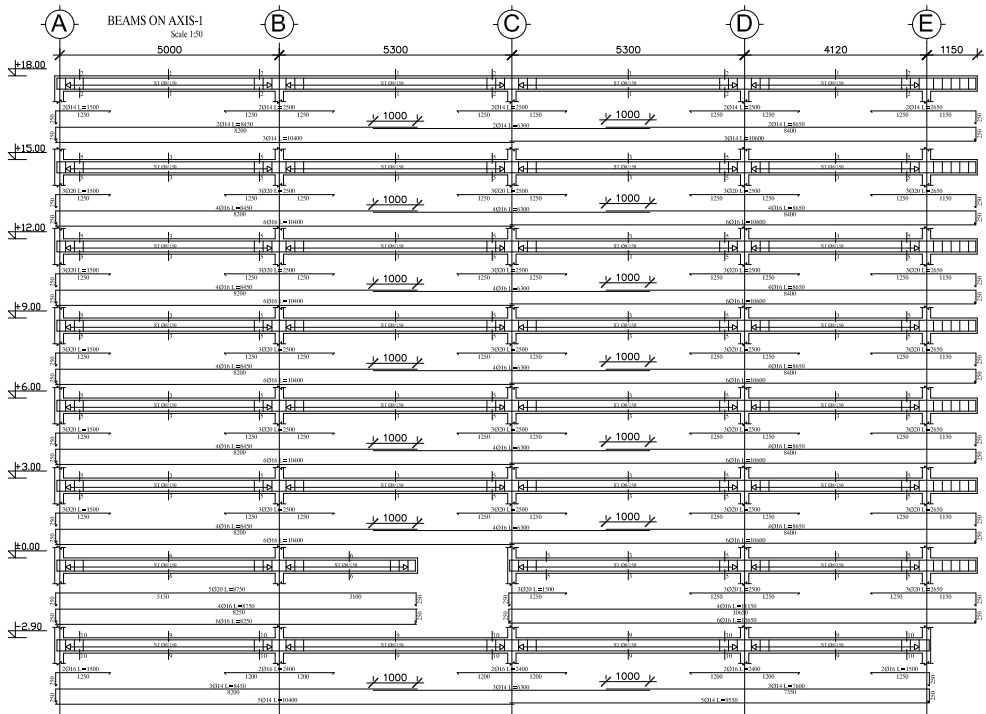
LOCATION:
ADDIS ABABA, ETHIOPIA

Rev./No.	Date	Changes OR Remark	By
NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...			
BUILDING TYPE: Case Study # 2: Eight Story-Building Ribbed Slab Floor System			
DRAWING NUMBER: ST-06		DRAWING TITLE: LIFT SHAFT ROOF FLOOR PLAN COLUMN DETAILS	
PROJECT STATUS:		Checked by:	
<input type="radio"/> FINAL DESIGN	<input type="radio"/> CONSTRUCTION	<input checked="" type="radio"/> UNDER SERVICE	Daniel Getachew
DESIGNED BY: DISCLOSED		ADVISOR: Adil Zekaria, Dr.-Ing	
DATE: 05-MAR-2010	REVISION SHEET #:	DATE: JAN-2011	REVISION SHEET #:
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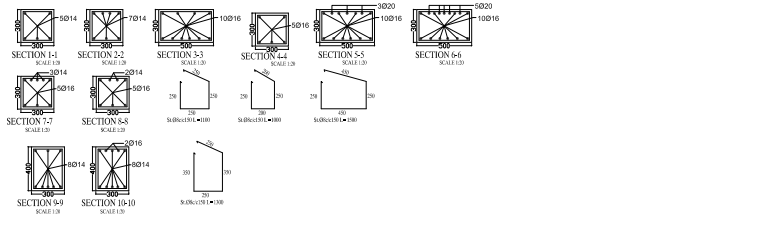
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DEPARTMENT OF CIVIL ENGINEERING

PROJECT:
ASSESSMENT OF SEISMIC VULNERABILITY
OF BUILDINGS: A CASE STUDY OF SELECTED
BUILDINGS CONSTRUCTED IN ADDIS ABABA

LOCATION:
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BEAMS SECTION



Rev.No:	Date:	Changes OR Remark:	By:
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NOTE : ALL DIMENSIONS ARE IN MM. UNLESS OTHERWISE STATED. ...

BUILDING TYPE: **Case Study # 2: Eight Story-Building Ribbed Slab Floor System**

DRAWING NUMBER: **ST-07**
DRAWING TITLE: **BEAM DETAILS (1 OF 4)**

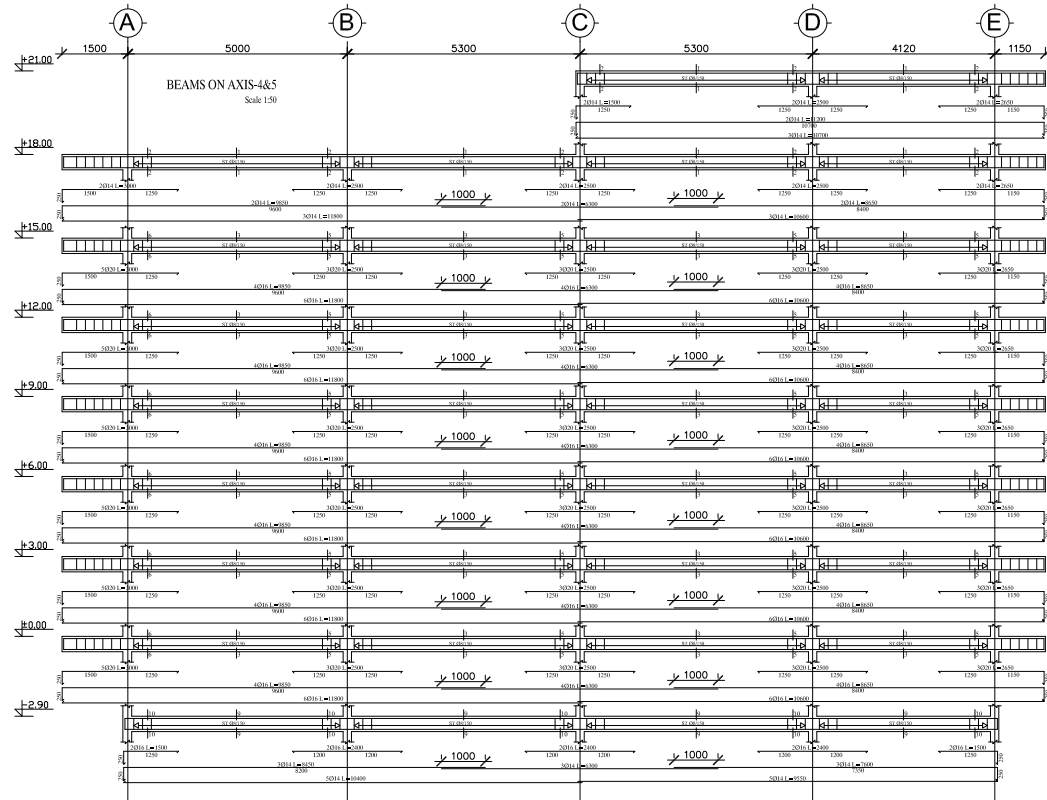
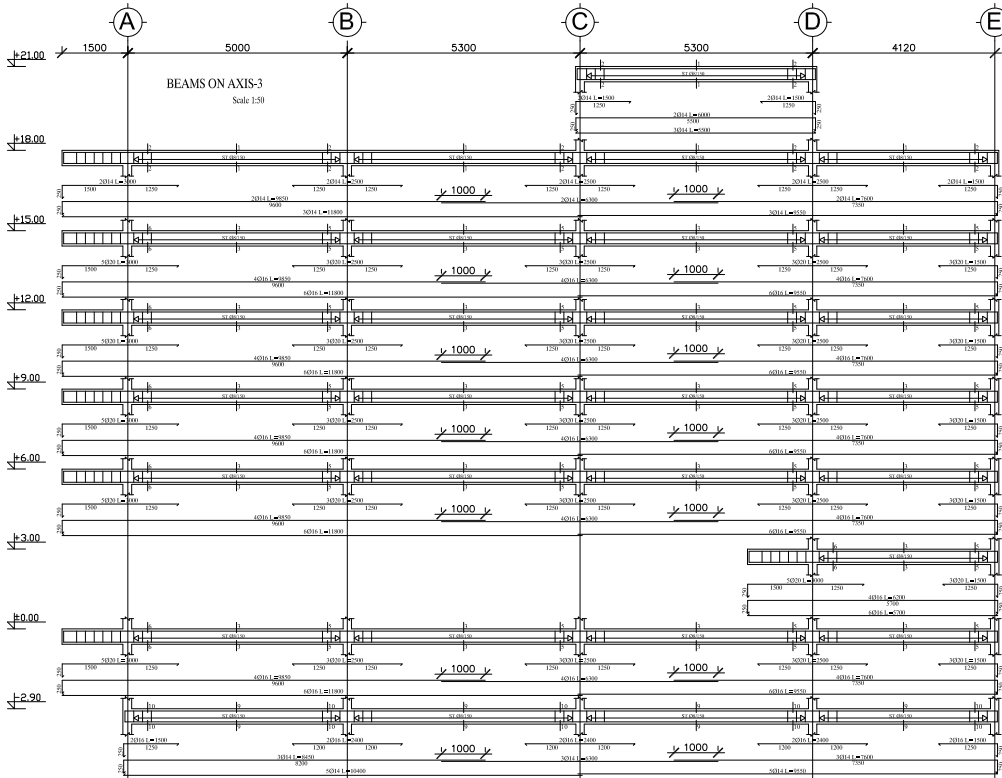
PROJECT STATUS:
 FINAL DESIGN
 CONSTRUCTION UNDER SERVICE
 DISCLOSED
 DATE: **05-MAR-2010**
 REVISION SHEET #: **A**

Checked by
Daniel Getachew
ADVISOR
Adil Zekaria, Dr.-Ing
DATE: **JAN-2011**
REVISION SHEET #: **A**

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 OF BUILDINGS: A CASE STUDY OF SELECTED
 BUILDINGS CONSTRUCTED IN ADDIS ABABA

LOCATION:
 ADDIS ABABA, ETHIOPIA



Rev.No.	Date	Changes OR Remark	By

NOTE : ALL DIMENSIONS ARE IN MM. UNLESS OTHERWISE STATED. ...

