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Seismic Performance Evaluation of Essential Facilities Building in Dhaka City by Inelastic Time History Analysis.

A Thesis Submitted in Partial
Fulfillment of the Requirements for the Bachelor of Science degree in

CIVIL & ENVIRONMENTAL ENGINEERING (CEE)

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Islamic University Of Technology (IUT)

The Thesis entitled "Seismic Performance Evaluation of Essential Facilities Building in Dhaka City by Inelastic Time History Analysis", by Md. Sabbir Ahmed & Farzad Khan Lodi has been approved in partial fulfillment of the requirements for the Bachelor of Science in Civil & Environmental Engineering (CEE).

Mohammad Shafiqual Alam ________________

ABSTRACT

Structures suffer significant inelastic deformation under a strong earthquake and dynamic characteristics of the structure change with time, so investigating the performance of a structure requires inelastic analytical procedures accounting for these features. Inelastic analytical procedures help to understand the actual behavior of structures by identifying failure modes and the potential for progressive collapse. Among the various inelastic procedures, the inelastic time history analysis is the most accurate method to predict the force and deformation demands at various components of the structure and hence, the application of inelastic time history analysis to evaluate the seismic performance of Secretariat Clinic Building located at the Secretariat of Government of Bangladesh, Dhaka is the focus of this thesis. Due to the unavailability of any strong ground motion record applicable for Dhaka city in the literature, artificial time history record generated based on response spectrum of Bangladesh National Building Code (BNBC,1993) has been used for analysis purpose in nonlinear finite element package, SeismoStruct-V5.2.2 (2011). The analysis reveals that the case study building possesses much greater stability in the longitudinal direction compared to the shorter direction in terms of energy dissipation capacity. It was also observed that storey sway mechanism develops in the bottom and top floors in both the directions under the input artificial time history records which may be attributed to the open ground floor for parking purposes and reduced steel area in columns at top floor from gravity load considerations.

Keywords: Inelastic time history analysis, inter-storey drift, storey sway mechanism, strong motion.

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TABLE OF CONTENTS

ABSTRACT.. i
ACKNOWLEDGEMENTS…………………………………………………………………….. ii
LIST OF FIGURES……………………………………………………………………………… v
LIST OF TABLES……..…………………………………………………………………………vii

CHAPTER

1 Introduction
1.1 Foreword……………………………………………………………………. 01
1.2 Methods of analysis………………………………………………………… 01
1.2.1 Elastic methods of analysis………………………………………………… 01
1.2.2 Inelastic methods of analysis………………………………………………. 03
1.2.3 Summary…………………………………………………………………… 03
1.3 Objectives…………………………………………………………………… 04
1.4 Outline of the thesis....…………………………..…………………………….. 04

2 Review of previous research
2.1 General……………………………………………………………………... 05
2.2 Inelastic time history analysis ………………………………..………………… 05
2.2.1 Modeling Approaches for time history analysis…….………………………….. 05
2.2.2 Use of time history results…………………………………………………. 06
2.3 Past studies on time history analysis……………………………………………. 07

3 Case studies and modeling approach
3.1 Description of case study building……………………………………………... 10
3.2 Load distribution in beams at different storeys…….…………………………….. 17
3.3 Non-linear finite element package…………………………..…………………. 23
3.3.1 Introduction to the program……………………..………………...……………… 23
3.3.2 Modeling parameters adopted……………………………………….……….. 27
3.3.3 Member level performance criteria and damage assessment………………………29
3.3.4 Global structural level damage assessment……………………………………….. 30
3.3.5 Global yielding criteria……………………………………………………………30
3.3.6 Global collapse criteria……………………………………………………………31
3.3.7 Inter-storey drift monitoring………………………………………………………31

4 Performance evaluation of case study building (Time History Analysis)
4.1 Introduction……………………………………………….…………………… 33
4.2 Time history record…………………………………………….……………..33
4.3 Time history record generation by SeismoArtificial …………….………………. 34
4.3.1 Brief overview of SeismoArtificial………………………………….……… 34

4.3.2 Time history record generation………………………………………………. 35
4.4 Inelastic time history results for case study building ……………………………. 40
4.4.1 Response in X-direction ……………………………………………………….. 40
4.4.2 Response in Y-direction ……………………………………………………….. 42

5 Conclusion
5.1 Summary……………………………………………………….……………..45
5.2 Future studies…………………………………………………..……………..45

References……….………………………………………………………….………………... 56

LIST OF FIGURES

Section	Name of figures	
Fig-3.1	Layout plan………………………………………………………….…………………..	10
Fig-3.2	Selected area for analysis…………………………………………….…………………..	11
Fig-3.3	3D model of Secretariat Clinic Building in SeismoStruct…………….……...………...	11
Fig-3.4	Layout of beams & columns..………….……………………………..…………….	12
Fig-3.5	Cross-section of beam-1………………………………………………………….…………	12
Fig-3.6	Cross-section of beam-2………………………………………………………….…………	13
Fig-3.7	Cross-section of beam-3………………………………………………………….…………	13
Fig-3.8	Cross-section of beam-4………………………………………………………….…………	14
Fig-3.9	Cross-section of column-1……………………………………….……..………….	14
Fig-3.10	Cross-section of column-2……………………………………….…...………….	15
Fig-3.11	Cross-section of column-5……..…………………………………...…...………….	15
Fig-3.12	Cross-section of column-6……………………………………….…...………….	16
Fig-3.13	Cross-section of column-8……………………………………….…...………….	16
Fig-3.14	Loads in beams at ground floor……………………………………….………..…..……	18
Fig-3.15	Loads in beams at 1st floor……….………………….……………………..……..…	19
Fig-3.16	Loads in beams at 2nd floor …………………...……………………….……..………	20
Fig-3.17	Loads in beams at 3rd floor …………………..………………………….…………….	21
Fig-3.18	Loads in beams at 4th floor …………………………………...……….…...……..……	22
Fig-3.19	Local chord reference systems (SeismoStruct-v5.2.2, 2011)…………………………….	23
Fig-3.20	Elastic component of element stiffness matrix (SeismoStruct-v5.2.2, 2011)……..…..…	24
Fig-3.21	Discretization of an RC section into fibres (SeismoStruct-v-5.2.2, 2011)……………….	25
Fig-3.22	Location of integration Gauss points within an element (SeismoStruct-v-5.2.2, 2011)….	26
Fig-3.23	Menegotto - Pinto (1973) steel model used in SeismoStruct-v-5.2.2, 2011……….…….	26
Fig-3.24	Mander et al (1988) concrete model in SeismoStruct-v-5.2.2, 2011……………………..	27
Fig-3.25	Bilinear stress-strain steel models with strain hardening (SeismoStruct-v-5.2.2, 2011)....	28
Fig-3.26	Base-shear vs. global drift monitoring (after Papanikolaouet al. 2011)……………..……	30
Fig-3.27	Idealized elasto-plastic force displacement relationship according (EC8, CEN 2003)….	31
Fig-4.1	Normalized response spectrum curve (BNBC, 1993)……………………….……………	33
Fig-4.2	Acceleration response spectrum (Sa/g) vs. time (T)…………………………………..…..	34
Fig-4.3	A view of putting the input response spectrum in SeismoArtificial software……………	36
Fig-4.4	A view of putting duration for the output time history data in SeismoArtificial software.	36
Fig-4.5	A view of putting the No. of accelerograms.…………………………………………..…	37
Fig-4.6	Time history record -1 or artificial accelerograms (1)…………………………………..	37
Fig-4.7	Time history record -2 or artificial accelerograms (2)………..……………….….….….	38
Fig-4.8	Time history record -3 or artificial accelerograms (3)……..…………………..…….….	38
Fig-4.9	Time history record -4 or artificial accelerograms (4)……..…………………..…….….	39
Fig-4.10	Time history record -5 or artificial accelerograms (5)……..…………………..…….….	39
Fig-4.11	Inelastic response of building in X-direction…………………………………..….….	40
Fig-4.12	Roof displacement with input time history record in X-direction……………………..…	41

Fig-4.13	Storey displacement of frame along the grid line X_3 in X direction under input motion...	41
Fig-4.14	Inter-storey drift of frame along the grid line X_3 in X direction under input motion…..…	42
Fig-4.15	Inelastic Response of building in Y-direction……………….………………..…………….	43
Fig-4.16	Roof Displacement with input time history record in Y-direction………….……………	43
Fig-4.17	Storey displacement of frame along the grid line Y_9 in Y direction under input motion...	44
Fig-4.18	Inter-storey drift of frame along the grid line Y_9 in Y direction under input motion…….	44

LIST OF TABLES

Section	Name of tables	
Table-3.1	Live load & dead load of the building…………………………………………	17
Table-3.2	Performance criteria for structural members……………….…………….…..….	29
Table-3.3	Selected material strains……….…………………..………………..................	29
Table-3.4	Performance levels and damage descriptions…………………………………	32
Table-4.1	Time and response spectrum……………………………………….……….	34

1 INTRODUCTION

1.1 FOREWORD

Old generation of design codes based on equivalent elastic force approaches proved to be ineffective in preventing earthquake destructive consequences. After recent major earthquakes (e.g. Northridge 1994, Kobe 1995, Kocaeli 1999), the necessity for using ever more accurate methods, which explicitly account for geometrical nonlinearities and material inelasticity, for evaluating seismic demand on structures, became evident. Within this framework, two analysis tools are currently offered with different levels of complexity and of required computational effort; nonlinear static analysis (pushover) and nonlinear dynamic analysis (time-history). Even if the latter is commonly considered to be complex and not yet mature enough for widespread professional use, it constitutes the most powerful and accurate tool for seismic assessment. In the latest generation of seismic regulations, dynamic analysis of three dimensional structural models is indeed recommended for the assessment of existing critical structures in zones of high seismic risk, as well in the planning and design of appropriate retrofitting strategies.

1.2 METHODS OF ANALYSIS

For seismic performance evaluation, a structural analysis of the mathematical model of the structure is required to determine force and displacement demands in various components of the structure. Several analysis methods, both elastic and inelastic, are available to predict the seismic performance of the structures.

1.2.1 ELASTIC METHODS OF ANALYSIS

The force demand on each component of the structure is obtained and compared with available capacities by performing an elastic analysis. Elastic analysis methods include code static lateral force procedure, code dynamic procedure and elastic procedure using demand-capacity ratios. These methods are also known as force-based procedures which assume that structures respond elastically to earthquakes.

In code static lateral force procedure, a static analysis is performed by subjecting the structure to lateral forces obtained by scaling down the smoothened soil-dependent elastic response spectrum by a structural system dependent force reduction factor, "R". In this approach, it is assumed that the actual strength of structure is higher than the design strength and the structure is able to dissipate energy through yielding.

In code dynamic procedure, force demands on various components are determined by an elastic dynamic analysis. The dynamic analysis may be either a response spectrum analysis or an elastic time history analysis. Sufficient number of modes must be considered to have a mass participation of at least 90% for response spectrum analysis. Any effects of higher modes are automatically included in time history analysis.

In demand/capacity ratio (DCR) procedure, the force actions are compared to corresponding capacities as demand/capacity ratios. Demands for DCR calculations must include gravity effects. While code static lateral force and code dynamic procedures reduce the full earthquake demand by an R-factor, the DCR approach takes the full earthquake demand without reduction and adds it to the gravity demands. DCRs approaching 1.0 (or higher) may indicate potential deficiencies.

Although force-based procedures are well known by engineering profession and easy to apply, they have certain drawbacks. Structural components are evaluated for serviceability in the elastic range of strength and deformation. Post-elastic behavior of structures could not be identified by an elastic analysis. However, post-elastic behavior should be considered as almost all structures are expected to deform in inelastic range during a strong earthquake. The seismic force reduction factor "R" is utilized to account for inelastic behavior indirectly by reducing elastic forces to inelastic. Force reduction factor, "R", is assigned considering only the type of lateral system in most codes, but it has been shown that this factor is a function of the period and ductility ratio of the structure as well.

Elastic methods can predict elastic capacity of structure and indicate where the first yielding will occur, however they don’t predict failure mechanisms and account for the redistribution of forces that will take place as the yielding progresses. Real deficiencies present in the structure could be missed. Moreover, force-based methods primarily provide life safety but they can’t provide damage limitation and easy repair. The drawbacks of force-based procedures and the dependence of damage on deformation have led the researches to develop displacement-based procedures for seismic performance evaluation. Displacement-based procedures are mainly based on inelastic deformations rather than elastic forces and use nonlinear analysis procedures considering seismic demands and available capacities explicitly.

Elastic Dynamic Analysis shall be used to estimate the displacement demands for structures where ESA does not provide an adequate level of sophistication to estimate the dynamic behavior. A linear elastic multi-modal spectral analysis utilizing the appropriate response spectrum shall be performed. The number of degrees of freedom and the number of modes considered in the analysis shall be sufficient to capture at least 90% mass participation in the longitudinal and transverse directions. A minimum of three elements per column and four elements per span shall be used in the linear elastic model.

EDA based on design spectral accelerations will likely produce stresses in some elements that exceed their elastic limit. The presence of such stresses indicates nonlinear behavior. The engineer should recognize that forces generated by linear elastic analysis could vary considerable from the actual force demands on the structure. Sources of nonlinear response that are not captured by EDA include the effects of the surrounding soil, yielding of structural components, opening and closing of expansion joints, and nonlinear restrainer and abutment behavior. EDA modal results shall be combined using the complete quadratic combination (CQC) method.

Multi-frame analysis shall include a minimum of two boundary frames or one frame and an abutment beyond the frame under consideration.

1.2.2 INELASTIC METHODS OF ANALYSIS

Structures suffer significant inelastic deformation under a strong earthquake and dynamic characteristics of the structure change with time so investigating the performance of a structure requires inelastic analytical procedures accounting for these features. Inelastic analytical procedures help to understand the actual behavior of structures by identifying failure modes and the potential for progressive collapse. Inelastic analysis procedures basically include inelastic time history analysis and inelastic static analysis which is also known as pushover analysis. The inelastic time history analysis is the most accurate method to predict the force and deformation demands at various components of the structure.

However, the use of inelastic time history analysis is limited because dynamic response is very sensitive to modeling and ground motion characteristics. It requires proper modeling of cyclic load deformation characteristics considering deterioration properties of all important components. Also, it requires availability of a set of representative ground motion records that accounts for uncertainties and differences in severity, frequency and duration characteristics. Moreover, computation time, time required for input preparation and interpreting voluminous output make the use of inelastic time history analysis impractical for seismic performance evaluation. Inelastic static analysis, or pushover analysis, has been the preferred method for seismic performance evaluation due to its simplicity. It is a static analysis that directly incorporates nonlinear material characteristics. Inelastic static analysis procedures include Capacity Spectrum Method, Displacement Coefficient Method and the Secant Method.

1.2.3 SUMMARY

The uncertainties involved in accurate determination of material properties, element and structure capacities, the limited prediction of ground motions that the Structure is going to experience and the limitations in accurate modeling of structural behavior make the seismic performance evaluation of structures a complex and difficult process. Displacement-based procedures provide a more rational approach to these issues compared to force-based procedures by considering inelastic deformations rather than elastic forces. The analytical tool for evaluation process should also be relatively simple which can capture critical response parameters that significantly affect the evaluation process.

1.3 OBJECTIVE

The objective of this study is to evaluate the seismic performance of Bangladesh Secretariat Clinic Building, Dhaka.

1.4 OUTLINE OF THE THESIS

This thesis is composed of five chapters.

Chapter 1 provides a brief discussion on analysis methods used for seismic performance evaluation.

Chapter 2 gives details about time history analysis and modeling approach. It also provides an overview of the previous research on time history analysis.

In Chapter 3, the computational scheme, the assumptions involved in modeling nonlinear member behavior and underlying principles of **SiesmoStruct-v5.2.2 (2003)** utilized to perform time history analysis are explained in detail. The chapter also describes the various features of the case study building in detail.

Chapter 4 describes the procedure of selecting appropriate time history record for analysis & also describes the performance of the case study building under the selected time history motion.

Chapter 5 contains the summary and future recommendations of the study.

2 REVIEW OF PREVIOUS RESEARCH

2.1 GENERAL

Structures are expected to deform in elastically when subjected to severe earthquakes, so seismic performance evaluation of structures should be conducted considering post-elastic behavior. Therefore, a nonlinear analysis procedure must be used for evaluation purpose as post-elastic behavior cannot be determined directly by an elastic analysis. Moreover, maximum inelastic displacement demand of structures should be determined to adequately estimate the seismically induced demands on structures that exhibit inelastic behavior. Various simplified nonlinear analysis procedures and approximate methods to estimate maximum inelastic displacement demand of structures are proposed in literature. The widely used simplified nonlinear analysis procedure, pushover analysis, has also been an attractive subject of study.

2.2 INELASTIC TIME HISTORY ANALYSIS

- ❑ A full time history will give the response of a structure over time during and after the application of lateral dynamic loading.
- ❑ Time history analyses are required to define real seismic response of structure especially for irregular, highly ductile, critical or higher modes induced structures.
- ❑ With advances in seismic analysis and design of structures, nonlinear time-history analyses are becoming more common in civil engineering area.
- ❑ One of the most important issues for such analyses is the selection of acceleration time histories as input parameter.

2.2.1 MODELING APPROACHES FOR TIME HISTORY ANALYSIS

Nonlinear time-history analyses are a very powerful tool, provided they are supported by proper approximations and modeling. The analysis is inherently complex and may be very time-consuming, depending on the choice of the integration time-step, of the integration scheme, of the nonlinear incremental iterative algorithm strategy, and of the size of the mesh: an optimum balance among all these features will cater for accurate solutions with relatively reduced computational effort.

As described by Spacone (2001), a suitable classification of the different modeling strategies available may be based on the objective of the numerical study.