BUILDING TYPE:
**Case Study # 2: Eight Story-Building
 Ribbed Slab Floor System**

DRAWING NUMBER: **ST-08**

DRAWING TITLE: **BEAM DETAILS (2 OF 4)**

PROJECT STATUS:
 FINAL DESIGN
 CONSTRUCTION UNDER SERVICE

DESIGNED BY:
DISCLOSED

DATE:
05-MAR-2010

REVISION SHEET #:
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Checked by
Daniel Getachew
 ADVISOR
Adil Zekaria, Dr.-Ing

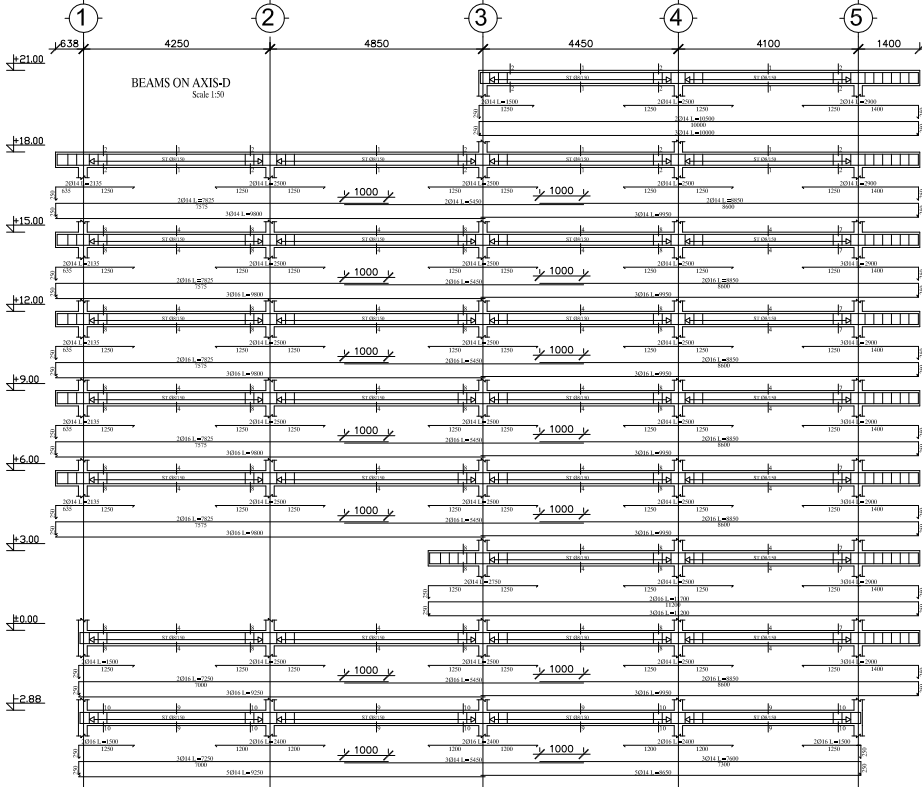
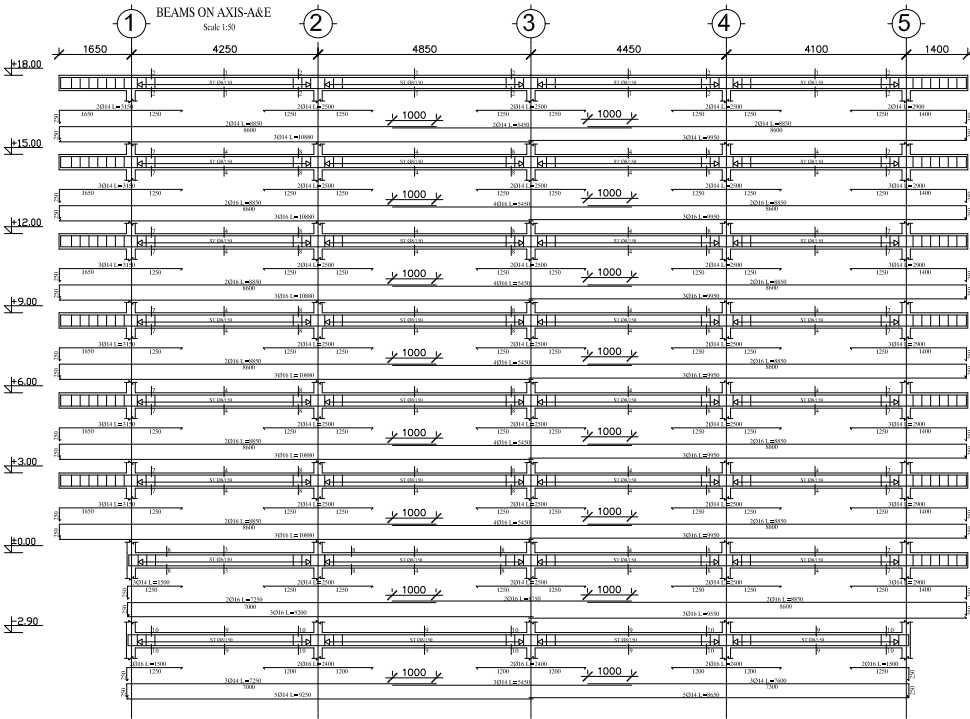
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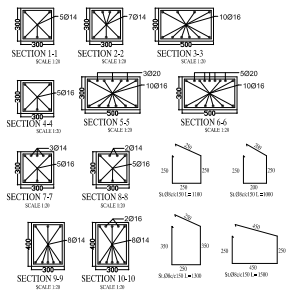
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DEPARTMENT OF CIVIL ENGINEERING

PROJECT:
ASSESSMENT OF SEISMIC VULNERABILITY
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LOCATION:
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BEAMS SECTION



Rev.No:	Date:	Changes OR Remark:	By:

NOTE : ALL DIMENSIONS ARE IN MM. UNLESS OTHERWISE STATED. ...