I. In Global Models (or Lumped Parameters Models), the nonlinear response of a structure is represented at selected degrees of freedom.

II. In Discrete FE Models (also called Member Models, or Structural Elements Models, or Frame Models) the structure is characterized as an assembly of interconnected frame elements with either lumped or distributed nonlinearities.

III. Microscopic Finite Element Models use the FE general method of structural analysis, in which the solution of a problem in continuum mechanics is approximated by the analysis of an assemblage of two or three-dimensional Fes which are interconnected at a finite number of nodal points and represent the solution domain of the problem.

The level of refinement of the model depends on the required accuracy and on the available computational resources. While refined FE models might be suitable for the detailed study of small parts of the structure, such as beam-column joints, frame models are currently the only economical solution for the nonlinear seismic analysis of structures with several hundred members. In other words, member FE models are the best compromise between simplicity and accuracy, as they represent the simplest class of models that nonetheless manage to provide a reasonable insight into both the seismic response of members and of the structure as a whole.

Assumptions and simplifications on the model with respect to the real structure are necessary, but need careful consideration because of their influence on results, which must be critically analyzed accordingly. In the particular case of bridges, for instance, the structural subsystems that may be potentially hit by intense seismic action are the deck, the bearing structure and the foundation system. Due to the cost and technical difficulties in its repairing, foundations are usually protected from damage, whilst for reasons of life safety, the deck is kept elastic (though cracking is inevitably allowed for). Indeed, the most common trend in earthquake-resistant design of bridges assigns therefore to the bearing structure, and by means of inelastic deformation mechanisms, a key role in dissipating the energy introduced by the earthquake loads, for which reason these are normally the elements requiring the most accurate modeling.

2.2.2 USE OF TIME HISTORY RESULTS

The expectation from Time History Analysis is to estimate critical response parameters imposed on structural system and its components as close as possible to those predicted by nonlinear dynamic analysis, such as:
These are;

- ❑Estimates of inter-story drifts and its distribution along the height.
- ❑Determination of force demands on brittle members, such as axial force demands on columns, moment demands on beam-column connections.
- ❑Determination of deformation demands for ductile members.
- ❑Identification of location of weak points in the structure (or potential failure modes).
- ❑Consequences of strength deterioration of individual members on the behavior of structural system.

- ❑Identification of strength discontinuities in plan or elevation that will lead to changes in dynamic characteristics in the inelastic range.
- ❑Verification of the completeness and adequacy of load path Pushover Analysis also expose design weaknesses that may remain hidden in an elastic analysis. These are story mechanisms, excessive deformation demands, strength irregularities and overloads on potentially brittle members.

2.3 PAST STUDIES ON TIME HISTORY ANALYSIS

With advances in seismic analysis and design of structures, linear and nonlinear time-history analyses are becoming more common in civil engineering area. One of the most important issues for such analyses is the selection of acceleration time histories as input parameter.

During the last two decades, a substantial research work has been done on the selection and scaling of earthquake records for use in dynamic analysis of structures. Most of this research is related to the requirements of the building codes for the use of time-history analysis in the design of building structures. Based on the literature review, there are two main approaches in this research. One approach is on the selection and scaling of real records, and the other approach is on the use of artificial accelerograms. The objectives of both approaches are to provide earthquake ground motions (i.e., accelerograms) which are compatible with the specified design spectrum (i.e., spectrum-compatible accelerograms).

Lew et al. (2008) discuss the challenges in the selection of earthquake accelerograms for use in the seismic design of tall buildings. They suggest that in order to cover the response effects of different modes, tall buildings need to be analyzed using many more ground motion accelerograms than the sets of three or seven accelerograms that are normally used in the current design practice for tall buildings.

Lestuzzi et al. (2004) discuss the selection of real ground motion records by considering the response of single-degree-of-freedom (SDOF) system with bilinear hysteretic model. The findings from this study are very limited, i.e., they are applicable only for building structures.

This can be modeled as a SDOF system. The response parameters considered are maximum displacement and ductility of the SDOF system. The study concludes that in the selection of real records for time history analysis, one should choose records with spectral accelerations that are close to the spectral acceleration of the design spectrum at the elastic (i.e., the initial) period, T0, of the SDOF system, or within the range between T0 and the period corresponding to the secant stiffness for either the expected ductility demand or the design ductility.

Malaga-Qhuquitaype et al. (2008) investigate various approaches for the selection and scaling of real records by considering the response of equivalent SDOF system. For a given building structure, an equivalent nonlinear SDOF system can be developed such the response of the system is similar to the roof displacement of the building considered. It is reported that selection and scaling of records that provide similar responses of the equivalent SDOF system are suitable for the nonlinear analysis of the building considered, i.e., the dispersion of the roof displacements of the building are quite small. This study is more general than that conducted by Lestuzzi et al. (2004).

While the method described in **Malaga-Qhuquitaype et al. (2008)** considers SDOF system, it is applicable for nonlinear analysis of actual multi-storey buildings, which is not the case with the method described in Lestuzzi et al. (2004).

Beyer and Bommer (2007) conducted an extensive study on the selection and scaling of real records for bi-directional analysis of buildings structures. The study considers the selection of the pair of ground motions rather than single component motions, as is the case with the majority studies on this subject. This is quite complex topic because it is known that the structural response depends significantly on the angle of incidence of the motion with respect to the structural axes. In order to reduce the number of analyses, it is suggested that for a given building, a simple model needs to be developed to determine the critical angle of incidence. It is concluded that "selecting records by matching with the target spectrum leads to smaller coefficients of variation (i.e., dispersion) of the structural response than if the records were selected according to an earthquake scenario defined in terms of magnitude and source-to-site distance". This is not surprising and has been observed in a number of previous studies.

Alimoradi et al. (2004) describe software for selection and scaling of real records based on the so-called General Algorithm, referred to as the GA method in their study. The computer program requires specification of the target spectrum, and selection criteria such as the range of scaling factors, number of records in the set to be selected, first mode period of the structure, T, and period range within which the spectra of the selected accelerograms should be close to the target spectrum (e.g., between 0.2T and 1.5, as required by ASCE 2006). From a given database, the program selects and scales a set of records (based on the selection criteria) such that the mean spectrum of the set has the smallest deviations around the target spectrum within the specified period range. The deviation from the target spectrum is measured by the mean square of the error between the square root of the sum of the squares (SRSS) of the average spectrum of the records and the target spectrum.

Katsanos et al. (2010) provide a detailed review of the available methods for the selection and scaling of real records. However, no time-history analyses have been done to see the effectiveness of the available methods, and it is not discussed which of the available methods are more appropriate for time-history analyses of building structures. Kalkan and Chopra (2010) propose a method for the scaling of real records to match the inelastic deformation of an equivalent nonlinear SDOF system, rather than to elastic design spectrum. The properties of the SDOF system are determined from pushover analysis of the building considered. The target inelastic deformation of the SDOF system is determined based on the spectral acceleration of the design spectrum for the first mode period of the building, multiplied by an empirical coefficient. It is concluded that the method is appropriate for selection and scaling of real records for use in nonlinear analysis. However the method seems quite complex for using in practice.

In addition to real accelerograms, simulated and artificial accelerograms have been used in seismic analysis of building **structures (Dancer 2003; Amiri-Hormozaki 2003; Tremblay and Atkinson 2001).** Scaling of such accelerograms can be done in the time or frequency domain. Naeim and Lew (1995) investigated the effects of the use of artificial accelerograms compatible with the design spectrum. The artificial accelerograms were obtained using scaling in the frequency domain. The study concluded that accelerograms scaled in the frequency domain are not appropriate for use in the seismic design since they might have unrealistic velocities, displacements, and energy content.

Most recently, **Atkinson (2009)** generated a comprehensive library of simulated accelerograms compatible with the NBCC 2005 design spectra for locations in eastern and western Canada. Because of the lack of recorded motions from Canadian earthquakes, it is expected that these accelerograms will be extensively used in the future.

Naumoski (2001) describes a method for the generation of spectrum-compatible accelerograms by modifying real accelerograms. These are referred to as the "modified real" accelerograms. The modification (i.e., the scaling) of a selected real accelerogram is conducted iteratively in the frequency domain until the spectrum of the modified accelerogram matches the specified target (i.e., design spectrum). A computer program (SYNTH) is developed based on this method.

Gasparini and Vanmarcke (1976) develop a computer program (SIMQKE) for the generation of artificial accelerograms compatible with a specified target spectrum. The characteristics of the target spectrum are included by spectrum density function, which is derived based on the design spectrum.

It should be mentioned that sets of accelerograms based on the methods proposed by **Atkinson (2009), Naumoski (2001), and Gasparini and Vanmarcke (1976)** are used in this study.

3 CASE STUDIES AND MODELING APPROACH

3.1 DESCRIPTION OF CASE STUDY BUILDING

Structure Name: Bangladesh Secretariat Clinic Building.
Location: Dhaka.
This structure is a 5 storey 4x3 bay clinic building. The full layout plan of the building has been shown in Fig. 3.1.

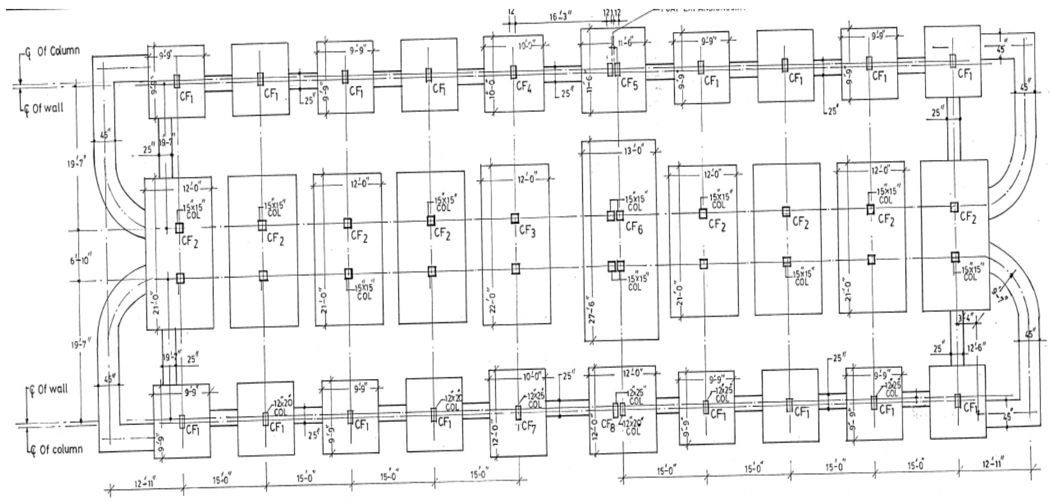

Fig. 3.1 Layout plan

The whole structure is composed of two buildings as shown in Fig. 3.1. For analysis purpose, the right building of the structure has been selected. The layout plan of the selected building is shown in Fig. 3.2.

The 3D building model in SeismoStruct is shown in the Fig. 3.3. The building was built at March 1985 & was designed as frame structure. Five types of columns & four types of beams were used in the Secretariat Clinic Building. Safe allowable bearing capacity of soil is assumed 2240lbs/sft.

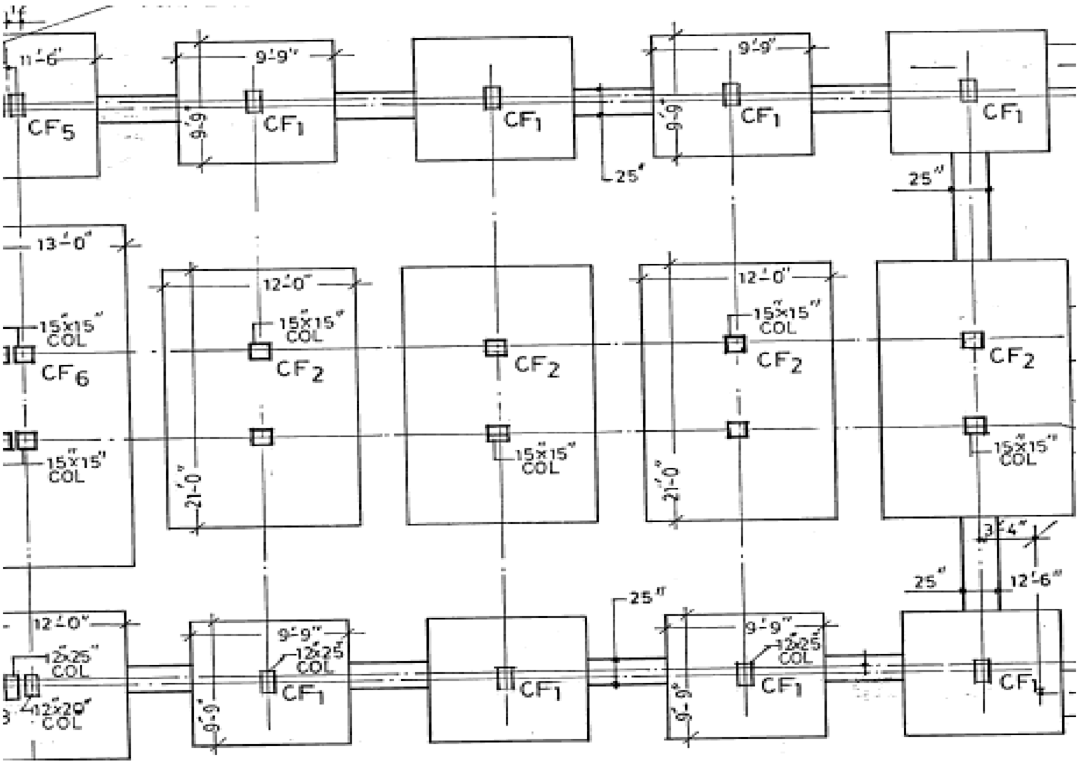

Fig. 3.2 Selected area for analysis

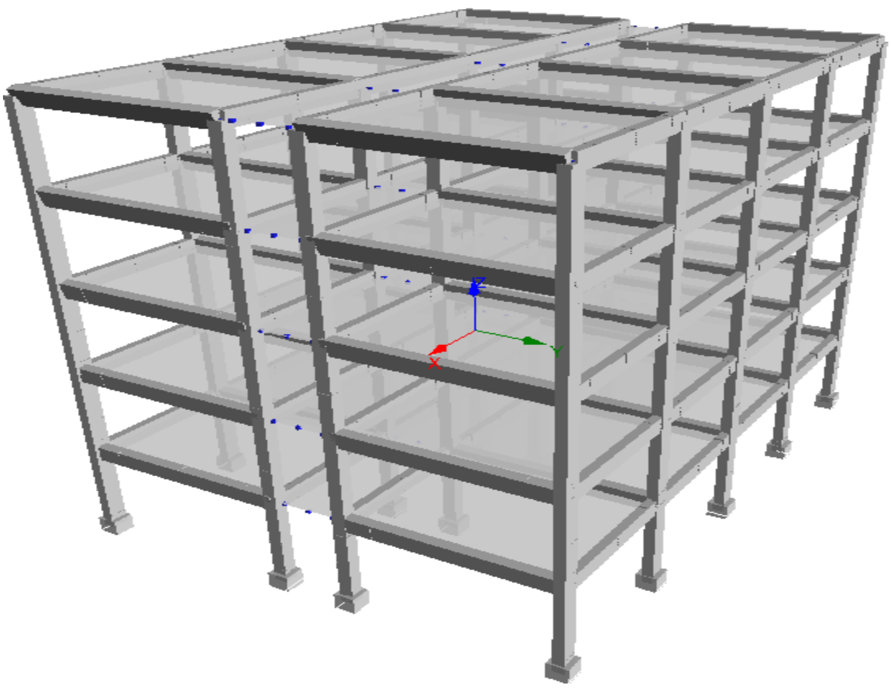

Fig. 3.3 3D model of Secretariat Clinic Building in SeismoStruct

The layout of beams & columns & also their cross-sectional details are shown in Fig. 3.4 to Fig. 3.13