BUILDING TYPE: **Case Study # 2: Eight Story-Building Ribbed Slab Floor System**

DRAWING NUMBER: **ST-09**
DRAWING TITLE: **BEAM DETAILS (3 OF 4)**

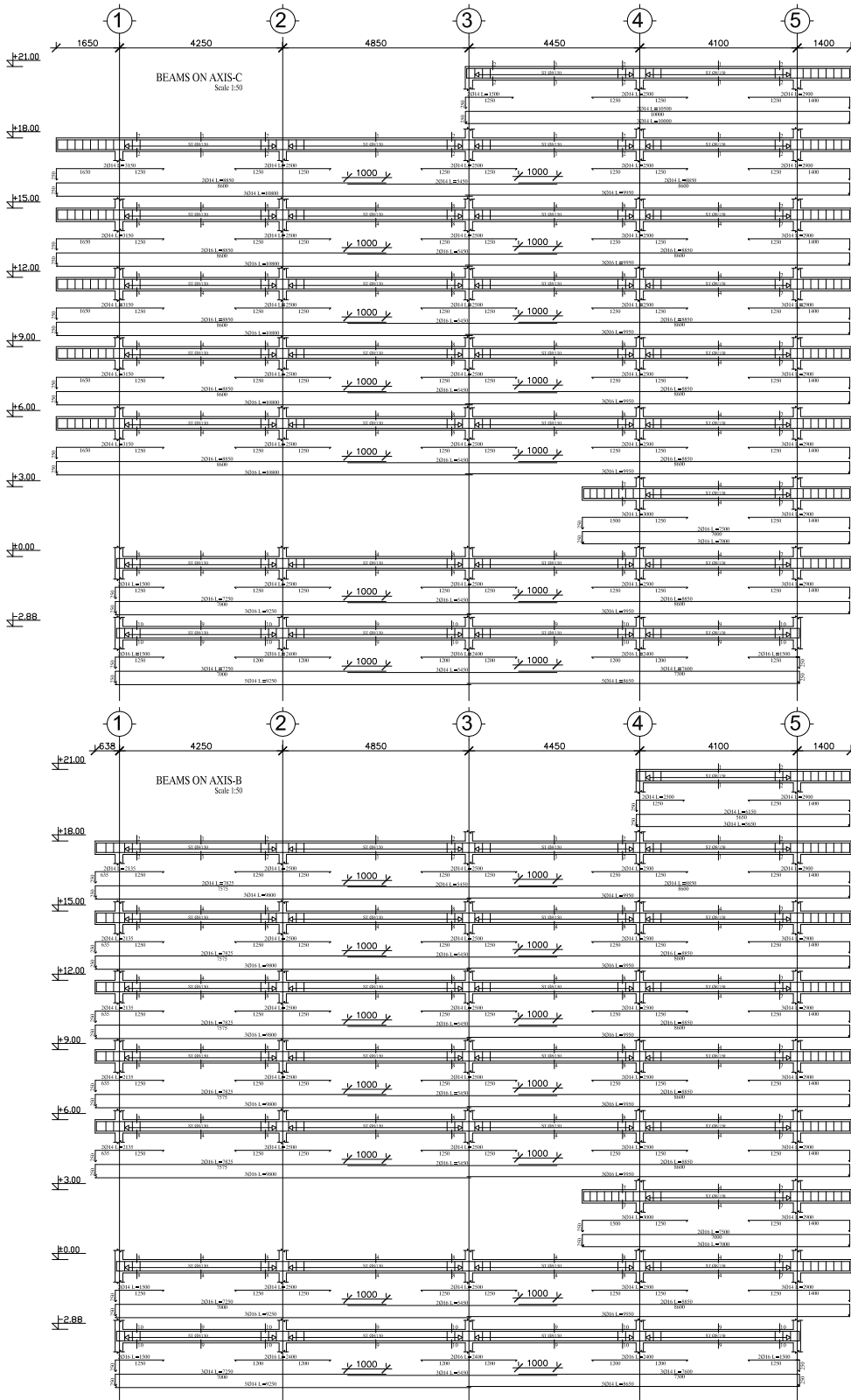
PROJECT STATUS:
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 CONSTRUCTION UNDER SERVICE
 DISCLOSED
 DATE: **05-MAR-2010**
 REVISION SHEET #: **A**

Checked by
Daniel Getachew
ADVISOR
Adil Zekaria, Dr.-Ing
DATE: **JAN-2011**
REVISION SHEET #: **A**

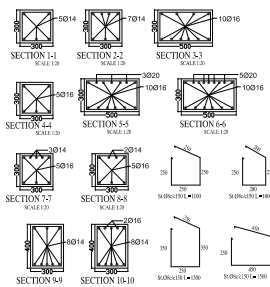
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INSTITUTION OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING

PROJECT:
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BUILDINGS CONSTRUCTED IN ADDIS ABABA

LOCATION:
ADDIS ABABA, ETHIOPIA



BEAMS SECTION



Rev.No:	Date:	Changes OR Remark:	By:

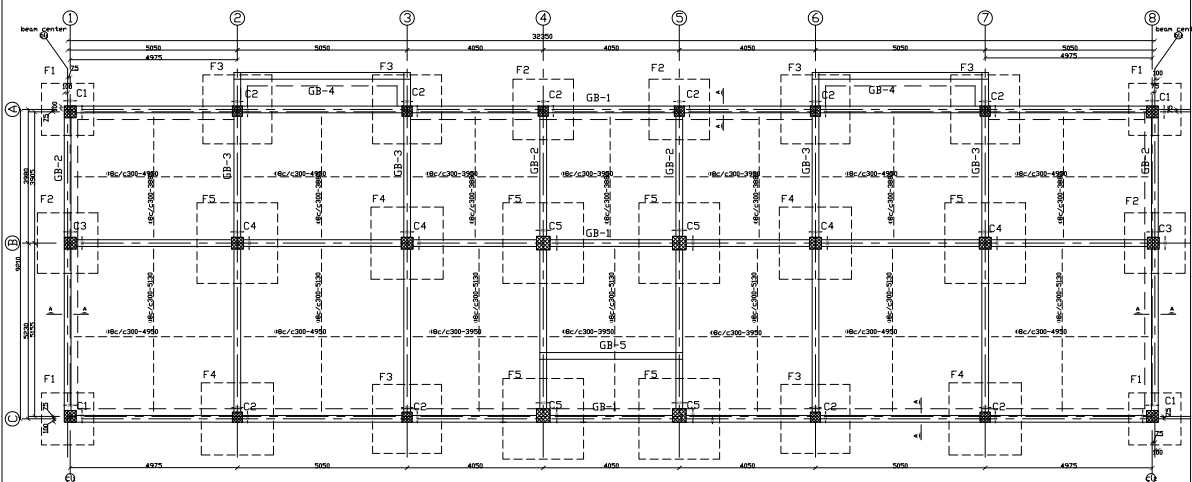
NOTE : ALL DIMENSIONS ARE IN MM. UNLESS OTHERWISE STATED. ...

BUILDING TYPE:
**Case Study # 2: Eight Story-Building
Ribbed Slab Floor System**

DRAWING NUMBER: **ST-10**
DRAWING TITLE: **BEAM DETAILS (4 OF 4)**

PROJECT STATUS:
 FINAL DESIGN
 CONSTRUCTION UNDER SERVICE
 DISCLOSED
 DATE: **05-MAR-2010** REVISION SHEET #: **A**

Checked by
Daniel Getachew
ADVISOR
Adil Zekaria, Dr.-Ing
DATE: **JAN-2011** REVISION SHEET #: **A**



FOOTING, COLUMN AND GROUND BEAM LAYOUT & GROUND FLOOR SLAB REINFORCEMENT.
(Ground floor slab thickness, $h=100\text{mm}$)
Scale 1:50

Rev/No.	Date	Changes OR Remark	By

NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...

BUILDING TYPE: **Case Study # 3: Six Story-Building
Pre Cast Rib Floor System**

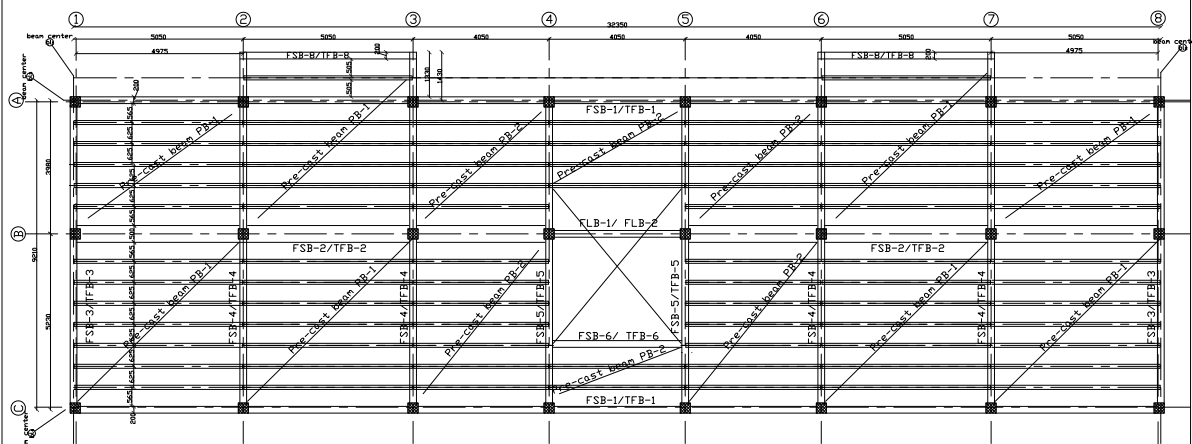
DRAWING NUMBER: **ST-01**

DRAWING TITLE: **FOUNDATION AND GRADE BEAM PLANS**

PROJECT STATUS:
 REAL-CASE
 CONSTRUCTION UNDER-GOING
 DISCLOSED

Checked by
Daniel Getachew
ADVISOR
DESIGNED BY:
Adil Zekaria, Dr.-Ing
DATE: **JAN-2011** REVISION SHEET #:
A

DATE: **05-MAR-2010** REVISION SHEET #:
A



TYPICAL FLOOR PRE-CAST BEAM ARRANGMENT & BEAM LAYOUT
Scale 1:50

Rev/No.	Date	Changes OR Remark	By

NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...