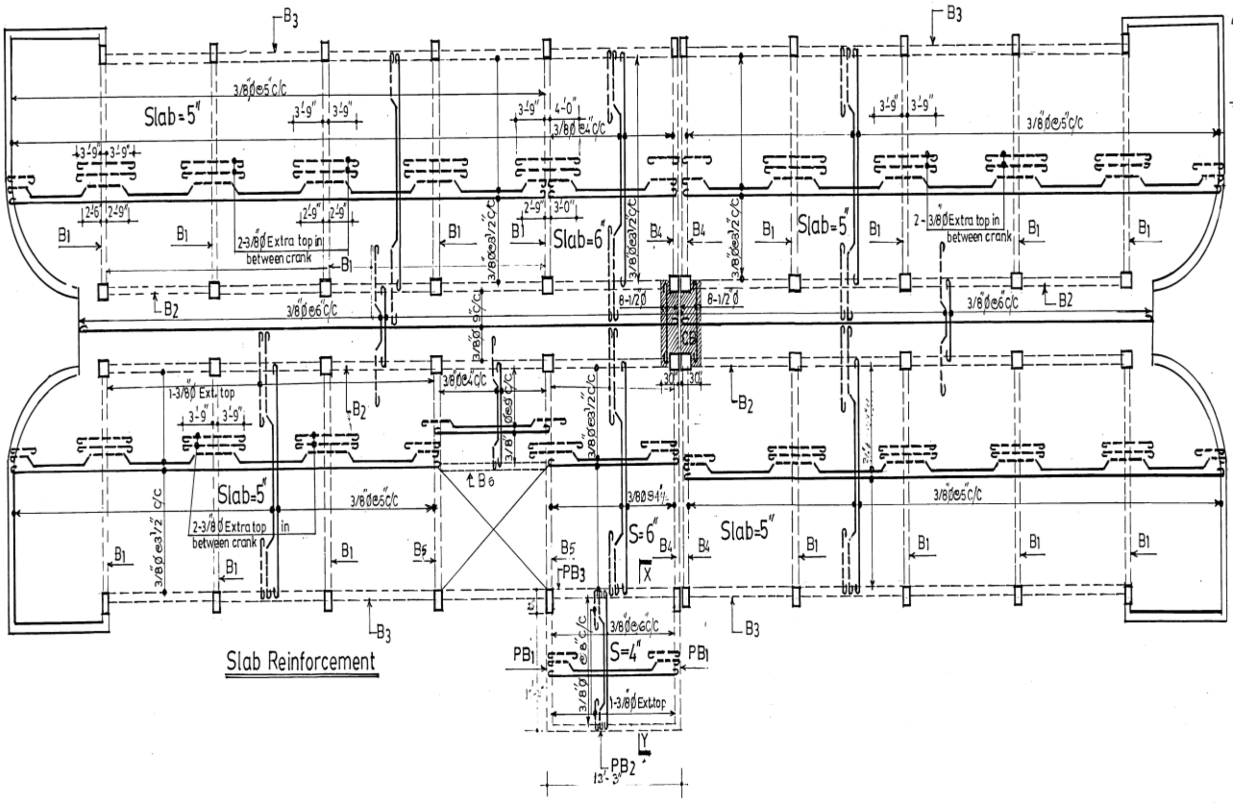

Fig. 3.4 Layout of beams & columns

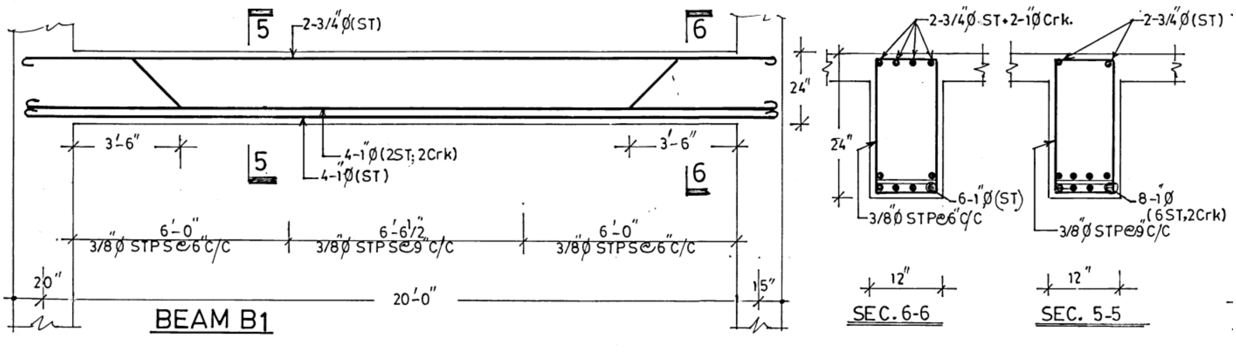

Fig. 3.5 Cross-section of beam-1

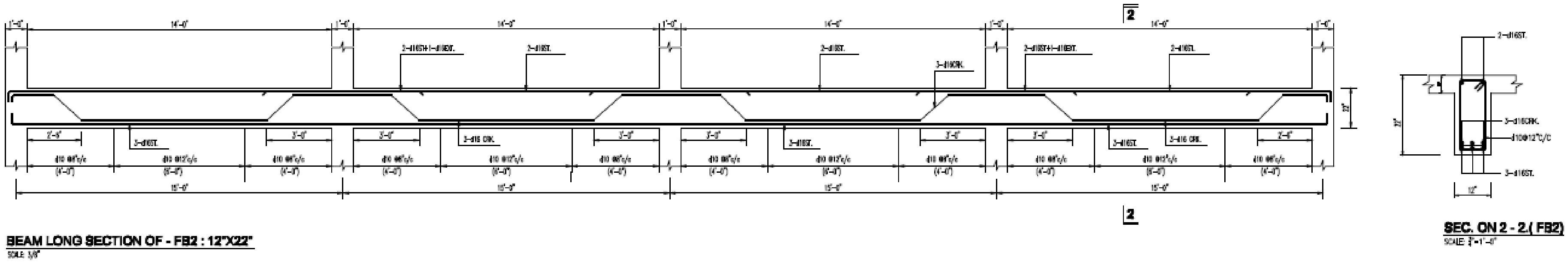

Fig. 3.6 Cross section of Beam-2

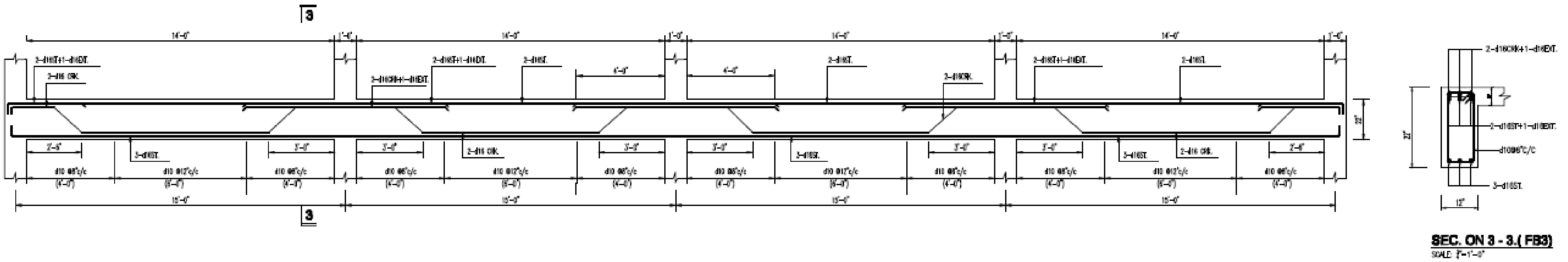

Fig. 3.7 Cross section of Beam-3

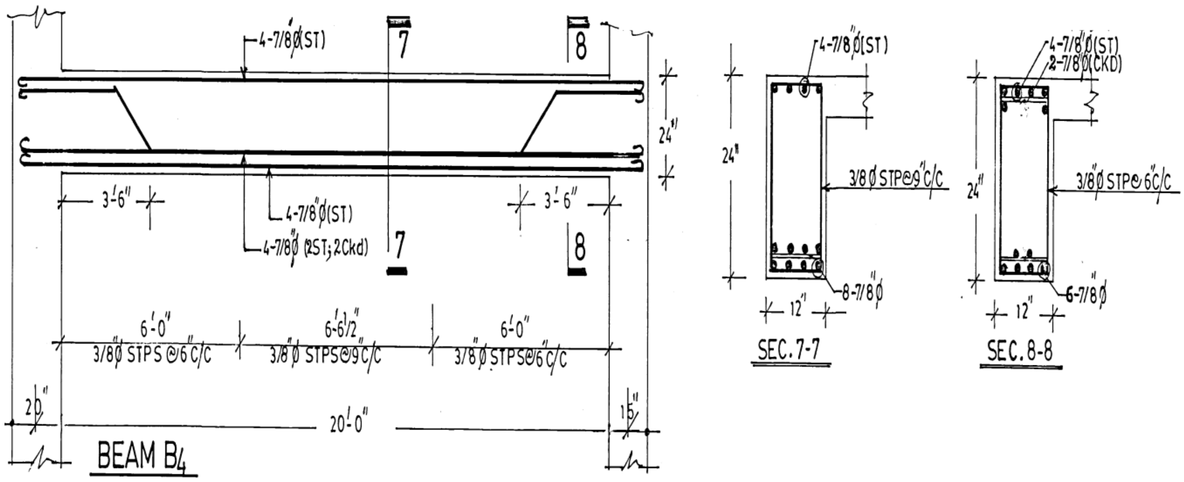

Fig. 3.8 Cross-section of beam-4

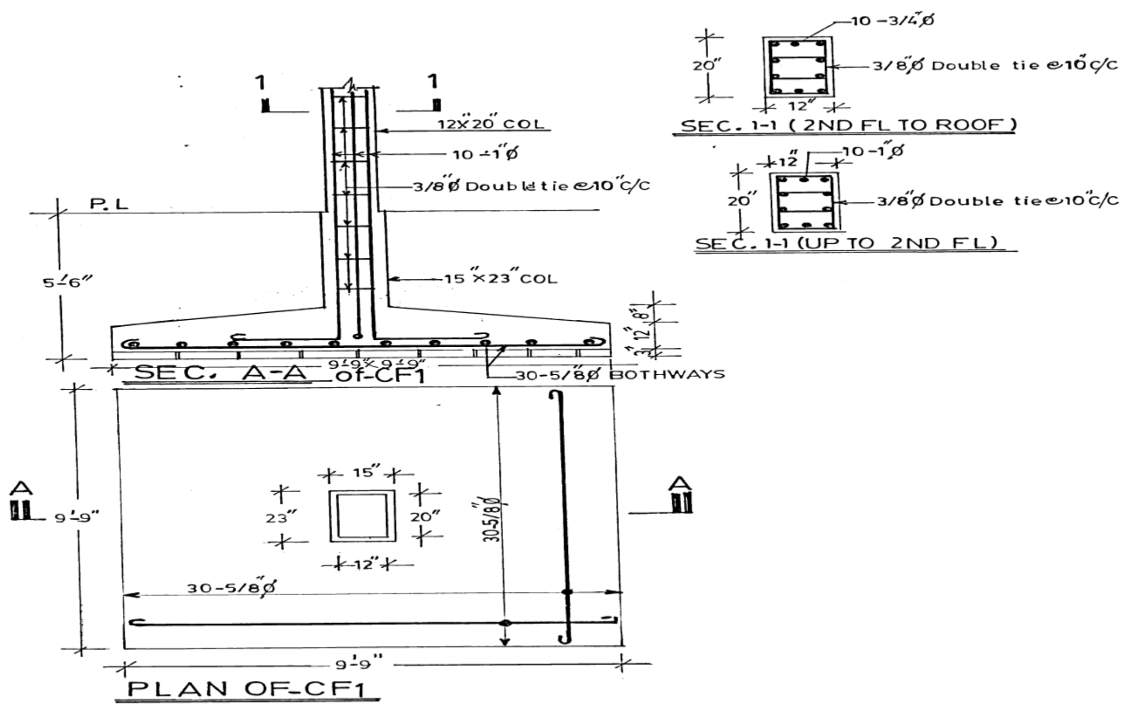

Fig. 3.9 Cross-section of column-1

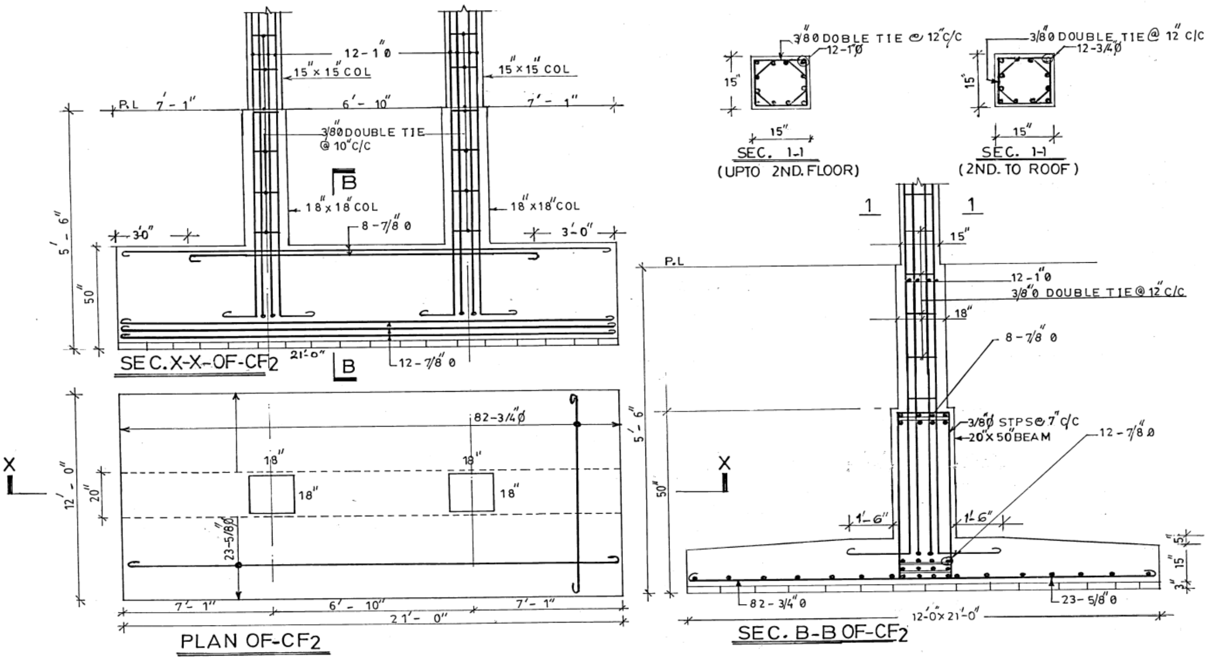

Fig. 3.10 Cross-section of column-2

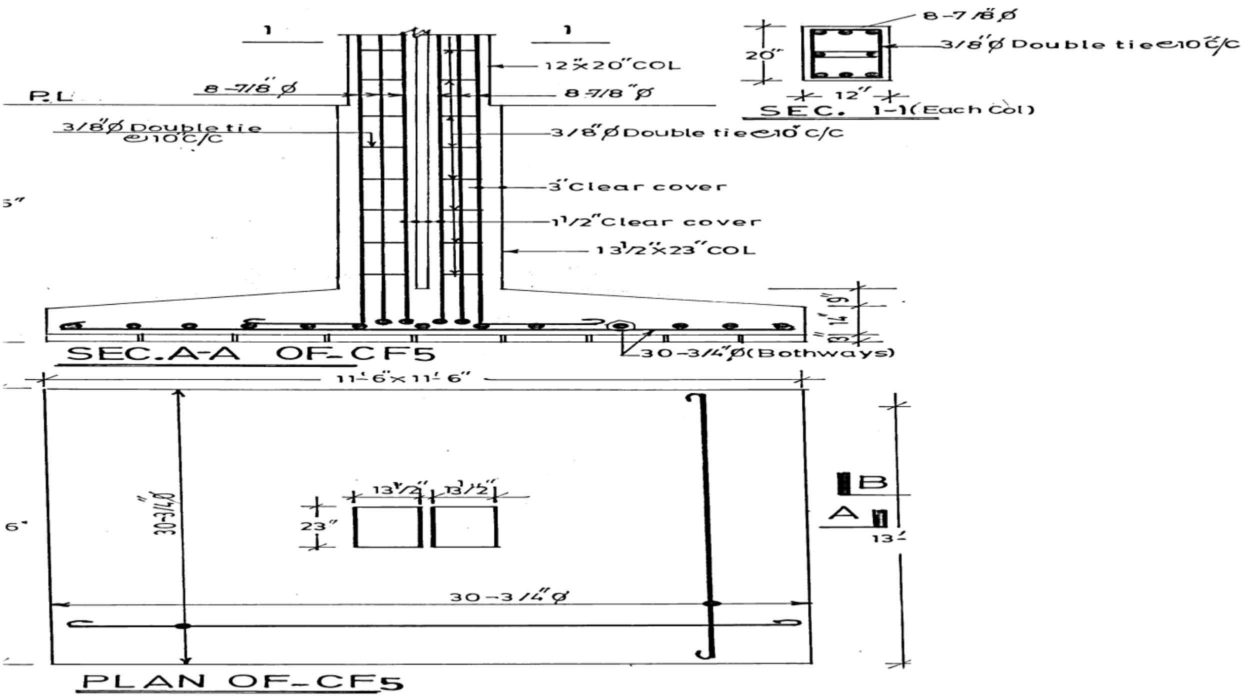

Fig. 3.11 Cross-section of column-5

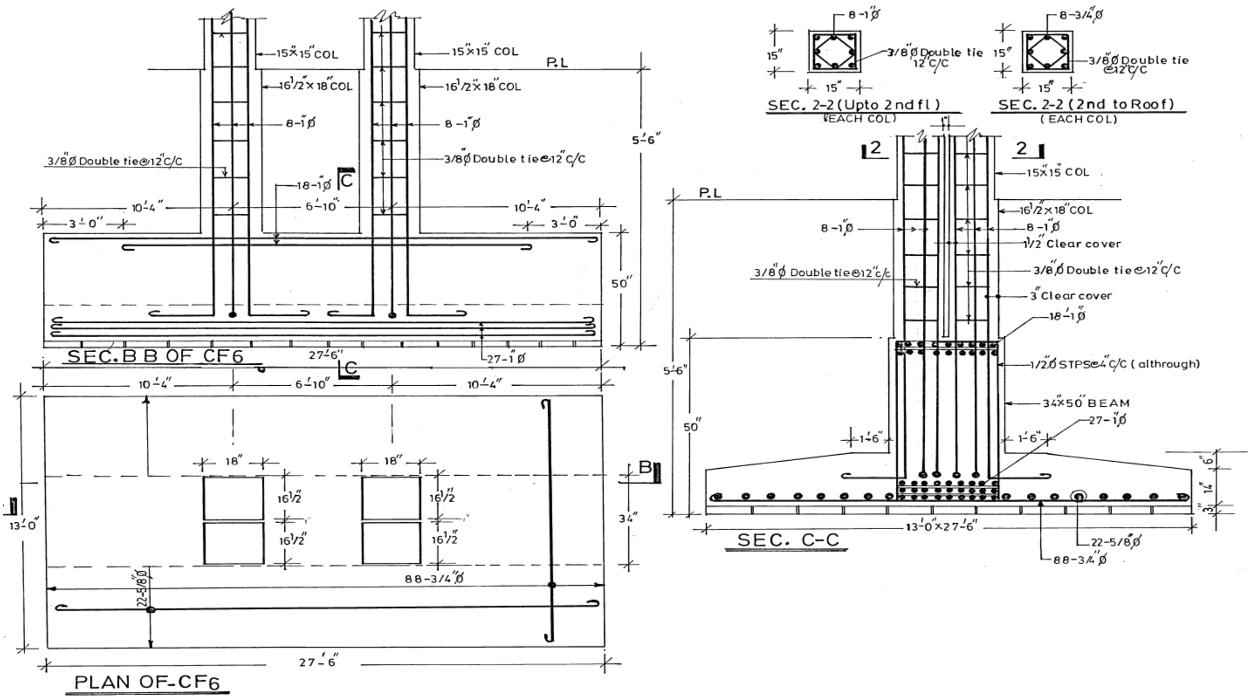

Fig. 3.12- Cross-section of column-6

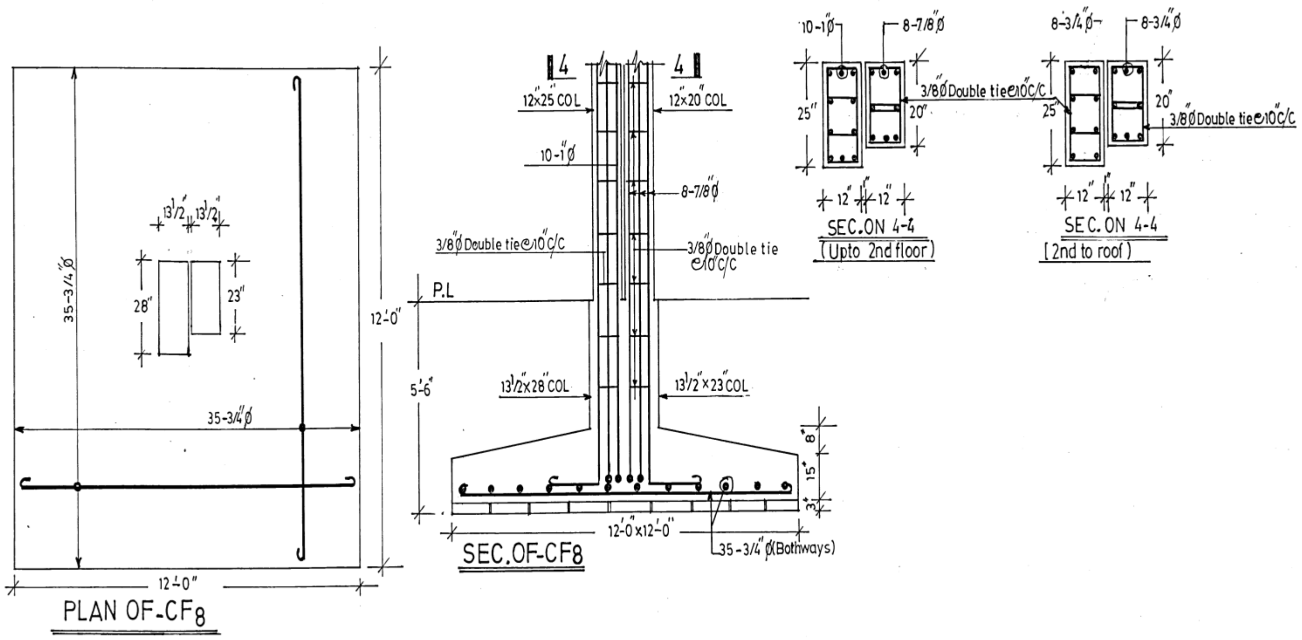

Fig. 3.13- Cross-section of column-8

3.2 LOAD DISTRIBUTION IN BEAMS AT DIFFERENT STOREYS

As per the input requirement of SeismoStruct, it was required to calculate the loads in all the beams of different storeys. The load distributions in the beams of different storeys are presented in Fig. 3.14 to Fig. 3.18. For load calculation, the live load & dead load as specified in Table 3.1 is used.

Table 3.1 Live load & dead load of the building

a)Load from slab	
i)Slab thickness(5 inch)	=5*120/12=62.5psf
ii)Plaster of ceiling(1/2 inch)	=0.5*120/12=5psf
iii)Floor finish	=25psf
Total dead load	=(62.5+5+25)=92.5psf
(b)Load from wall	
i)Unit load of 5” wall	=500lb/ft
ii)Unit load of 10” wall	=1000lb/ft
(c)Live load	=60*25% =15psf

Sample load calculation of beam shown in dotted line in Fig. 3.14:

Tributary partition wall length = 19.6/69.2 * 19.6 = 5.6ft
Direct wall length = 16.6ft

Load in beam:

i) For wall = [{(5.6+16.6) * 500}/19.6]
=566.32lb/ft

ii) For slab =WS/3{(3-m^2)/2} = 107.5 * 15/3{(3-0.765^2)/2} = 649lb/ft

Where,

W = (Live load + Dead load) in psf = (15+92.5) =107.5psf
S = Short span in ft = 15ft
L = Large span in ft = 19.6ft
m = S/L = 15/19.6 = 0.765