BUILDING TYPE: **Case Study # 3: Six Story-Building
Pre Cast Rib Floor System**

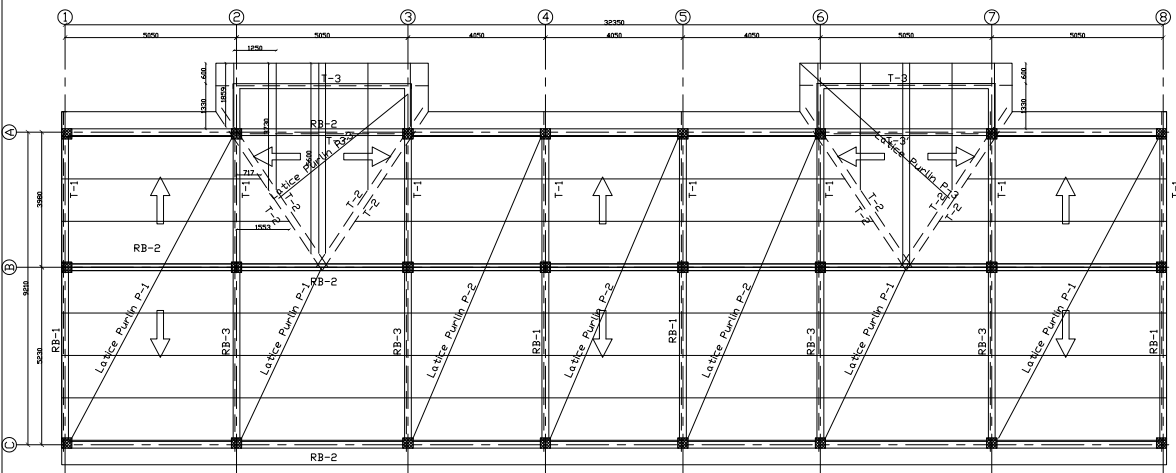
DRAWING NUMBER: **ST-02**

DRAWING TITLE: **TYPICAL FLOOR LAYOUT**

PROJECT STATUS:
 REAL-CASE
 CONSTRUCTION UNDER-GOING
 DISCLOSED

Checked by
Daniel Getachew
ADVISOR
DESIGNED BY:
Adil Zekaria, Dr.-Ing
DATE: **JAN-2011** REVISION SHEET #:
A

DATE: **05-MAR-2010** REVISION SHEET #:
A



TOP-TIE BEAM & LATTICE PURLINE LAYOUT
Scale 1:50

Rev./No.	Date	Changes OR Remark	By

NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...

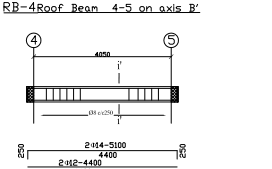
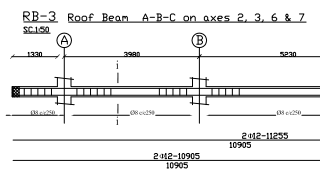
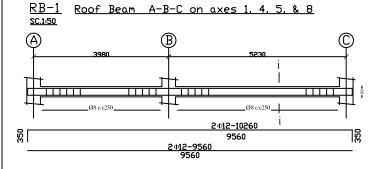
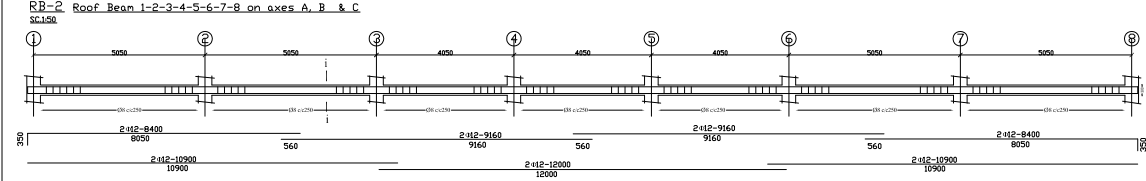
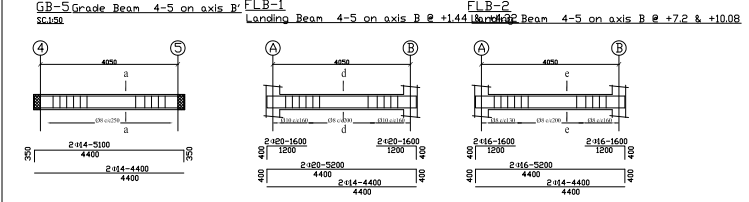
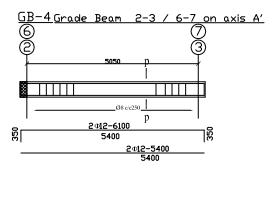
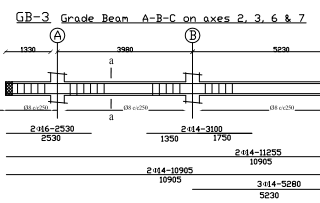
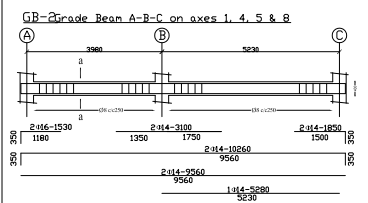
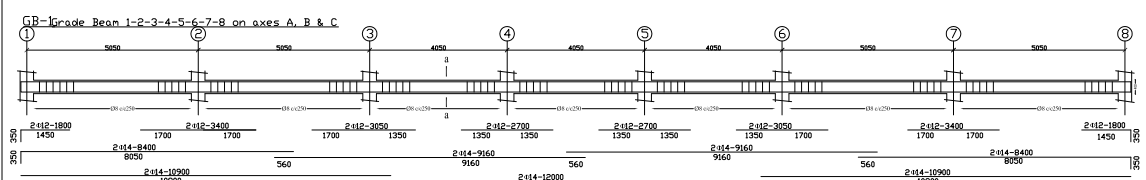
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Pre Cast Rib Floor System**

DRAWING NUMBER: **ST-03**

DRAWING TITLE: **TOP TIE BEAM LAYOUT**

PROJECT STATUS:
 FINAL DESIGN
 CONSTRUCTION UNDER SERVICE
 DISCLOSED
 DATE: 05-MAR-2010
 REVISION SHEET #:

Checked by:
Daniel Getachew
ADVISOR
 DESIGNED BY:
Adil Zekaria, Dr.-Ing
DATE: JAN-2011
 REVISION SHEET #:



Rev./No.	Date	Changes OR Remark	By

NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...

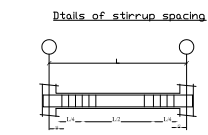
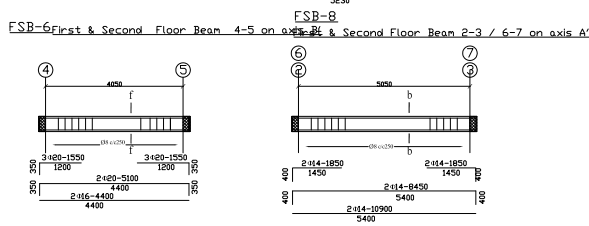
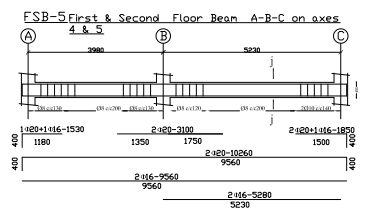
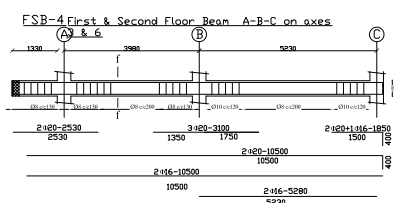
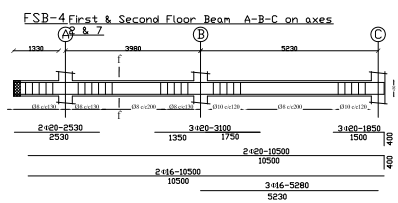
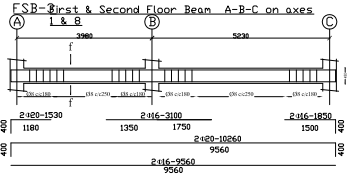
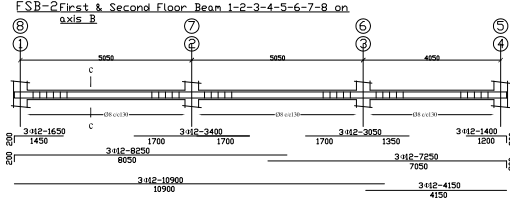
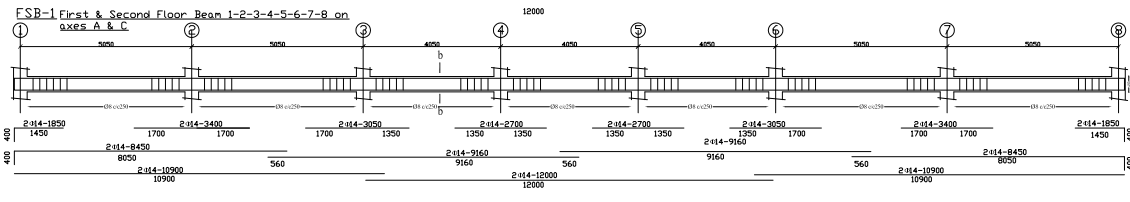
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Pre Cast Rib Floor System**

DRAWING NUMBER: **ST-04**

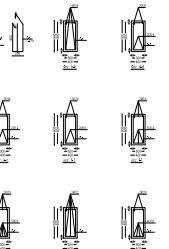
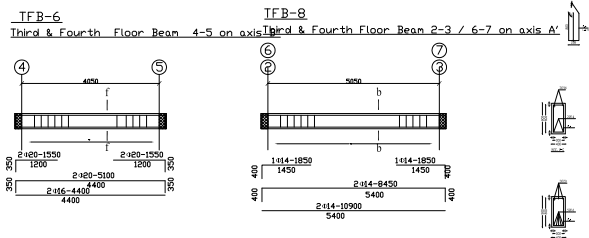
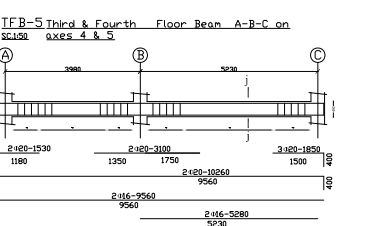
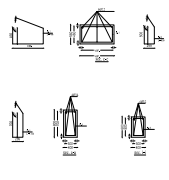
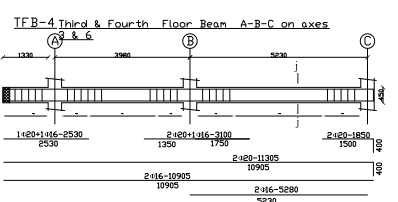
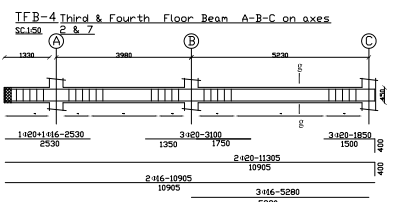
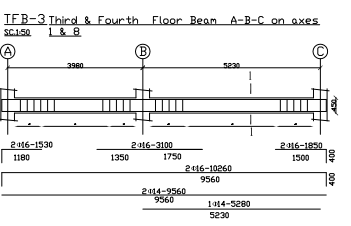
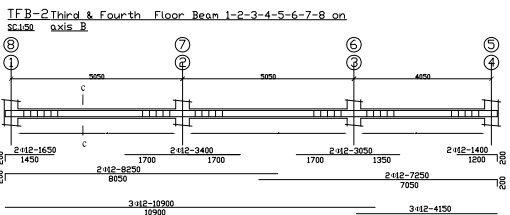
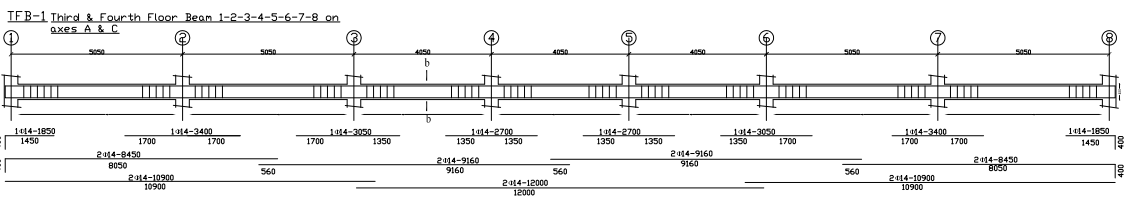
DRAWING TITLE: **BEAM DETAILS 1**

PROJECT STATUS:
 FINAL DESIGN
 CONSTRUCTION UNDER SERVICE
 DISCLOSED
 DATE: 05-MAR-2010
 REVISION SHEET #:

Checked by:
Daniel Getachew
ADVISOR
 DESIGNED BY:
Adil Zekaria, Dr.-Ing
DATE: JAN-2011
 REVISION SHEET #:



Rev./No.	Date	Changes OR Remark	By
NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...			
BUILDING TYPE: Case Study # 3: Six Story-Building Pre Cast Rib Floor System			
DRAWING NUMBER: ST-05		DRAWING TITLE: BEAM DETAILS 2	
PROJECT STATUS: DISCLOSED		Checked by: Daniel Getachew	
DESIGNED BY: DISCLOSED		ADVISOR: Adil Zekaria, Dr.-Ing	
DATE: 05-MAR-2019	REVISION SHEET #:	DATE: JAN-2011	REVISION SHEET #:
	A		A



BEAM SECTION

Rev./No.	Date	Changes OR Remark	By
NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...			
BUILDING TYPE: Case Study # 3: Six Story-Building Pre Cast Rib Floor System			
DRAWING NUMBER: ST-06		DRAWING TITLE: BEAM DETAILS 3	
PROJECT STATUS: DISCLOSED		Checked by: Daniel Getachew	
DESIGNED BY: DISCLOSED		ADVISOR: Adil Zekaria, Dr.-Ing	
DATE: 05-MAR-2019	REVISION SHEET #:	DATE: JAN-2011	REVISION SHEET #:
	A		A

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

PROJECT:
ASSESSMENT OF SEISMIC VULNERABILITY
OF BUILDINGS: A CASE STUDY OF SELECTED
BUILDINGS CONSTRUCTED IN ADDIS ABABA

LOCATION:
ADDIS ABABA, ETHIOPIA

BUILDING TYPE:
**Case Study # 3: Six Story-Building
Pre-Cast Rib Floor System**

DRAWING NUMBER:
ST-07

DRAWING TITLE:
**COLUMN DETAILS, STAIR DETAILS AND
PRE-CAST BEAM DETAILS**

PROJECT STATUS:

PHASE DESIGN CONSTRUCTION UNDER SERVICE

DESIGNED BY:

REVISION SHEET #:

DATE:

05-MAR-2010 A

Checked by:

Daniel Getachew

ADVISOR

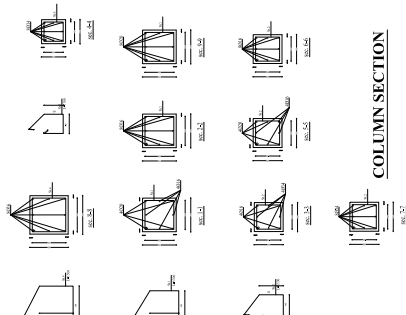
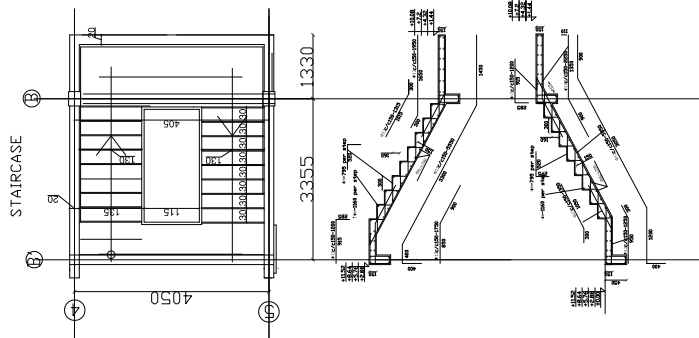
Adil Zekaria, Dr.-Ing

DATE: JAN-2011

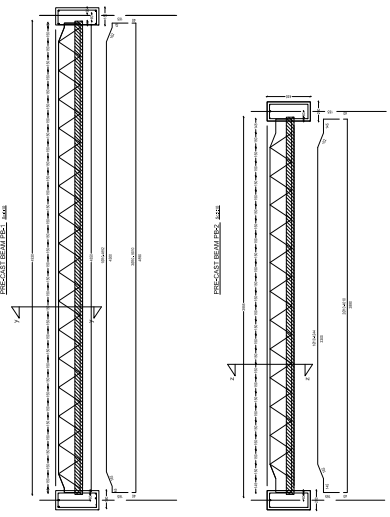
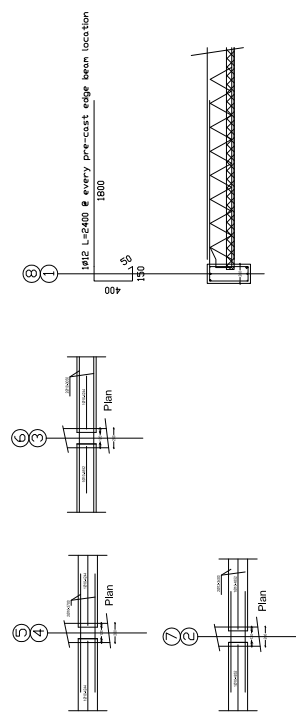
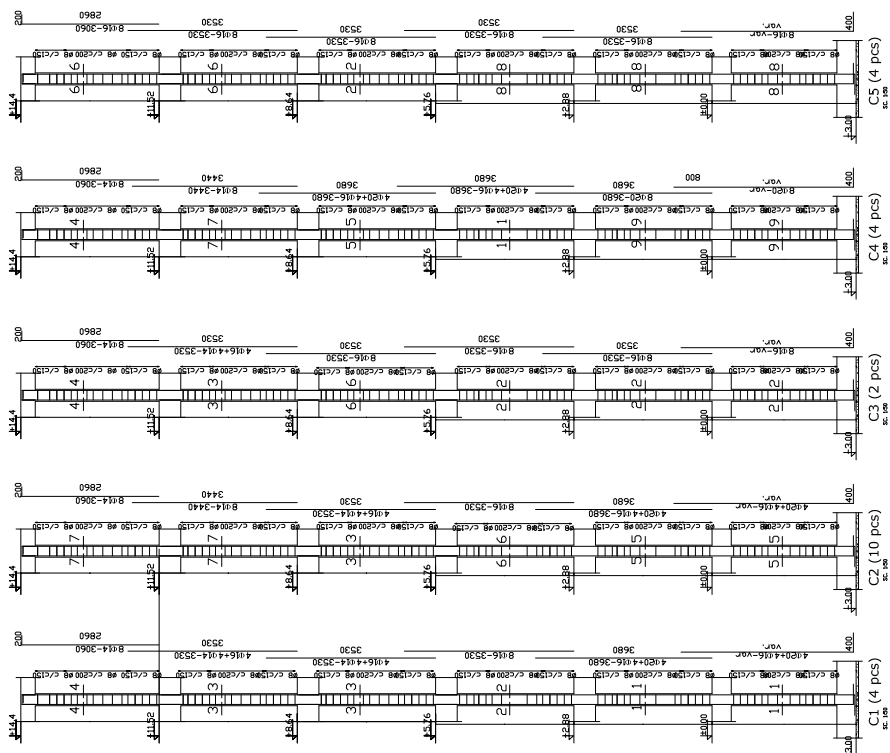
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Rev No.	Date	Changes OR Remark	By

NOTE: ALL DIMENSIONS ARE IN MM. UNLESS OTHERWISE STATED....