Therefore, total load in the selected beam = (566.32+649) = 1215.32lb/ft

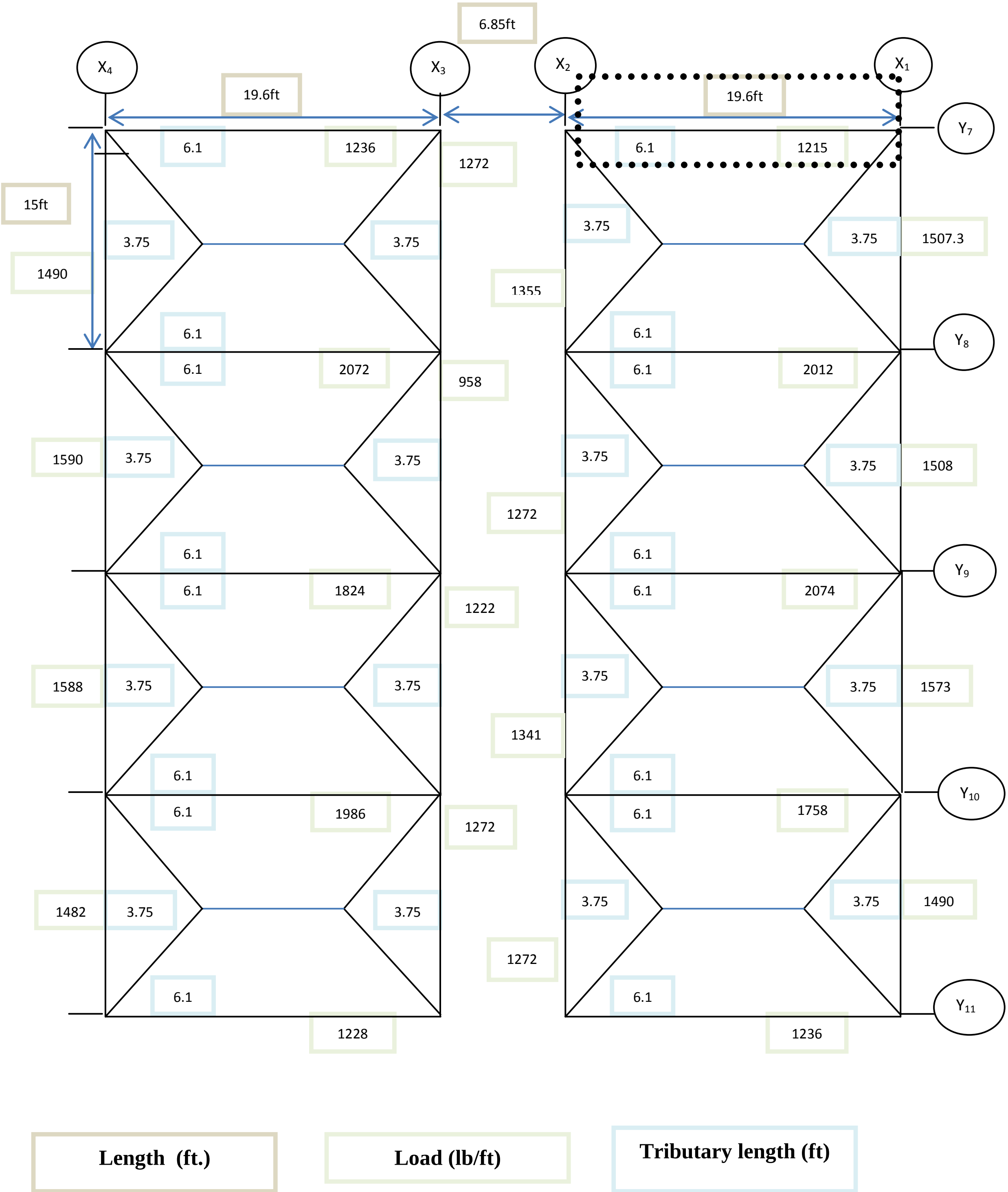

Fig. 3.14 Loads in beams at ground floor

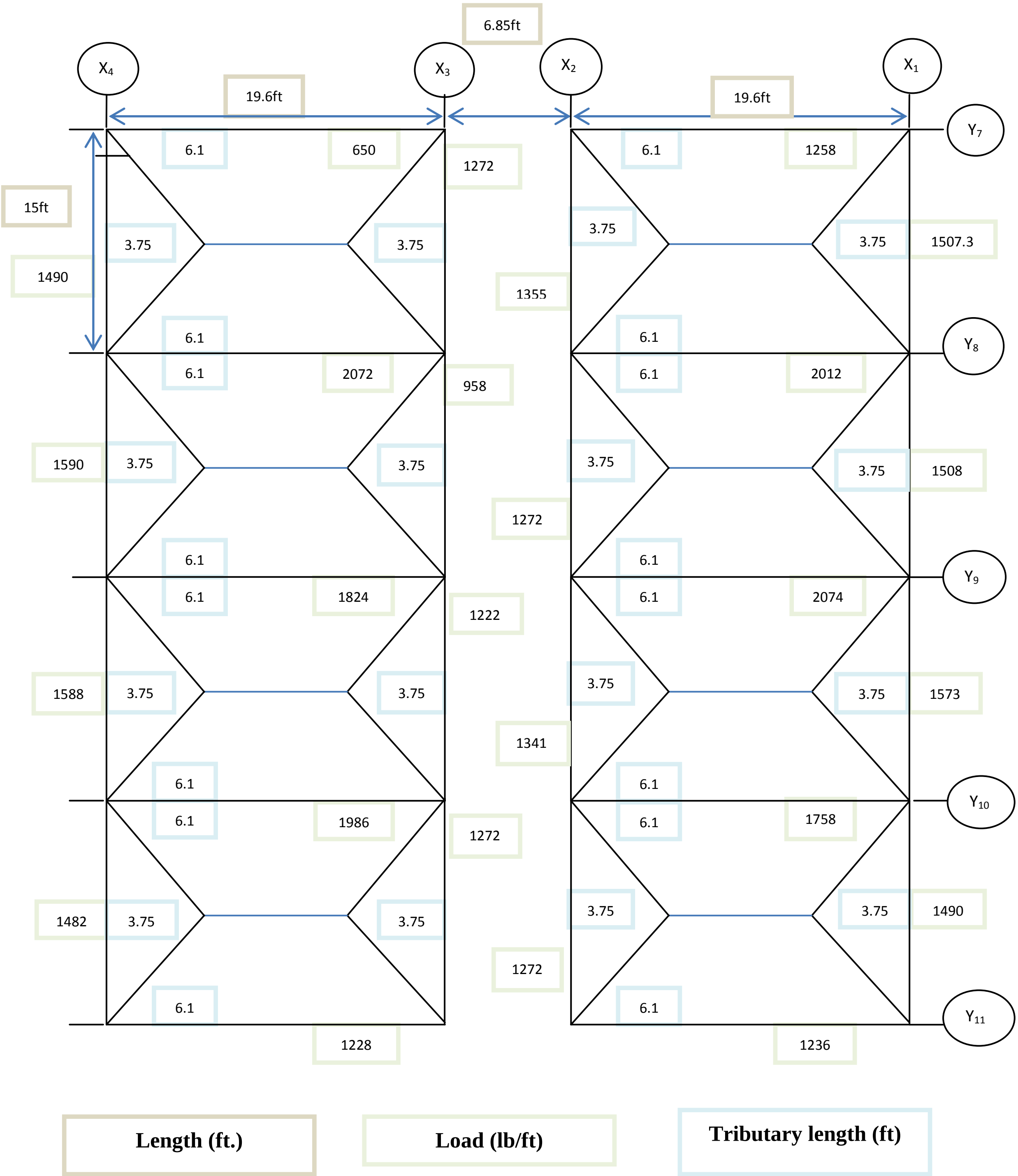

Fig. 3.15 Loads in beams at 1st floor

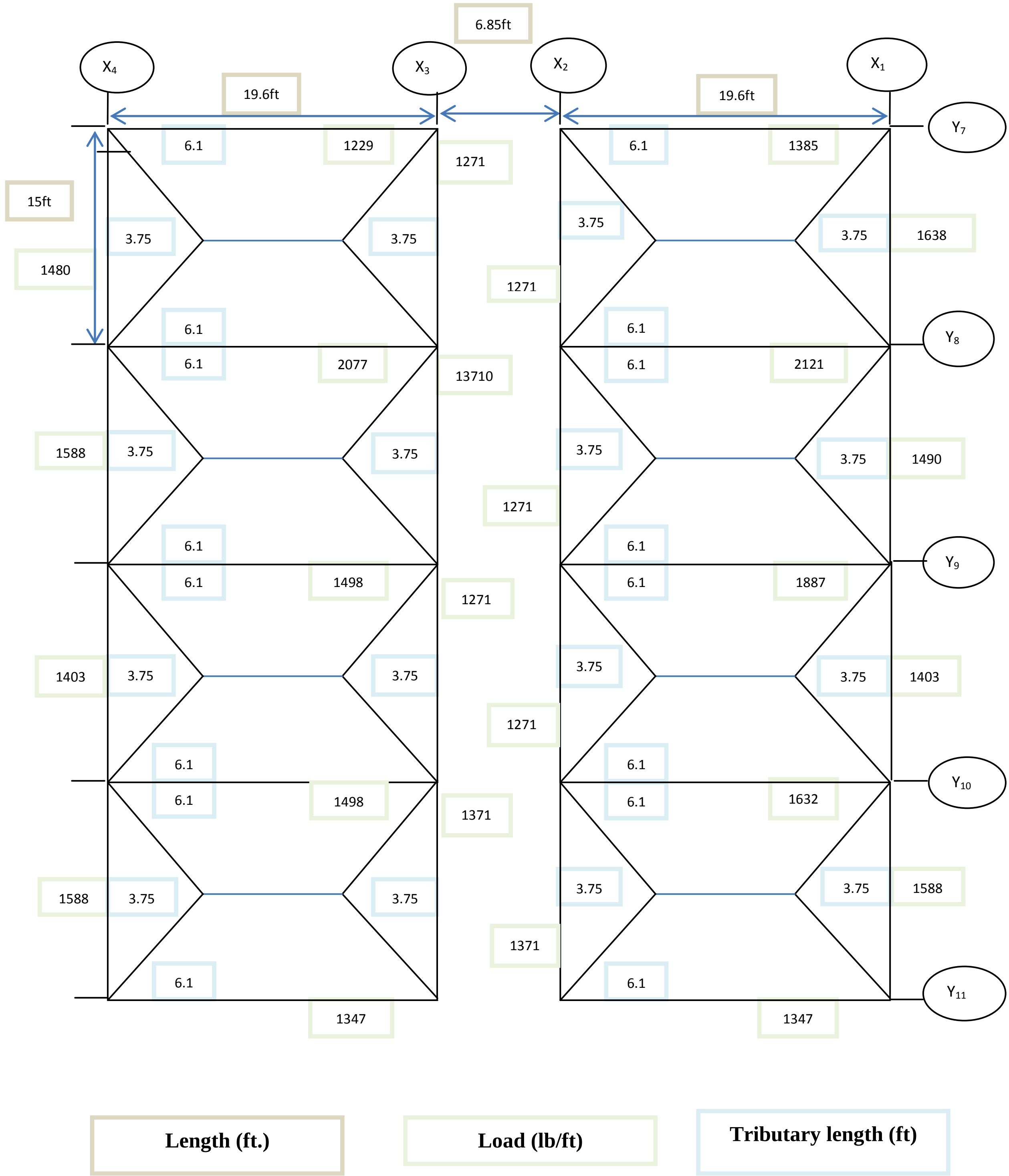

Fig. 3.16 Loads in beams at 2nd floor

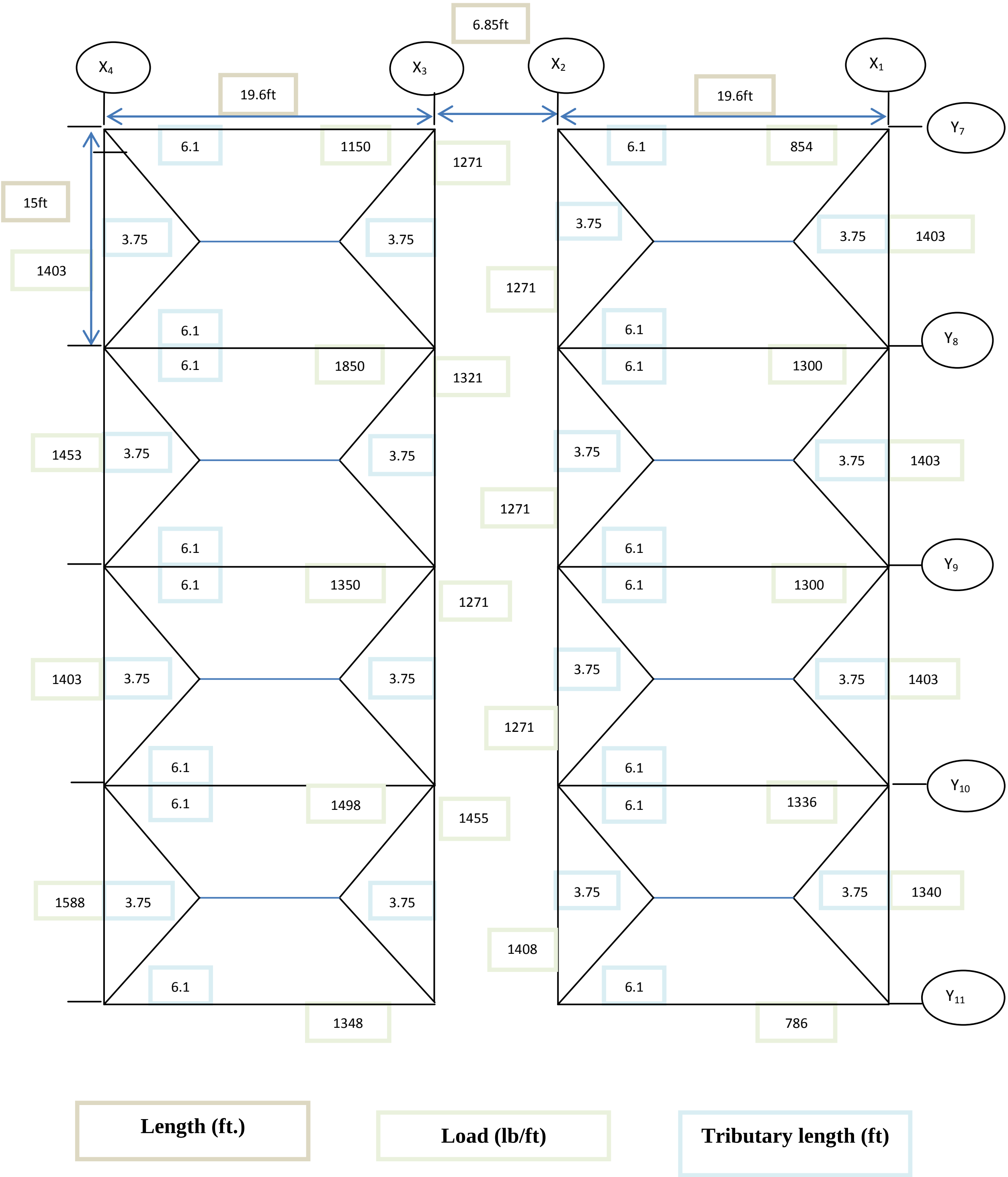

Fig. 3.17 Loads in beams at 3rd floor

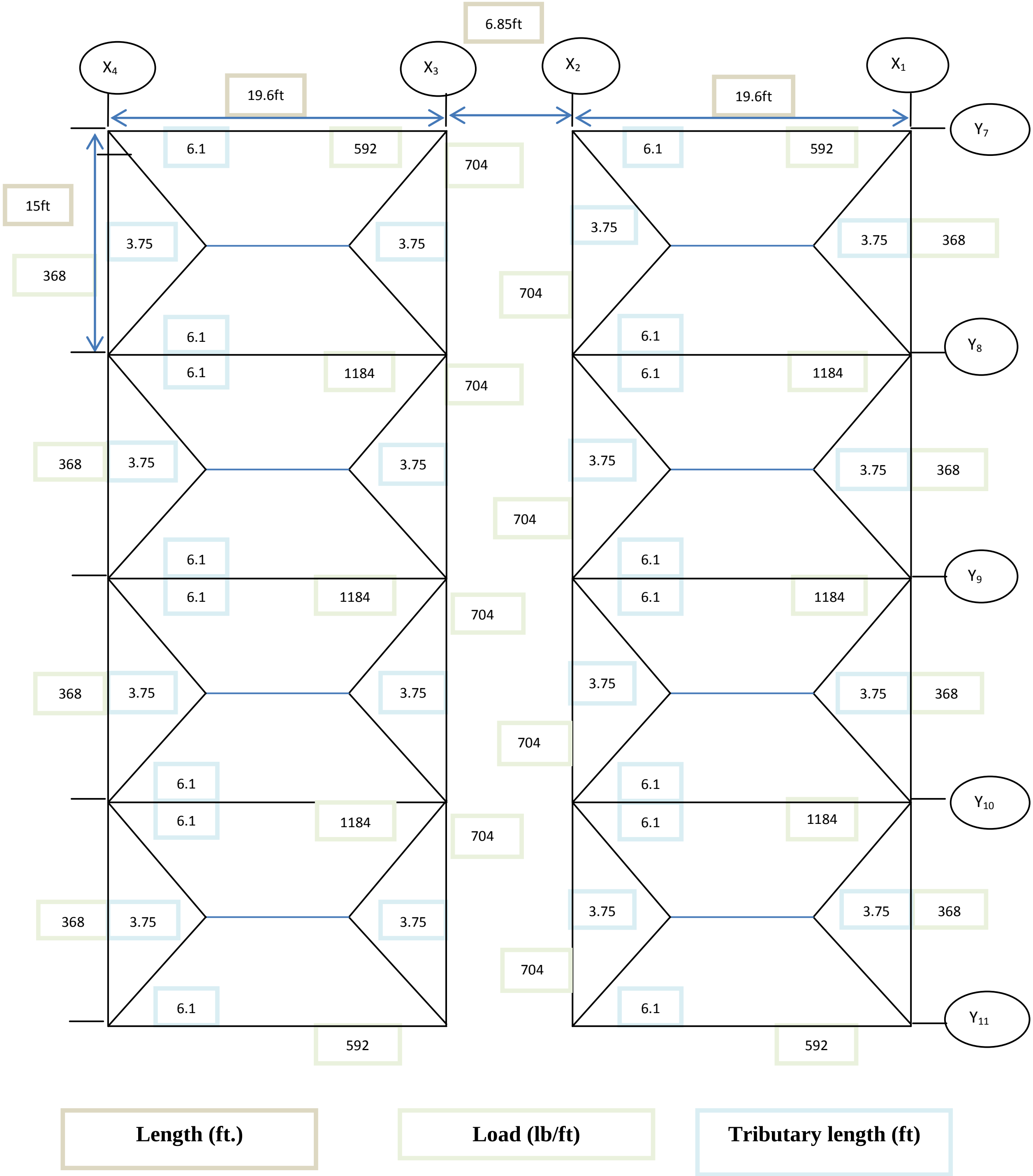

Fig. 3.18 Loads in beams at 4th floor

3.3 NON-LINEAR FINITE ELEMENT PACKAGE

The non-linear finite element program SeismoStruct-v5.2.2 (2011) has been chosen to model the frames and subsequently calculate their fundamental period. This package carries out distributed inelasticity fiber analysis as opposed to a concentrated plasticity approach present in ‘plastic hinge modeling’ programs. The program is capable of predicting the large displacement behavior of space frames under static or dynamic loading, taking into account both local and global geometric nonlinearities and material inelasticity.

3.3.1 INTRODUCTION TO THE PROGRAM

In SeismoStruct-v5.2.2 (2011), both local (beam-column effect) and global (large displacements/rotations effect) sources of geometric nonlinearity are automatically taken into account. Modeling of the latter is carried out through the employment of a co-rotational formulation (e.g. Izzuddin, 2001), whereby local element displacements and resulting internal forces are defined with regard to a moving local chord system. In this local system six basic degrees-of-freedom are employed ($\theta_{2(A)}$, $\theta_{3(A)}$, $\theta_{2(B)}$, $\theta_{3(B)}$, Δ, θ_T), as shown in Fig. 3.19. Exact transformation of element internal forces ($M_{2(A)}$, $M_{3(A)}$, $M_{2(B)}$, F, M_T) and the stiffness matrix, obtained in the local chord system, into the global system of coordinates allows for large displacements/rotations to be accounted for (e.g. Izzuddin, 1991) the interaction between

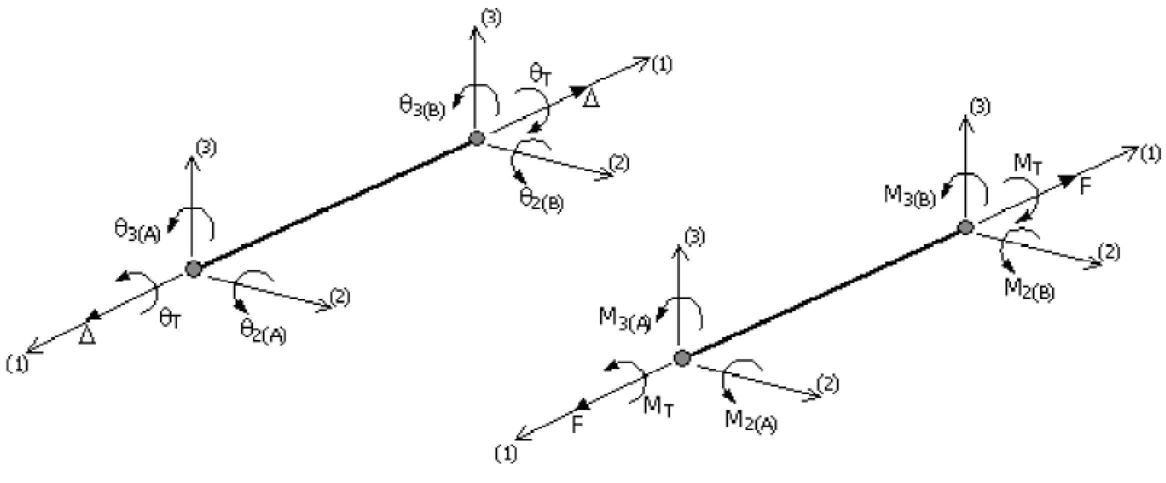

Fig. 3.19 Local chord reference systems (SeismoStruct-v5.2.2, 2011)

axial force and transverse deformation of the element (beam-column effect), on the other hand, is implicitly incorporated in the element cubic formulation suggested by Izzuddin (1991), whereby the strain states within the element are completely defined by the generalized axial strain and curvature along the element reference axis (x), whilst a cubic shape function is employed to calculate the transverse displacement as a function of the end rotations of the element:

$$u_2(x) = \left(\frac{\theta_{A2} + \theta_{B2}}{L^2}\right)x^3 - \left(\frac{2\theta_{A2} + \theta_{B2}}{L}\right)x^2 + \theta_{A2}x \tag{3.1}$$

$$u_3(x) = \left(\frac{\theta_{A3} + \theta_{B3}}{L^2}\right)x^3 - \left(\frac{2\theta_{A3} + \theta_{B3}}{L}\right)x^2 + \theta_{A3}x \tag{3.2}$$

The resulting elastic component of the stiffness matrix of the element, as defined in the local chord system (Izzuddin, 2001) is:

$$\frac{1}{L}\begin{bmatrix} 4EI_2 & 0 & 2EI_2 & 0 & 0 & 0 \\ 0 & 4EI_3 & 0 & 2EI_3 & 0 & 0 \\ 2EI_2 & 0 & 4EI_2 & 0 & 0 & 0 \\ 0 & 2EI_3 & 0 & 4EI_3 & 0 & 0 \\ 0 & 0 & 0 & 0 & EA & 0 \\ 0 & 0 & 0 & 0 & 0 & GJ \end{bmatrix}$$

Fig. 3.20 Elastic component of element stiffness matrix (SeismoStruct, 2011)