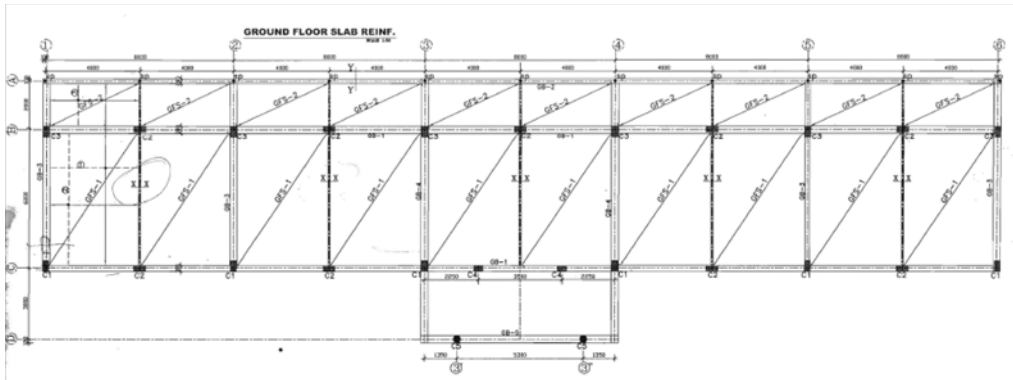
COLUMN SECTION



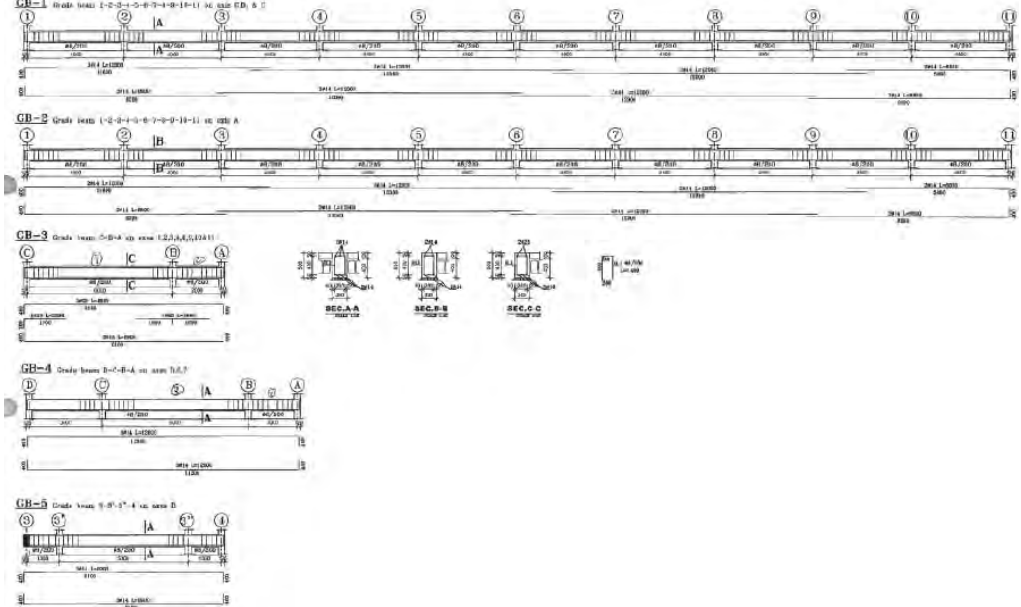
ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES
INSTITUTION OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING

PROJECT:
ASSESSMENT OF SEISMIC VULNERABILITY
OF BUILDINGS: A CASE STUDY OF SELECTED
BUILDINGS CONSTRUCTED IN ADDIS ABABA

LOCATION:
ADDIS ABABA, ETHIOPIA



GROUND FLOOR BEAMS
(SCALE 1/100)



Rev. No.	Date	Change or Remark	By

NOTE: ALL DIMENSIONS ARE IN MM UNLESS OTHERWISE STATED...

BUILDING TYPE: Case Study # 4: Five Story-Building
Pre Cast Rib Floor System

DRAWING NUMBER: ST-01

DRAWING TITLE: GRADE BEAM PLANS AND DETAILS

PROJECT STATUS:
 FINAL DESIGN CONSTRUCTION UNDER SERVICE

Checked by: **Daniel Getachew**
ADVISOR

DESIGNED BY: **Adil Zekaria, Dr.-Ing**

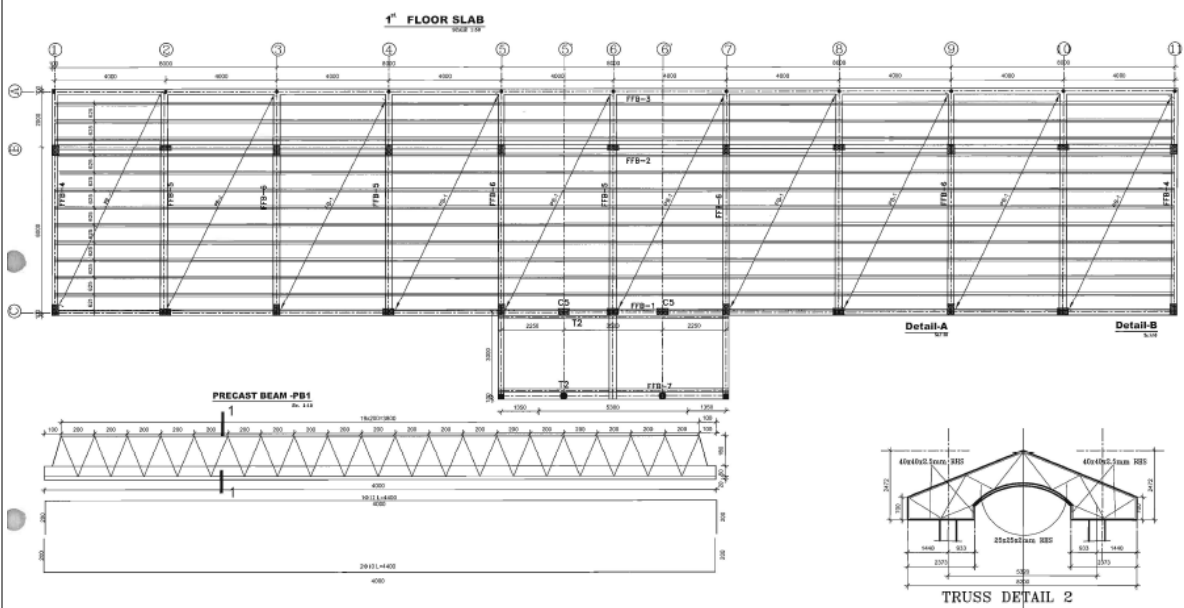
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DATE: 05-MAR-2010 REVISION SHEET #: A DATE: JAN-2011 REVISION SHEET #: A

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

PROJECT:
ASSESSMENT OF SEISMIC VULNERABILITY
OF BUILDINGS: A CASE STUDY OF SELECTED
BUILDINGS CONSTRUCTED IN ADDIS ABABA

LOCATION:
ADDIS ABABA, ETHIOPIA



Rev. No.	Date	Change or Remark	By

NOTE: ALL DIMENSIONS ARE IN MM UNLESS OTHERWISE STATED...