In Fig. 3.20, E denotes the modulus of elasticity, A is the cross-sectional area and I_2 and I_3 are the moments of inertia about the local axes (2) and (3). The torsional constant is denoted by J, whilst G stands for the modulus of rigidity, obtained as $G = E/(2(1+\nu))$, where ν is then Poisson's ratio.

Since a constant generalized axial strain shape function ($\Delta(x) = \Delta$) is assumed in the adopted cubic formulation, it results that its application is only fully valid to model the nonlinear response of relatively short members (Izzuddin, 1991) and hence a number of elements (3-4 per structural member) is required for the accurate modeling of structural frame members.

It should be noted that shear strains across the element cross-section are not modeled, thus the strain state of a section is fully represented by the curvature and centroidal strains alone (Izzuddin, 1991). Also warping strains and warping effects (cross-section distortion) are not considered.

The spread of inelasticity along the member length and across the section depth is explicitly modeled in SeismoStruct-v5.2.2 following a fiber modeling approach, thus allowing for an accurate estimation of damage distribution.

The sectional stress-strain state of inelastic beam-column frame elements is obtained through the integration of the nonlinear uniaxial stress-strain response of the individual fibres into which the section has been subdivided. The discretization of a typical reinforced concrete section is illustrated in Fig. 3.21. The user is required to define a sufficient number of fibres (about 200 is recommended for spatial analysis) and then the distribution of material nonlinearity across the section area is accurately modeled, even in the highly inelastic range.

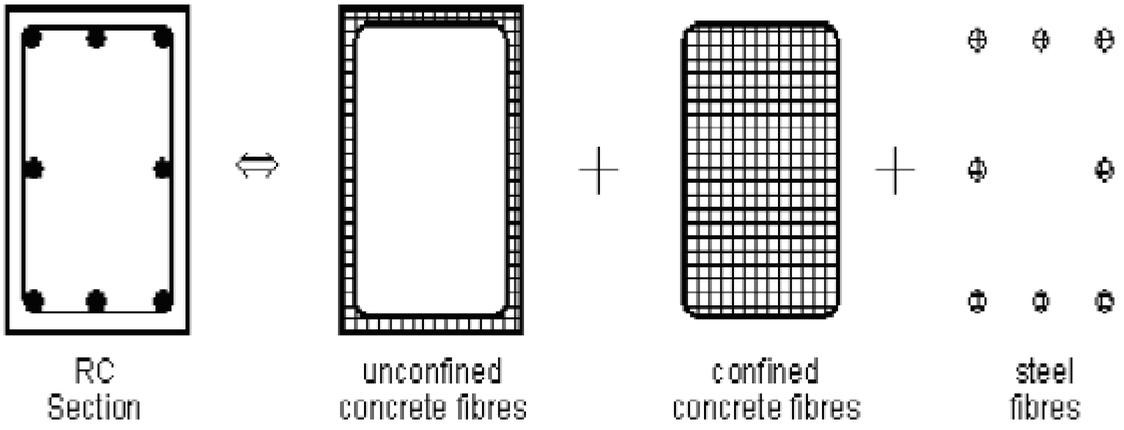

Fig. 3.21 Discretization of an RC section into fibres (SeismoStruct-v-5.2.2, 2011)

The spread of inelasticity along the member length arises as a result of the inelastic cubic formulation suggested by Izzuddin (1991), on which the beam-column elements within SeismoStruct are based. Two integration Gauss points per element are used for the numerical integration of the governing equations of the cubic formulation, as shown in Fig. 3.22. If a sufficient number of elements is used (5-6 per structural member), then the plastic hinge length of structural members subjected to high levels of material inelasticity can accurately estimate. It is evident that if the plastic hinges are to be accurately modeled, more elements should be defined where hinges are expected to form. The division of the member into shorter elements also renders valid the use of the cubic formulation to model nonlinear response, as mentioned previously.

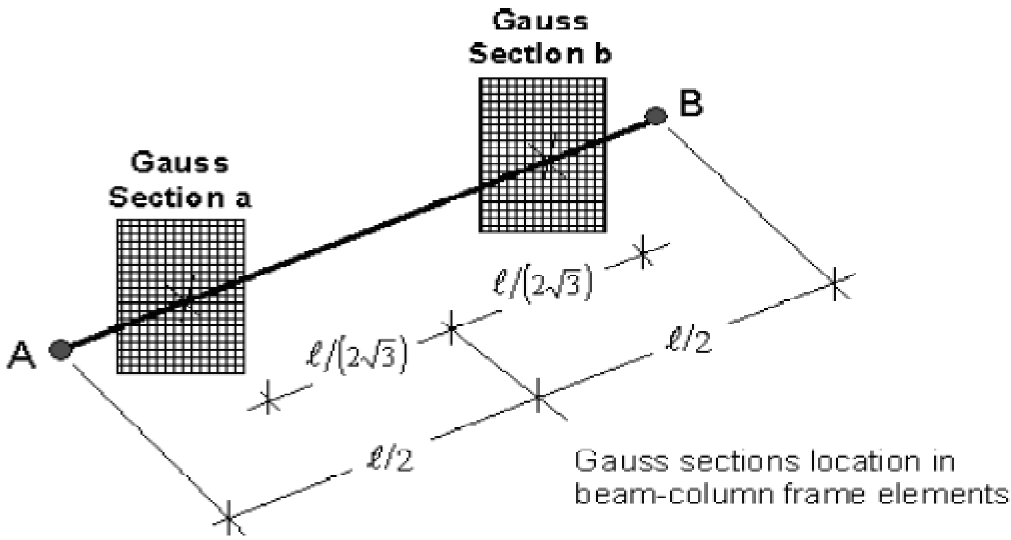

Fig. 3.22 Location of integration Gauss points within an element (SeismoStruct-v-5.2.2, 2011)

There are seven material models of steel and concrete available within the program library such that the user can define the material behavior to the required degree of accuracy, however it should be noted that more calibration is required for increasingly complex models. Steel models include a bilinear stress-strain model with strain hardening, a Menegotto – Pinto (1973) model which utilizes a damage modulus to represent more accurately the unloading stiffness under loading reversals (Fig. 3.23) and a Monti – Nuti (1992) model able to describe the Post-elastic buckling behavior of reinforcing bars.