BUILDING TYPE: Case Study # 4: Five Story-Building
Pre Cast Rib Floor System

DRAWING NUMBER: ST-02

DRAWING TITLE: FIRST FLOOR LAYOUT

PROJECT STATUS:
 FINAL DESIGN CONSTRUCTION UNDER SERVICE

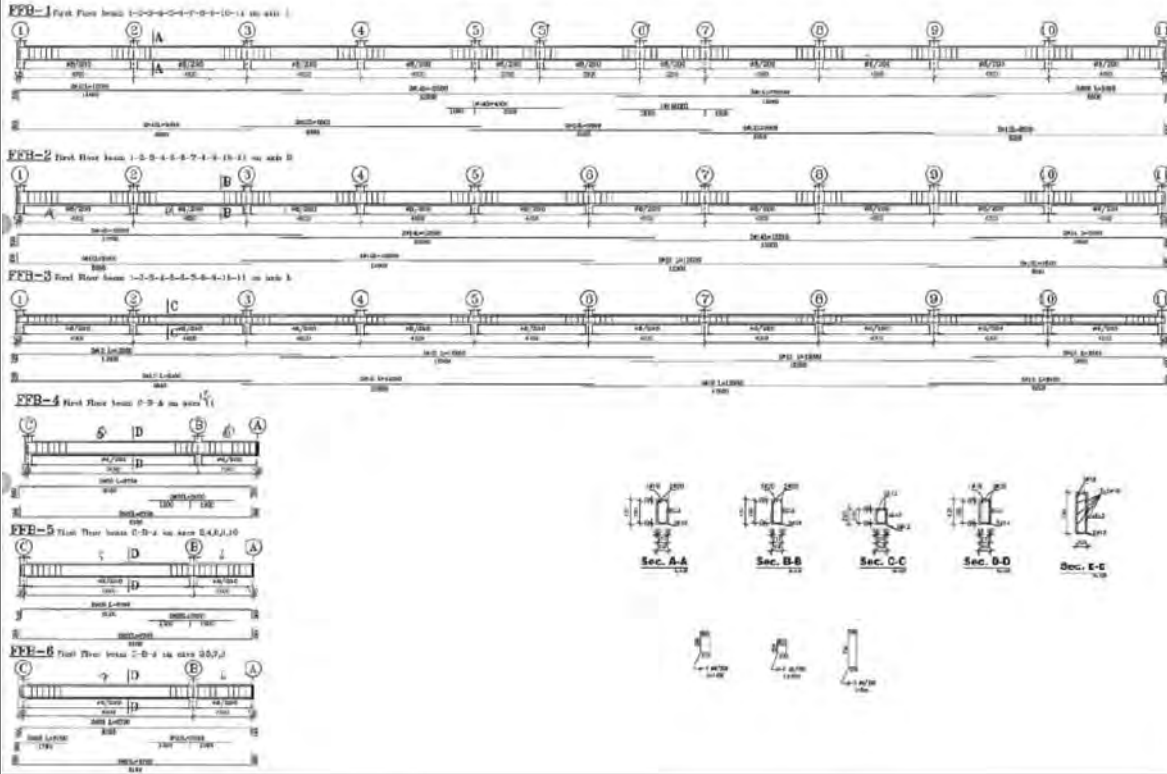
Checked by: **Daniel Getachew**
ADVISOR

DESIGNED BY: **Adil Zekaria, Dr.-Ing**

DISCLOSED

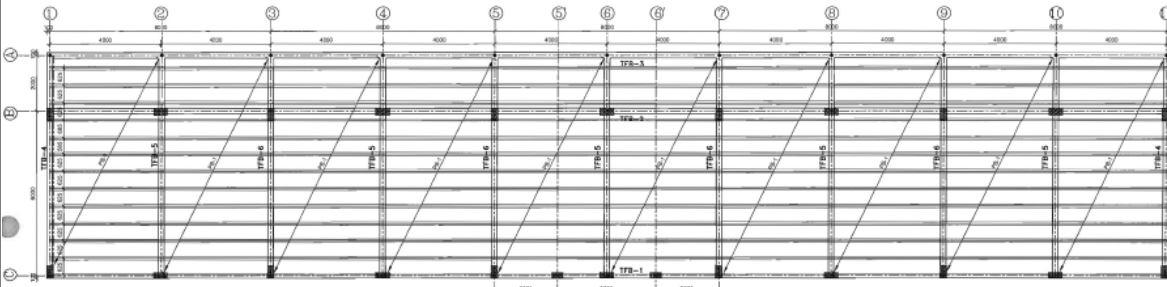
DATE: 05-MAR-2010 REVISION SHEET #: A DATE: JAN-2011 REVISION SHEET #: A

FIRST FLOOR BEAMS

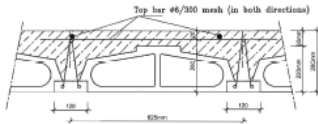


Rev./No.	Date	Changes OR Remark	By
NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...			
BUILDING TYPE: Case Study # 4: Five Story-Building Pre Cast Rib Floor System			
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DESIGNED BY: DISCLOSED		Checked by: Daniel Getachew	
DATE: 05-MAR-2019		ADVISOR: Adil Zekaria, Dr.-Ing	
REVISION SHEET #:		DATE: JAN-2011	
A		A	

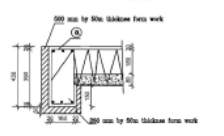
TYPICAL 2nd, 3rd & 4th FLOOR SLAB



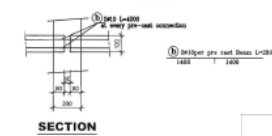
MCB SLAB DETAIL



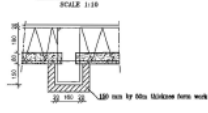
REINFORCEMENT AND FORM WORK DETAIL AT PRE CAST BEAM LOCATION AT EDGE SUPPORT, AXIS C