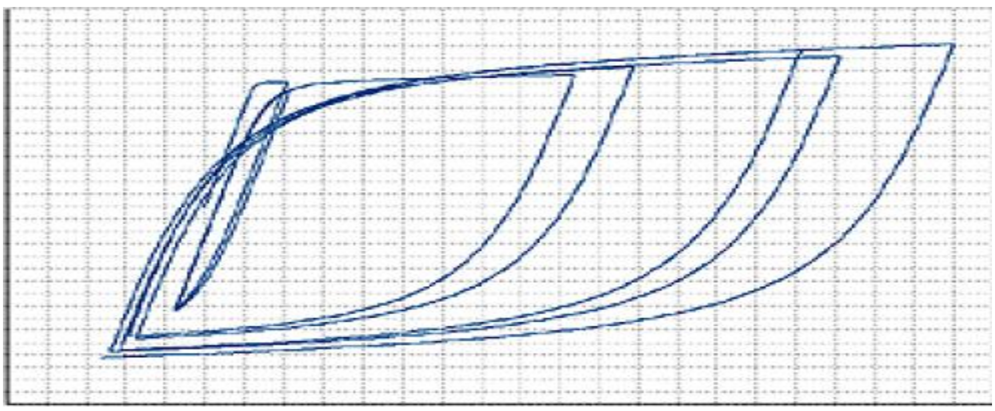

Fig. 3.23 Menegotto - Pinto (1973) steel model used in SeismoStruct-v-5.2.2, 2011

Concrete can be modeled most simply by a simple tri-linear model with no tensile resistance. More accurate models for normal strength concrete are available considering either constant confinement following the Mander et al (1988) model (see Fig. 3.24) or variable confinement as proposed by Madas and Elnashai (1992). A high-strength concrete model is also available as proposed by Kappos and Konstantinidis (1999) which allows for constant confinement modeling.

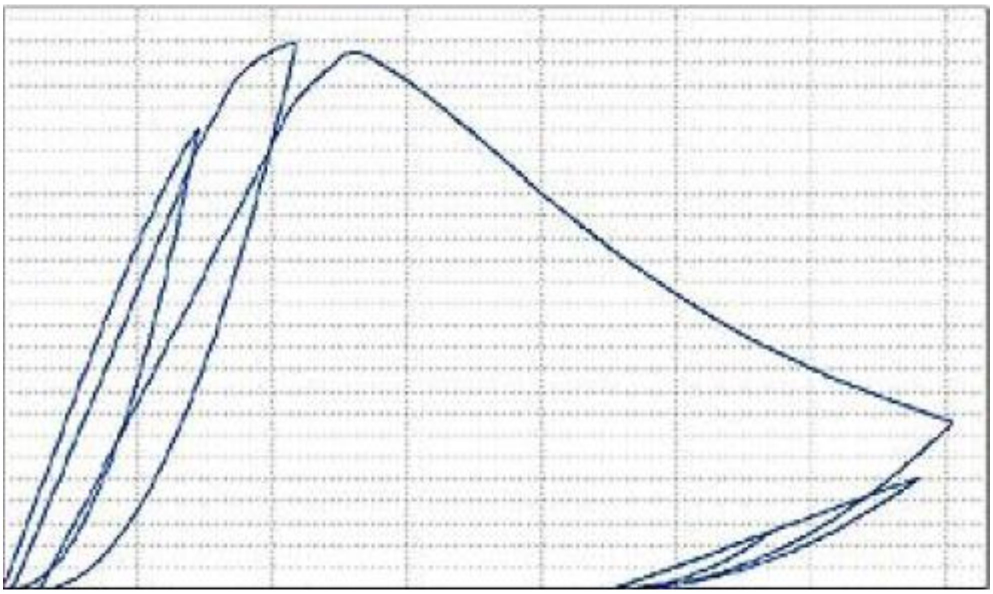

Fig. 3.24 Mander et al (1988) concrete model in SeismoStruct-v-5.2.2, 2011

3.3.2 MODELING PARAMETERS ADOPTED

The following parameters have been applied to all frames that have been modeled:

(1)The Mander et al (1988) constant confinement material model (see Fig. 3.24) was used for concrete. This requires the input of concrete compressive strength ($f_c^/$), tensile strength (f_t), strain at peak stress and a confinement factor. The confinement factor is defined as the ratio between the confined and unconfined compressive stress of the concrete, and is used to scale up the stress-strain relationship throughout the entire strain range. This material model was used for both confined and unconfined concrete, with the confinement factor for the latter being taken as 1.0. When information regarding the tensile strength and strain at peak stress was not available, the former has been calculated as $0.1f_c^/$ and the latter taken as 0.002.

The bi-linear stress strain with strain hardening model was used for steel (see Fig. 3.25). This model requires the definition of yield strength (f_y), modulus of elasticity (E), and a strain hardening parameter. The strain hardening parameter is the ratio between the post-yield stiffness (E_{sp}) of the material and the initial elastic stiffness (E_s). The former is defined as $E_{sp} = (f_{ult}-f_y)/(\varepsilon_{ult}-f_y/E_s)$, where f_{ult} and ε_{ult} represent the ultimate or maximum stress and strain capacity of the material, respectively. When information was not available for the modulus of elasticity and strain hardening parameter, the former was assumed to be 29000 ksi and the latter to be 0.005.

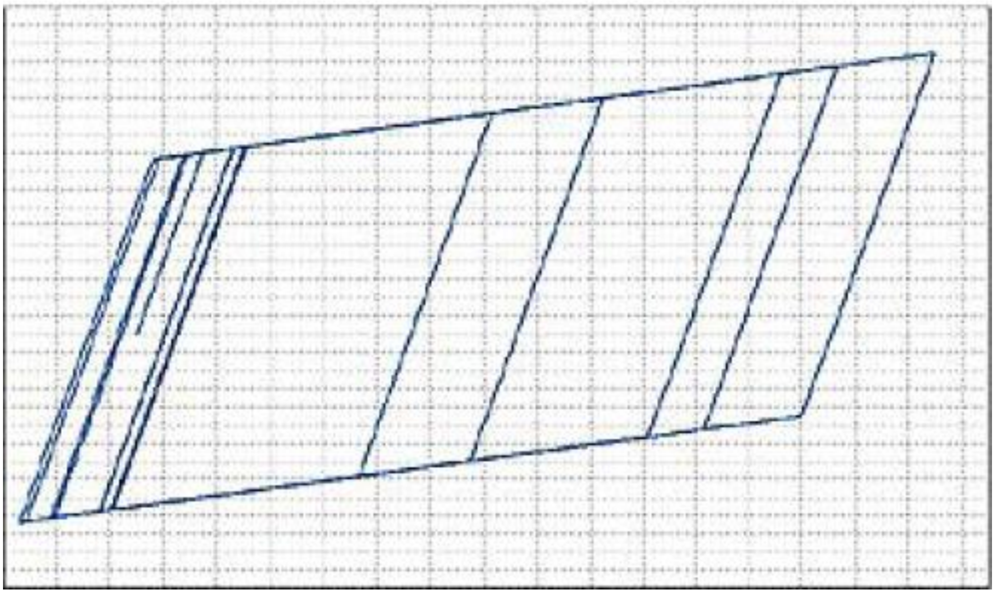

Fig. 3.25 Bilinear stress-strain steel models with strain hardening (SeismoStruct-v-5.2.2, 2011)

(2) 4-5 elements, with smaller elements at member ends, were used to model beams and columns to ensure inelasticity could be accurately modeled;

(3) The inertia was taken as the dead load plus approximately 25% of the live load;

(4) Beams and columns were modeled as extending from the center of one beam-column joint to the center of the next.

SeismoStruct-v-5.2.2, 2011 cannot model shear deformations however they need to be accounted for in both joints and members so that frame deformations can be accurately predicted. Within joints, alongside the shear deformation, there is also increased flexibility due to yield penetration and bar slip. A large proportion of the current building stock in Europe is considered to have been constructed with smooth bars, following the general building practice up until about 30 – 40 years ago. Therefore it is proposed that the inclusion of bar slip in the determination of yield stiffness is justified.

In this study, the increased member length of beams and columns has been assumed to account for the increased joint flexibility, until further research and calibration can be carried out. This would require the joint offset to be modeled with an elastic element and springs to be added where the beam connects to the joint to account for bar slippage and shear deformation. The shear deformation of the members may still not be accounted for in the model, however until further calibration is carried out the models will be used as defined above.

3.3.3 MEMBER LEVEL PERFORMANCE CRITERIA AND DAMAGE ASSESSMENT

Within the context of performance-based engineering, it is important to identify the instants at which different performance limit states (e.g. non-structural damage, structural damage, and collapse) are reached. This can be efficiently carried out in **SeismoStruct-v-5.2.2, 2011** through the definition of performance criteria, whereby the attainment of a given threshold value of material strain, section curvature, element chord-rotation and/or element shear during the analysis of a structure is automatically monitored by the program. Within the context of a fiber-based modeling approach, such as that implemented in **SeismoStruct-v-5.2.2, 2011**, material strains do usually constitute the best parameter for identification of the performance state of a given structure.

Two performance levels are defined for this study. The first performance level is related to moderate structural damage band as described in Displacement based earthquake loss assessment (DBELA) methodology (Crowley et al., 2004) in which member flexural strengths could be achieved, and some limited ductility developed, provided that concrete spalling in plastic hinges did not occur. For assessment of reinforced concrete buildings, limit values of ε_c = 0.0035-0.004 and ε_s = 0.01-0.015 are suggested. Note that similar values are normally considered appropriate for ultimate limit state when designing for gravity loads. The validity of these strain limits can be determined as follows. Typically, spalling of concrete is initiated at extreme fibre compression strains between ε_c= 0.006 and 0.01 (Calvi, 1999) .Thus, the limit of 0.004 is a conservative estimate of the onset of structural damage.

The strain limit of ε_s = 0.015 was determined to ensure that residual crack widths would not exceed 1.0 mm. The second performance level considered is related to extensive structural damage band in which significant repair required to building, wide flexural or shear cracks, buckling of longitudinal reinforcement may occur. This may correspond to local deformations in the critical section in the order of ε_c= 0.006-0.01 and ε_s = 0.03-0.04. It may be worth noticing that this limit state corresponds essentially to what is normally defined as an ultimate, or collapse limit state in most codes of practice. As per the guidance of Table 3.2, the following material strains listed in Table 3.3 are used at different limit states for analysis.

Table 3.2 Performance criteria for structural members

Performance Level	Material strain limit states	
	Concrete - ε_c	Steel - ε_s
Moderate –Member post- yield limit state	0.0035 – 0.004	0.01 – 0.015
Extensive – Member Collapse limit state	0.006 – 0.01	0.03 – 0.04

Table 3.3 Selected material strains

	Concrete ε_c	steel ε_s
Moderate-Member post-yield limit state	0.004	0.01
Extensive member collapse limit state	0.01	0.04

3.3.4 GLOBAL STRUCTURAL LEVEL DAMAGE ASSESSMENT

For global structural level, the monitored quantity is the base shear (V) versus top displacement (d) as shown in Fig. 3.26. Horizontal forces (V_i) of the support nodes were added and plotted against the horizontal displacement of the top floor.

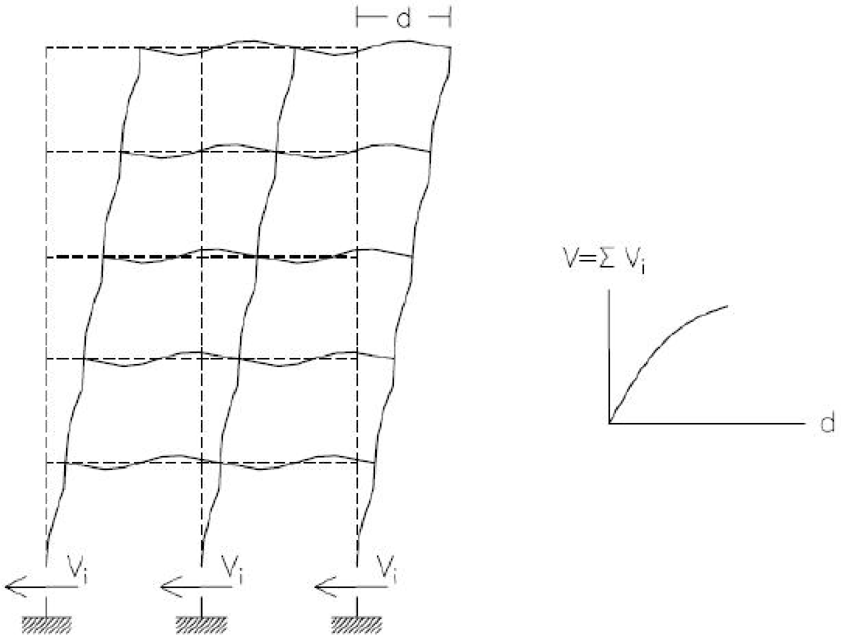

Fig. 3.26 Base-shear vs. global drift monitoring (after Papanikolaou*et al. 2005*)

3.3.5 GLOBAL YIELDING CRITERIA

Since the yield point is not clear in the plot of base shear versus top displacement, an idealized elasto-plastic system was assumed to find the approximate yield point in the global response of the structure. Following the procedures of EC8 (CEN 2003), the yield force F_y*, which represents also the ultimate strength of the idealized system, is equal to the base shear force at the formation of the plastic mechanism. The initial stiffness of the idealized system is determined in such a way that the areas under the actual and the idealized force – deformation curves are equal as shown in Fig. 3.27.

Based on this assumption, the yield displacement of the idealized system d_y* is given by:

$$d_y^* = 2\left(d_m^* - \frac{E_m^*}{F_y^*} \right)$$

Where E_m* is the actual deformation energy up to the formation of the plastic mechanism.

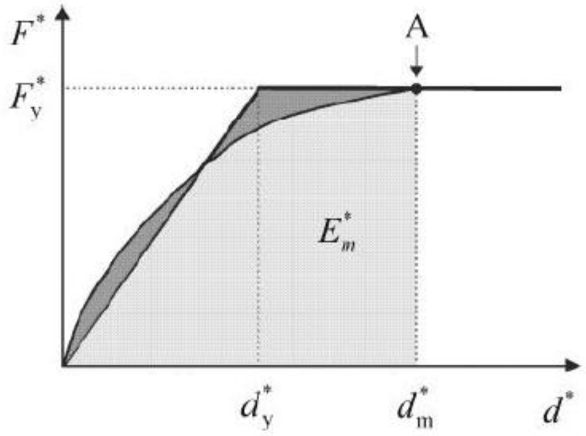

Fig. 3.27 Idealized elasto-plastic force displacement relationship according. (EC8, CEN 2003)

3.3.6 GLOBAL COLLAPSE CRITERIA

Significant strength and stiffness degradation can be a criterion of the collapse points on the pushover force displacement curves. According to the Turkish earthquake code (2007), a reliable measure of structural collapse can be defined when there is 30% loss of shear resistance at any storey level. This can be measured using the member collapse limit state by neglecting the columns that reached this limit state at its ends in resisting lateral loads until the load increment at which the limit of 30% reduction of shear resistance is reached. This loading step is considered as the collapse point.

3.3.7 INTER-STOREY DRIFT MONITORING

Lateral drifts are the main cause of structural damage in buildings subjected to earthquake ground motions. Additionally, lateral drifts are also responsible for earthquake-induced damage to many types of non-structural elements in buildings. Based on these observations, several recent studies have shown that present criteria for the seismic design of new structure sand for the seismic evaluation of existing structures can be improved if they are based on the explicit consideration of lateral deformation demands as the main seismic design parameter rather than lateral forces (Jeong and Elnashai 2004).

Jeong and Elnashai (2004) stated that during the preliminary design of new buildings or for a rapid seismic evaluation of existing buildings there is a need for estimating the maximum lateral displacements that can occur in the building. On the global structural level, the inter storey drift (ID) is one of the simplest and most commonly used damage indicators. For both global yielding as well as global collapse limits defined for each pushover capacity curve, the inter-storey drift (ID) profile is obtained. It should be noted that several ID values corresponding to collapse for a building have been suggested by different researchers. At values in excess of the collapse limit, it is assumed that significant P-Δ effect leads to failure of a building. An ID of 2.5% has been suggested by SEAOC (Structural engineers association of California) (1995) as the collapse limit for three-quarters of RC buildings as shown in Table 3.4.

Table 3.4 Performance levels and damage descriptions classified according to the ID ratio, (SEAOC – Vision 2000, 1995)

Performance level	Overall building damage	Transient drift
Fully operational	Negligible	ID < 0.2%
Operational	Light	0.2% < ID < 0.5%
Life Safety	Moderate	0.5% < ID < 1.5%
Near collapse	Severe	1.5% < ID < 2.5%
Collapse	Complete	2.5% < ID

4 PERFORMANCE EVALUATION OF CASE STUDY BUILDING

4.1 INTRODUCTION

Time history analyses are required to define real seismic response of structure especially for irregular, highly ductile, critical or higher modes induced structures. With advances in seismic analysis and design of structures, nonlinear time-history analyses are becoming more common in civil engineering area.

4.2 TIME HISTORY RECORD

One of the most important issues for time history analyses is the selection of acceleration time histories as input parameter. Unfortunately there is no strong motion record available in the literature applicable for Dhaka city buildings. But normalized response spectra for different soil profiles are available in the Bangladesh National Building Code (BNBC, 1993) as shown in Fig 4.1.