REINFORCEMENT AND FORM WORK DETAIL AT PRE CAST BEAM LOCATION AT INTERIOR SUPPORT, AXIS B



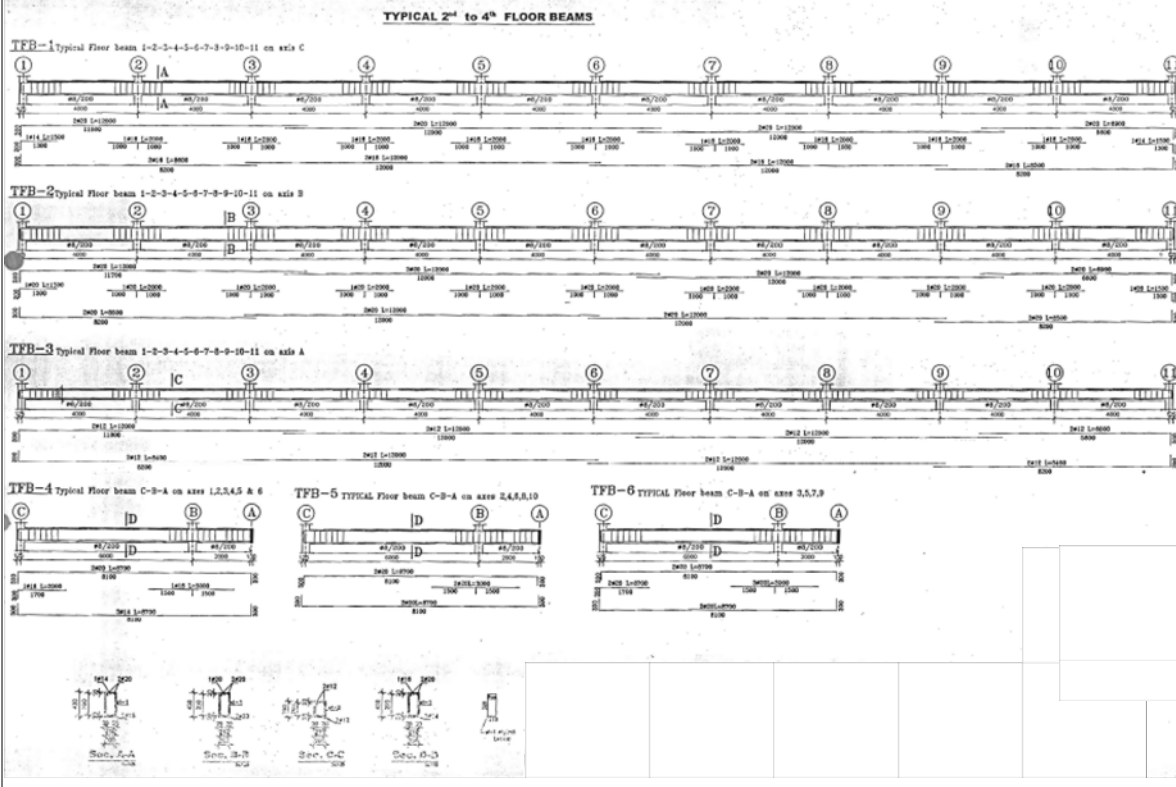
SECTION



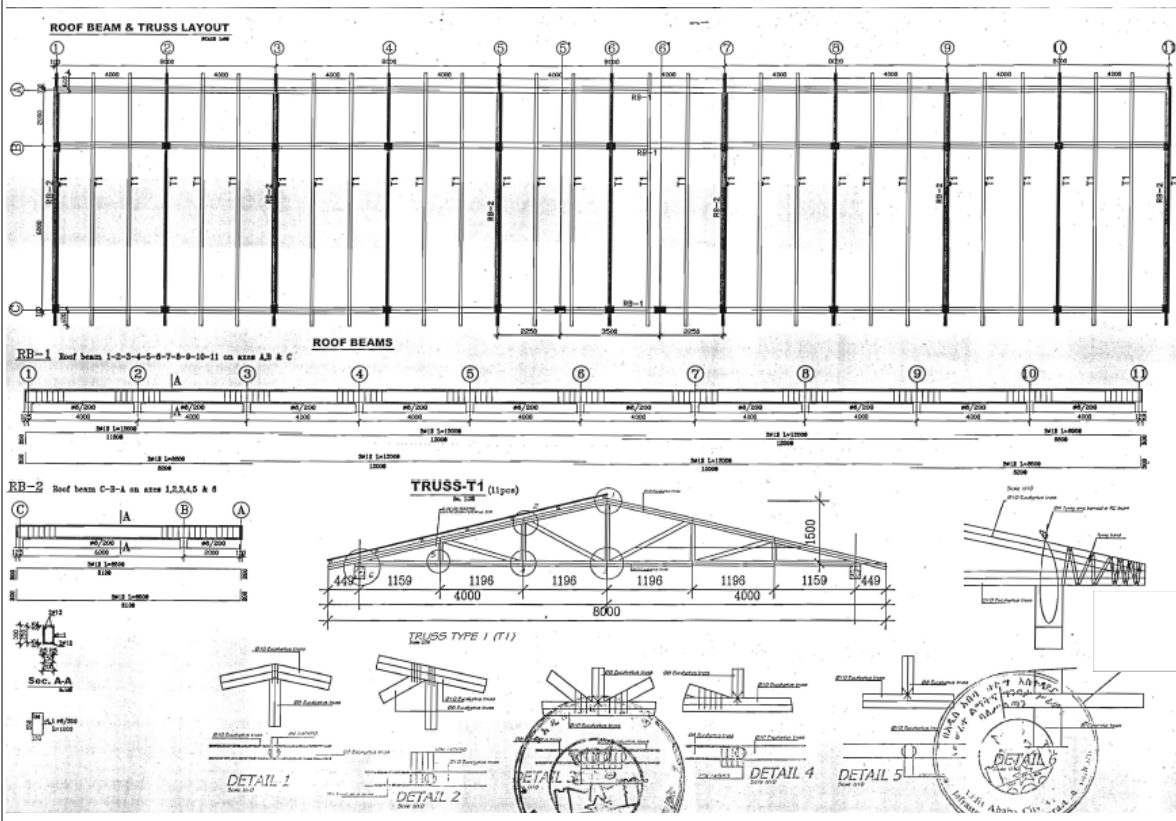
Rev./No.	Date	Changes OR Remark	By
NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...			
BUILDING TYPE: Case Study # 4: Five Story-Building Pre Cast Rib Floor System			
DRAWING NUMBER: ST-04		DRAWING TITLE: 2ND TO 4TH FLOOR PLAN	
PROJECT STATUS: <input type="radio"/> FINAL DESIGN <input type="radio"/> CONSTRUCTION <input checked="" type="radio"/> UNDER SERVICE			
DESIGNED BY: DISCLOSED		Checked by: Daniel Getachew	
DATE: 05-MAR-2019		ADVISOR: Adil Zekaria, Dr.-Ing	
REVISION SHEET #:		DATE: JAN-2011	
A		A	

PROJECT:
ASSESSMENT OF SEISMIC VULNERABILITY
OF BUILDINGS: A CASE STUDY OF SELECTED
BUILDINGS CONSTRUCTED IN ADDIS ABABA

LOCATION:
ADDIS ABABA, ETHIOPIA



Rev./No.	Date	Changes OR Remark	By
NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...			
BUILDING TYPE: Case Study # 4: Five Story-Building Pre Cast Rib Floor System			
DRAWING NUMBER: ST-05		Checked by: Daniel Getachew	
DRAWING TITLE: 2ND TO 4TH FLOOR BEAM DETAIL			
PROJECT STATUS: FINAL DESIGN CONSTRUCTION UNDER SERVICE			
DESIGNED BY: DISCLOSED		ADVISOR: Adil Zekaria, Dr.-Ing	
DATE: 05-MAR-2010	REVISION SHEET #: A	DATE: JAN-2011	REVISION SHEET #: A



Rev./No.	Date	Changes OR Remark	By
NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED...			
BUILDING TYPE: Case Study # 4: Five Story-Building Pre Cast Rib Floor System			
DRAWING NUMBER: ST-06		Checked by: Daniel Getachew	
DRAWING TITLE: ROOF FLOOR PLAN AND BEAM DETAILS			
PROJECT STATUS: FINAL DESIGN CONSTRUCTION UNDER SERVICE			
DESIGNED BY: DISCLOSED		ADVISOR: Adil Zekaria, Dr.-Ing	
DATE: 05-MAR-2010	REVISION SHEET #: A	DATE: JAN-2011	REVISION SHEET #: A

ADDIS ABABA UNIVERSITY
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 DEPARTMENT OF CIVIL ENGINEERING

PROJECT:
 ASSESSMENT OF SEISMIC VULNERABILITY
 OF BUILDINGS: A CASE STUDY OF SELECTED
 BUILDINGS CONSTRUCTED IN ADDIS ABABA

LOCATION:
 ADDIS ABABA, ETHIOPIA

Rev No.	Date	Changes OR Remark	By

NOTE: ALL DIMENSIONS ARE IN MM, UNLESS OTHERWISE STATED....

BUILDING TYPE:
**Case Study # 4: Five Story-Building
 Pre Cast Rib Floor System**

DRAWING NUMBER:
ST-07

DRAWING TITLE:
COLUMN DETAILS

PROJECT STATUS:

FINAL DESIGN CONSTRUCTION UNDER SERVICE

DESIGNED BY:
DISCLOSED

DATE:
05-MAR-2010

Checked by
Daniel Getachew
 ADVISOR

DESIGNED BY:
Adil Zekaria, Dr.-Ing

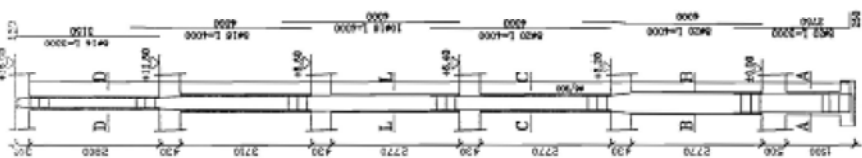
DATE:
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REVISION SHEET #:
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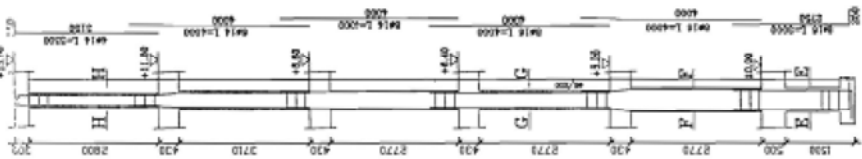
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COLUMNS

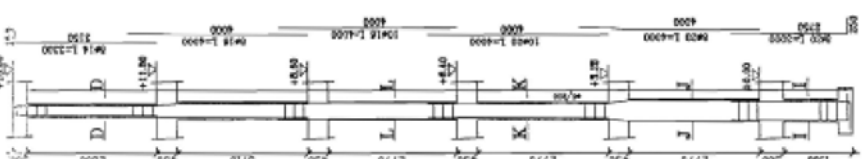
C-1 (Type)



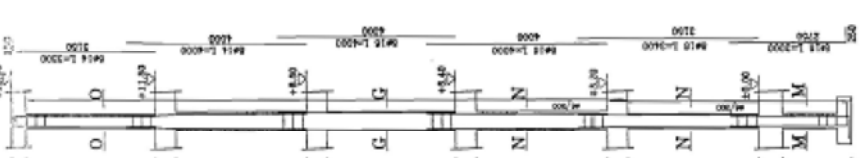
C-2 (Type)



C-3 (Type)



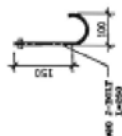
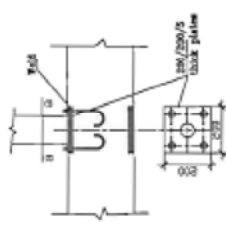
C-4 (Type)



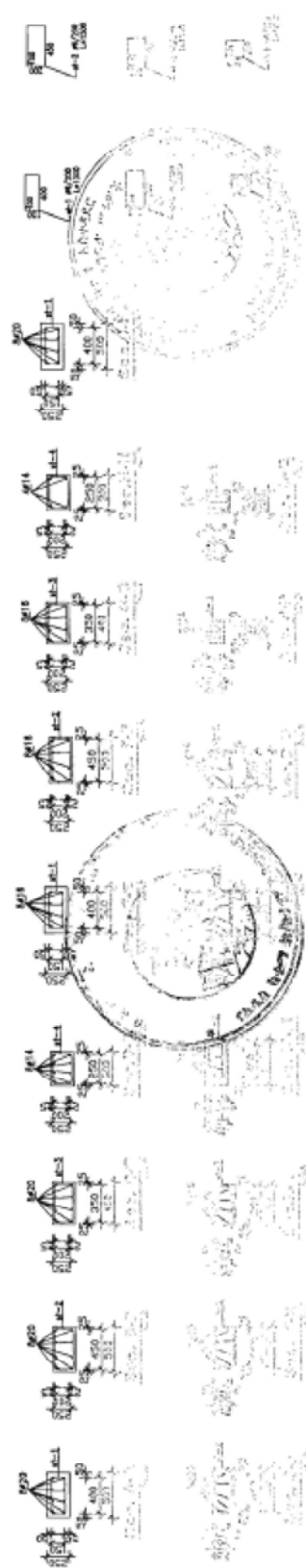
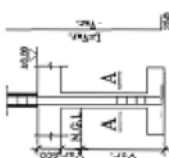
**Steel Post (SP)
 Detail Prior Floor**



Detail A



TYPICAL FOUNDATION DETAIL



DECLARATION

This thesis is my original work and has not been presented for a degree in any other university, and that all sources of material used for the thesis have been duly acknowledged.

NAME: DANIEL GETACHEW GEBREYOHANNES

SIGNATURE:  _____

PLACE: ADDIS ABABA INSTITUTE OF TECHNOLOGY, ADDIS ABABA UNIVERSITY

DATE OF SUBMISSION: MAY, 2011