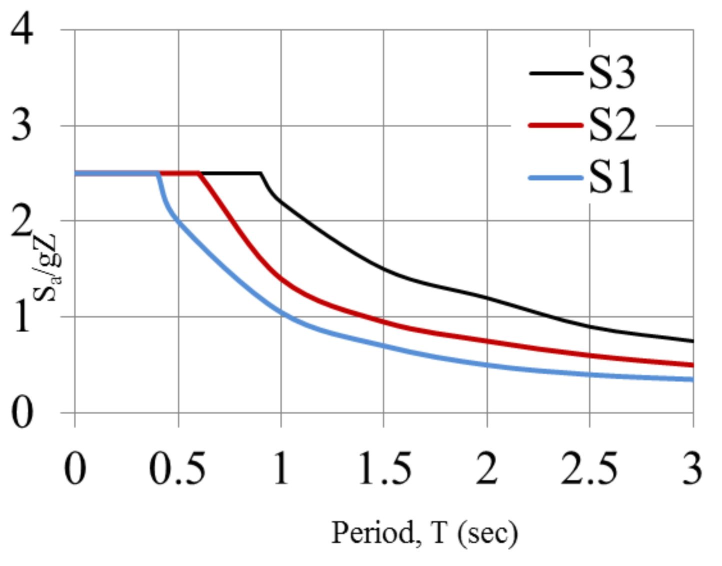

Fig. 4.1 Normalized response spectrum curve (BNBC, 1993)

As there is no strong motion record in the literature, it is required to back calculate it from response spectrum in order to get the time history analysis record. At first zone and soil profile have been selected according to the case study building location. The building is located in zone (Z2) and soil profile at the site of the building is (S2).So from BNBC, response spectra corresponding to seismic zone-2 (Z2) & soil profile-2 (S2) was selected as shown in Fig.4.2 & acceleration values in terms of g shown in Table 4.1.

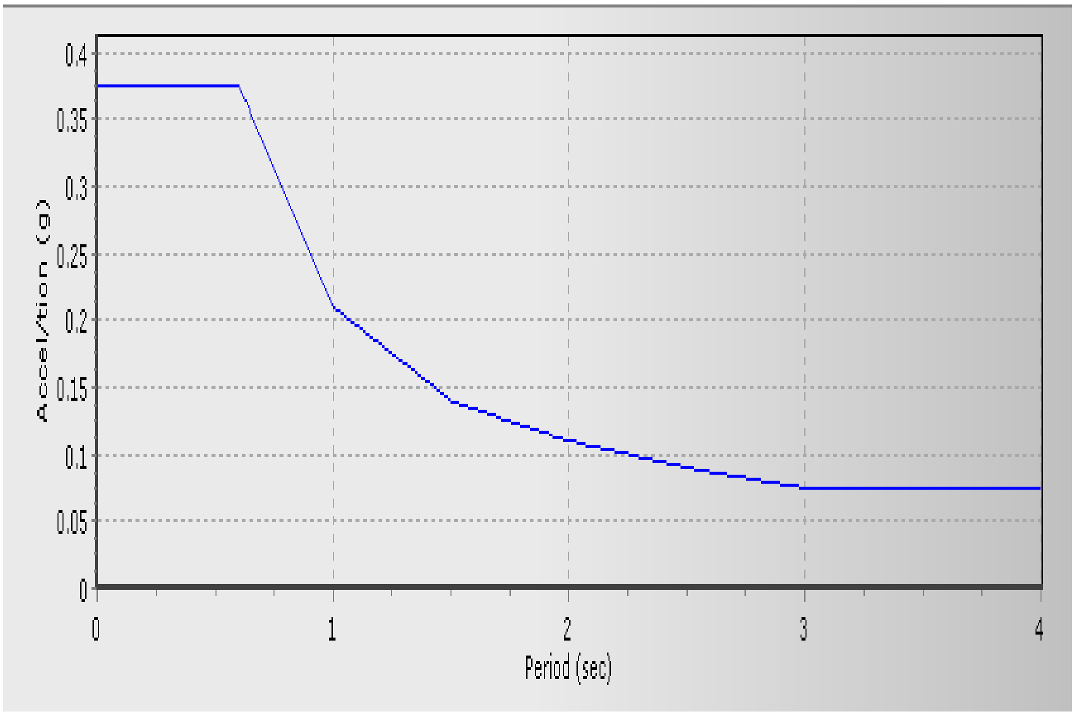

Fig. 4.2 Acceleration response spectrum (Sa/g) vs. time (T)

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Table 4.1 Time and response spectrum

S2,Z2

T	Sa/g
0	0.375
0.5	0.375
0.6	0.375
1	0.21
1.5	0.14
2	0.11
2.5	0.09
3	0.075

4.3 TIME HISTORY RECORD GENERATION BY SEISMOARTIFICIAL

4.3.1 BRIEF OVERVIEW OF SEISMOARTIFICIAL

SeismoArtif(2012) is an application capable of generating artificial earthquake accelerograms matched to a specific target response spectrum using different calculation methods and varied assumptions. It is noted that the use of real accelerograms and spectrum matching techniques, together with records selection tools, tends to be recommended for the derivation of suits of records for use in nonlinear dynamic analysis of structures. However, in those cases where access to real accelerograms is, for whatever reason, challenging or inappropriate, then a tool such as SeismoArtif will be of pertinence and usefulness.

This software can be used to generate suites of accelerograms for nonlinear dynamic analysis of new or existing structures. Users should refer to the literature for further discussion on the issue of random processes and artificial, as well as to publications on the topics of records appropriateness verification algorithms, selection of suites of records for nonlinear dynamic analysis of new or existing structures.

The program is capable of reading accelerograms and spectra saved in different text file formats. This collection of ground motion records and spectra is then used in the simulation phase for the definition of target spectrum or envelope shapes.

Finally, due to its full integration with the Windows environment, SeismoArtif allows for numerical and graphical results to be copied to any Windows application (e.g. MS Excel, MS Word, etc.), noting that the characteristics of the plots can be fully customized from within the program itself.

4.3.2 TIME HISTORY RECORD GENERATION

The target response spectrum as shown in Fig. 4.3 has been used as an input parameter in the SeismoArtificial software. At first, in SeismoArtificial software, 30 seconds of duration has been taken for each of the accelerograms although more or less duration could have been selected. The damping of 5%, time step of 0.01 sec, smallest period of desired response spectrum of 0.02 sec, largest period of response spectrum of 3.00 sec have also been defined & 5 numbers of artificial accelerograms have been generated as shown in the Fig. 4.3 to Fig. 4.10. Amongst the five generated accelerograms, the accelerogram whose response spectrum is closest to the target response spectrum was selected as the input time history for performance evaluation analysis by SeismoStruct. Here the no-2 accelerogram has been taken because of its response spectrum being closer to the target response spectrum and this strong motion has been used as an input parameter in the SeismoStruct-v5.2.2, 2011 software.

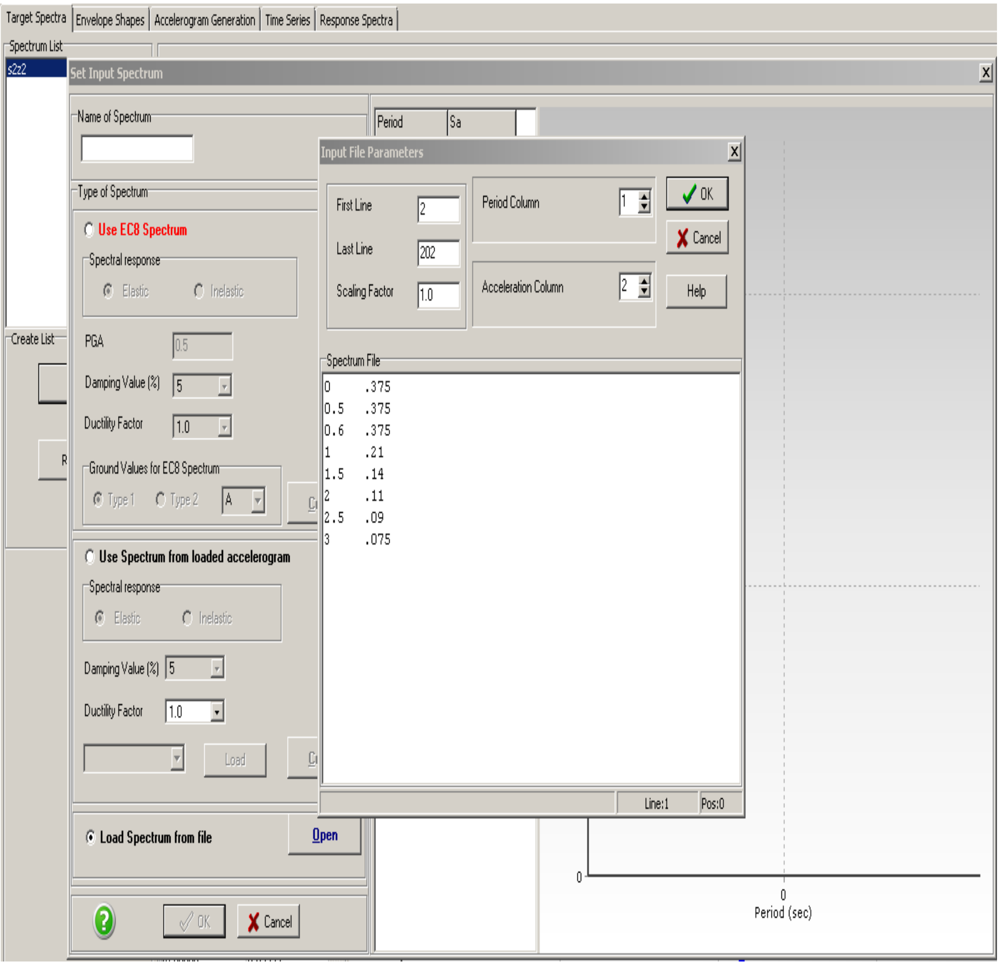

Fig. 4.3 A view of putting the input response spectrum in SeismoArtificial software

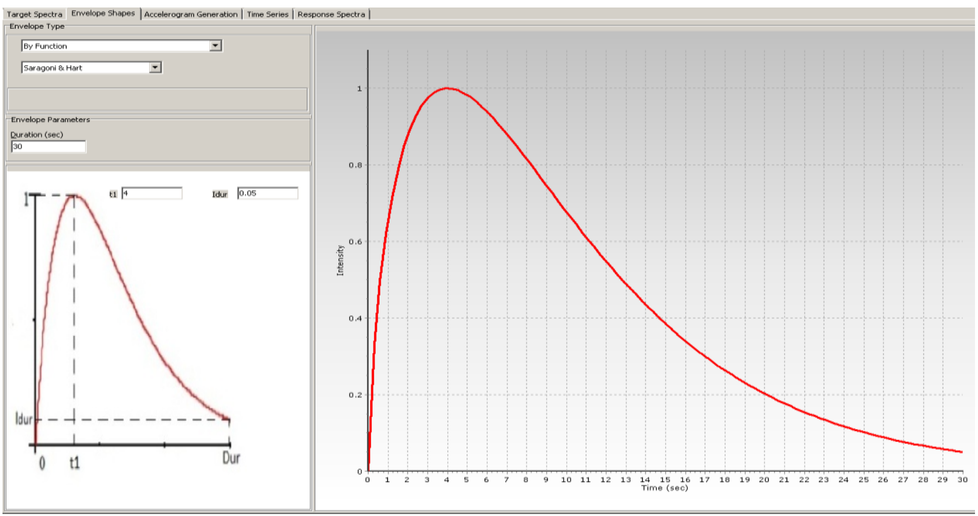

Fig. 4.4 A view of putting duration for the output time history data in SeismoArtificial software

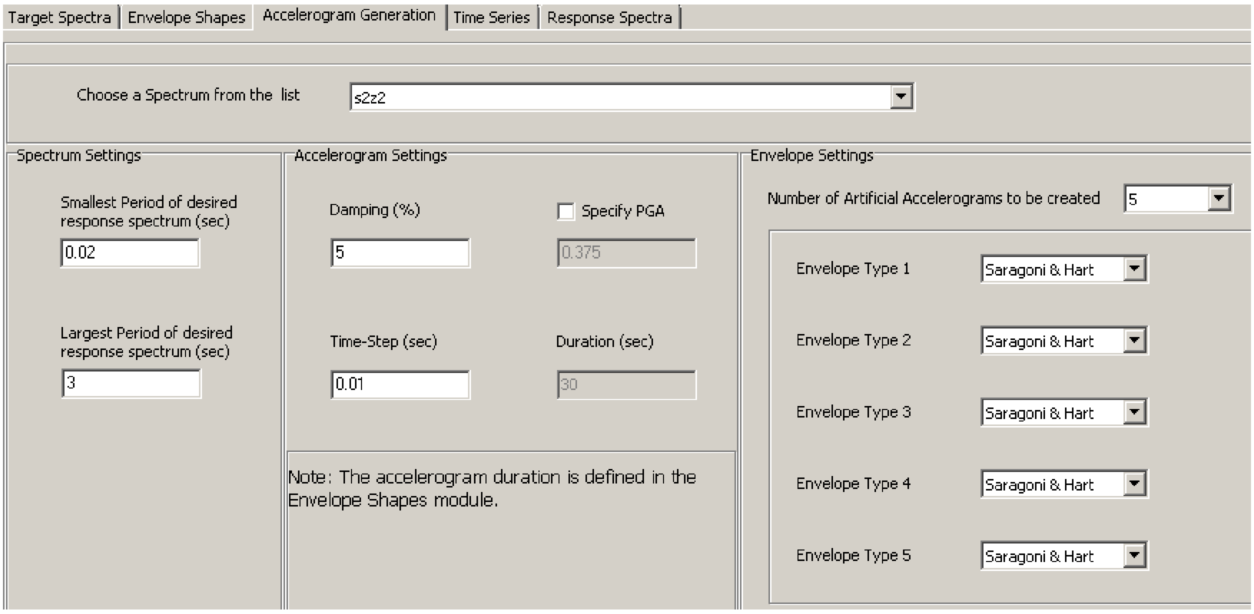

Fig. 4.5 A view of putting the No. of accelerograms

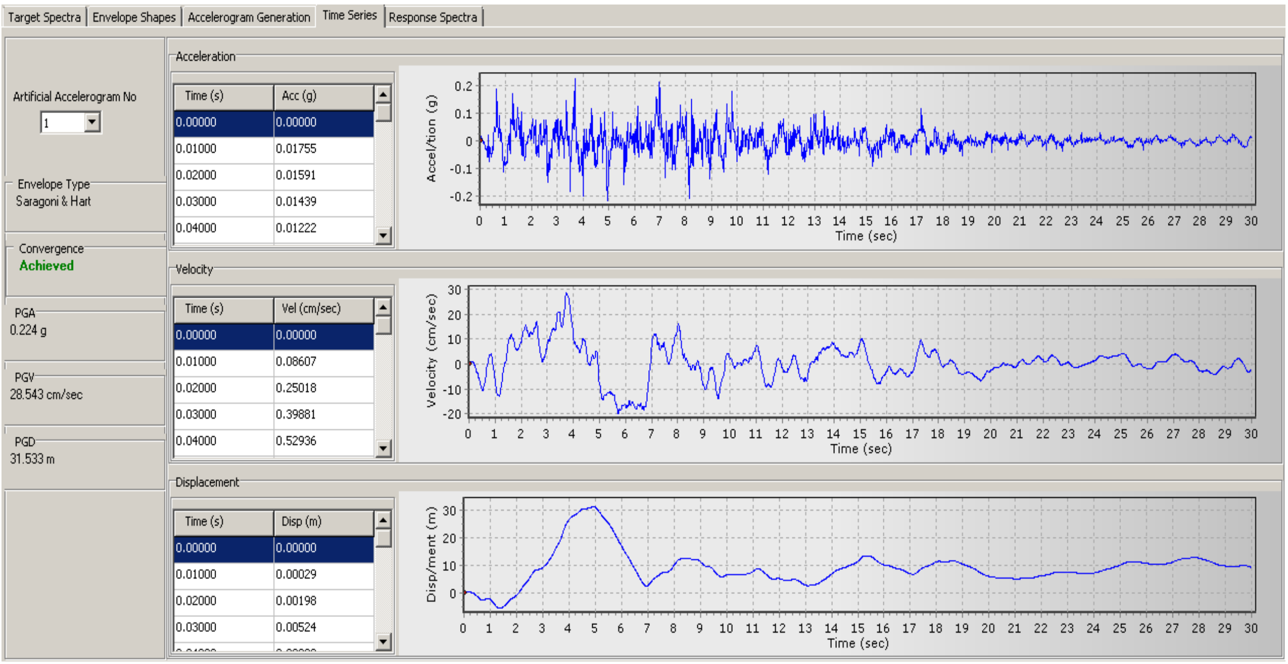

Fig. 4.6 Time History record -1 or artificial accelerograms (1)

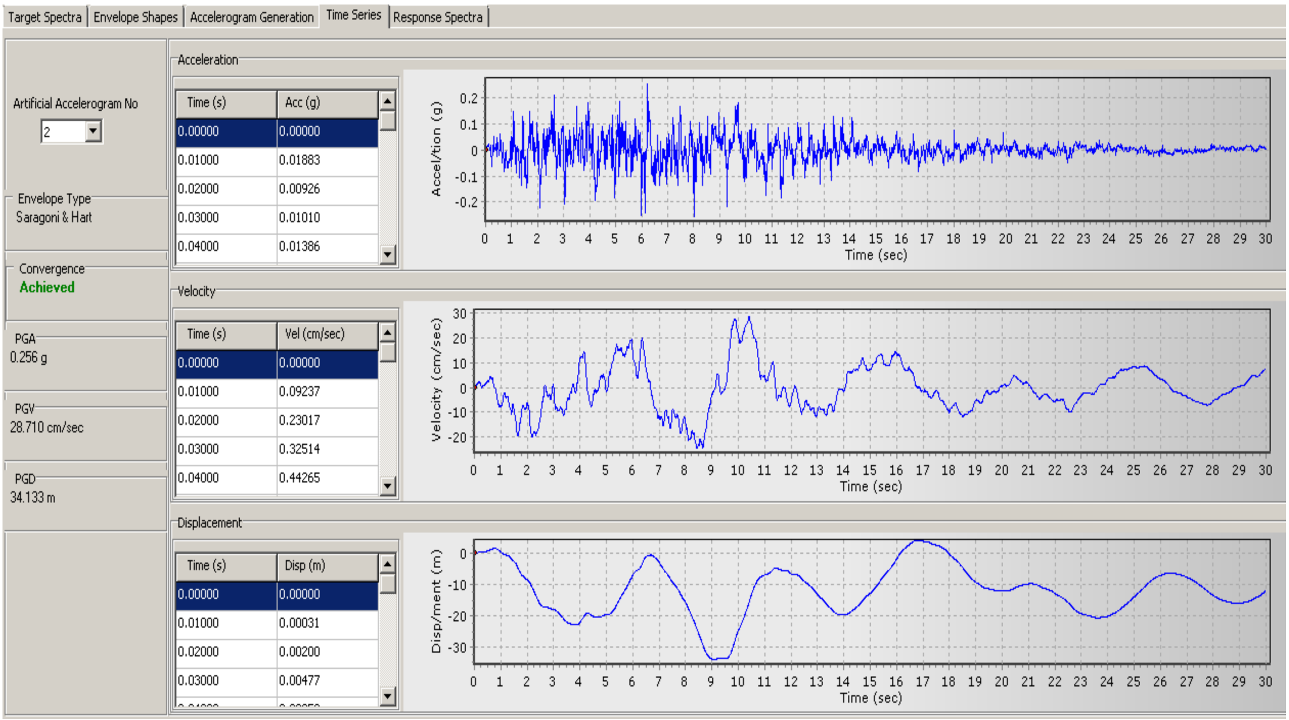

Fig. 4.7 Time History record -2 or artificial accelerograms (2)

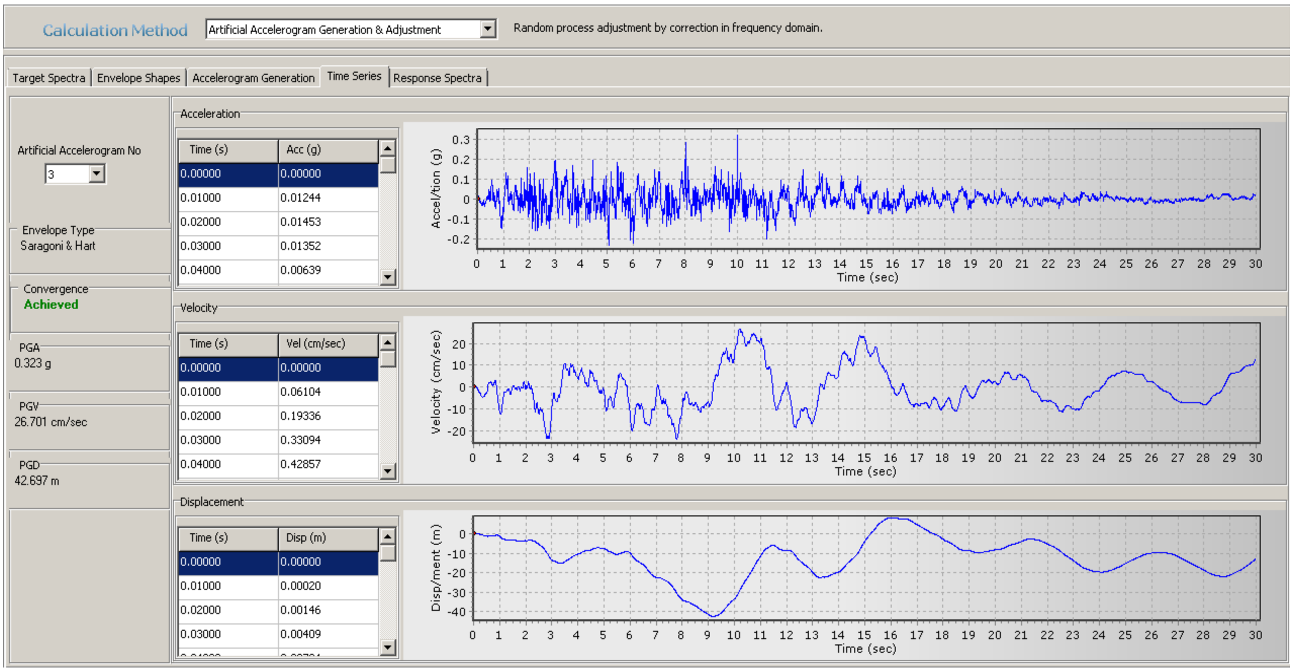

Fig. 4.8 Time History record -3 or artificial accelerograms (3)

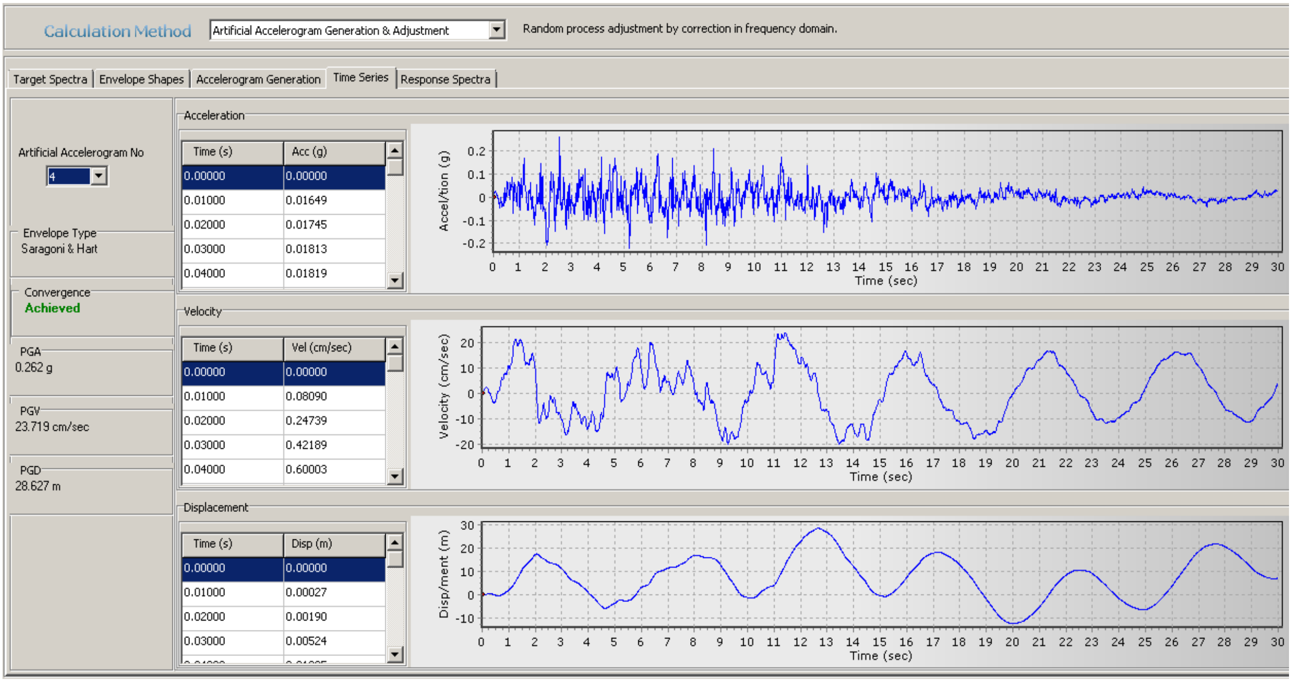

Fig. 4.9 Time History record -4 or artificial accelerograms (4)

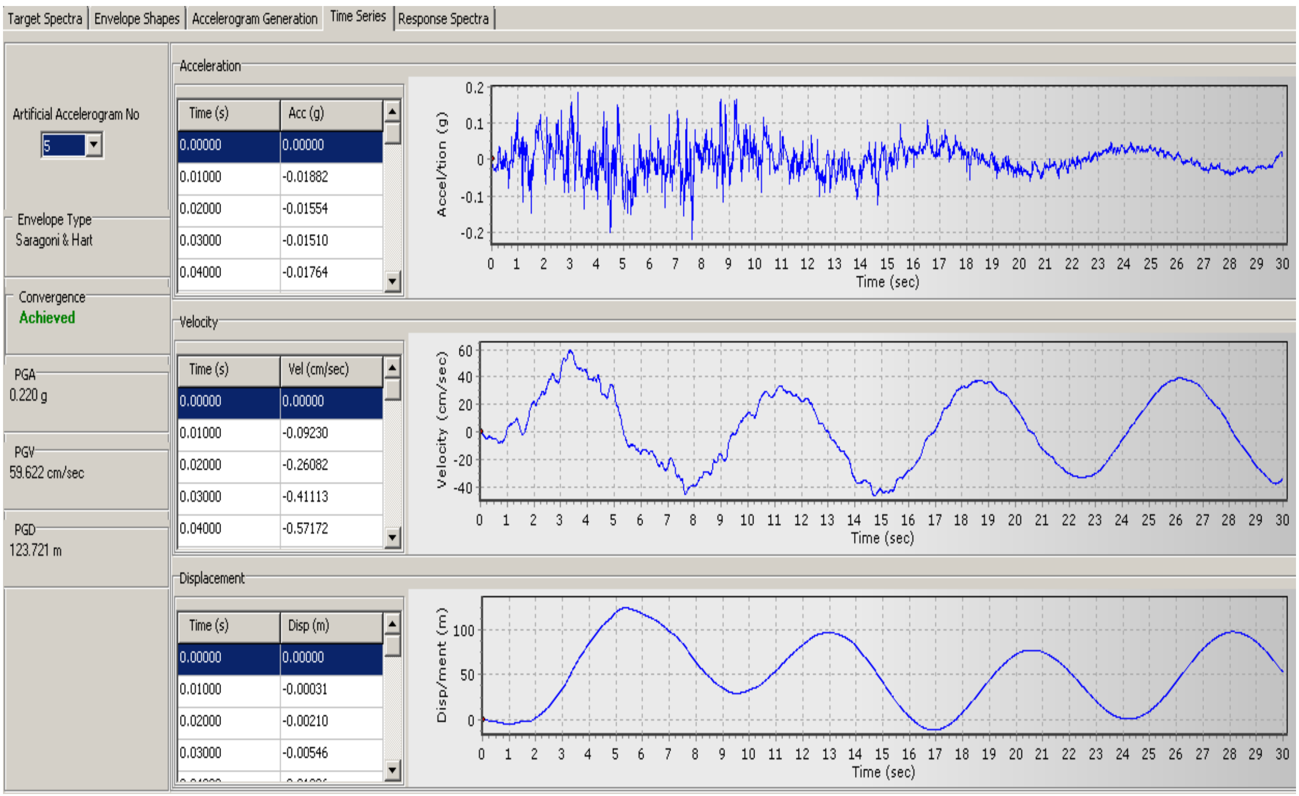

Fig. 4.10 Time History record -5 or artificial accelerograms (5)

4.4 INELASTIC TIME HISTORY RESULTS FOR CASE STUDY BUILDING

In the following section, the time history analyses results for the case study secretariat clinic building in two major orthogonal directions is discussed in terms of global structural damage criteria mentioned in the Section 3.3.4.

The force deformation response curve under the time-history input in terms of base shear vs. roof displacement, which is a measure of buildings performance with respect to hysteretic energy dissipation, is presented.

The vertical distribution of storey displacement & inter-storey drift are also presented for two internal frames (one along grid line X_3 & another along grid line Y_9 of Fig. 3.14) selected in two directions. The displacement and drift profiles have been captured at the global structural collapse step with a view to observe the side-sway mechanism during collapse steps.

4.4.1 RESPONSE IN X-DIRECTION

The building in the longitudinal direction was analyzed for the input time history motion as presented in Section 4.2 by **SeismoStruct-v-5.2.2, 2011** and the analysis terminated at 8.1 second of the input motion, when the building was unable to resist further load reversal of the input motion. Fig. 4.11 shows the inelastic response (base shear vs. roof displacement) of the case study building in X-direction. It is apparent from the Fig. 4.11 that there is a sudden drop of 38% of base shear (from 253 kip in time step 6.1 sec to 156 kip in time step 6.3 sec). So based on global collapse criteria, response corresponding to time step of 6.3 sec marks the beginning of collapse of the structure. Another important thing to note is that, the structure shows stable hysteresis loop prior to the beginning of the collapse. The loop has large area which indicates it has higher energy dissipation capacity.

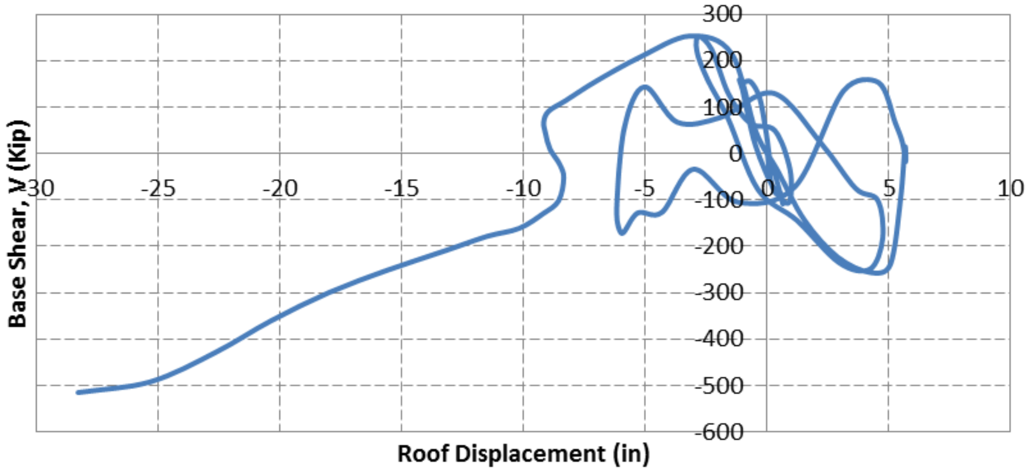

Fig. 4.11 Inelastic Response of building in X-direction

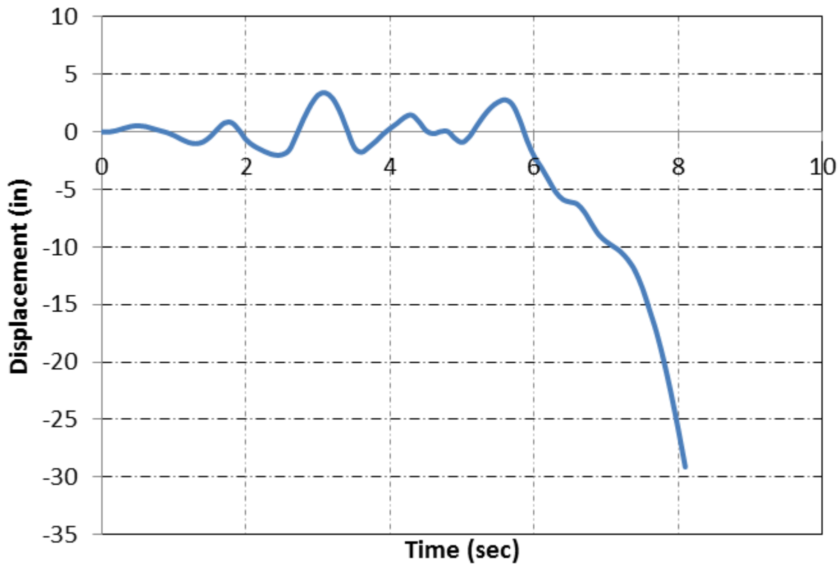

Fig. 4.12 Roof Displacement with input time history record in X-direction

Fig. 4.12 represents the monitored roof displacement with the input motion in X-direction. Consistent with the observation of Fig. 4.11, it is observed that roof displacement shoots up from 3 in at time step 6.1 sec to 5.2 in at time step 6.3 sec, which amounts to about 73% increase in displacement. This sudden increase in deformation as well as strength degradation is due to the substantial loss of stiffness of different building elements (beam, column, walls) due to load reversal of input earthquake motion.

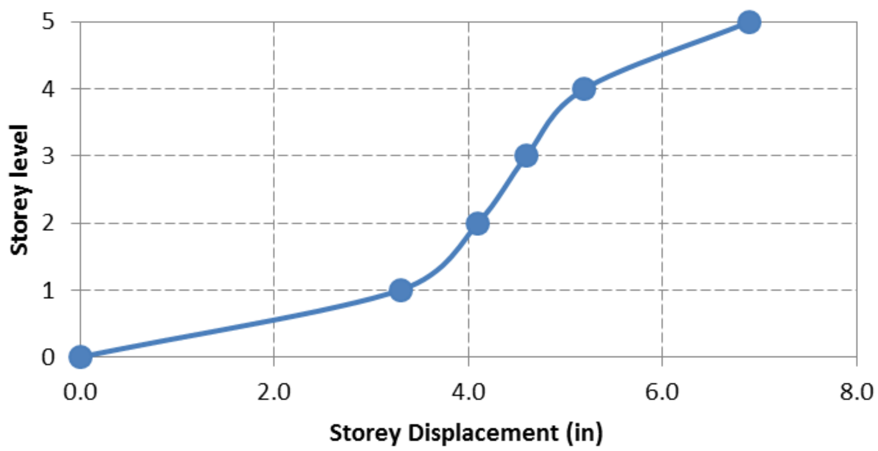

Fig. 4.13 Storey displacement of frame along the grid line X3 in X direction under input motion

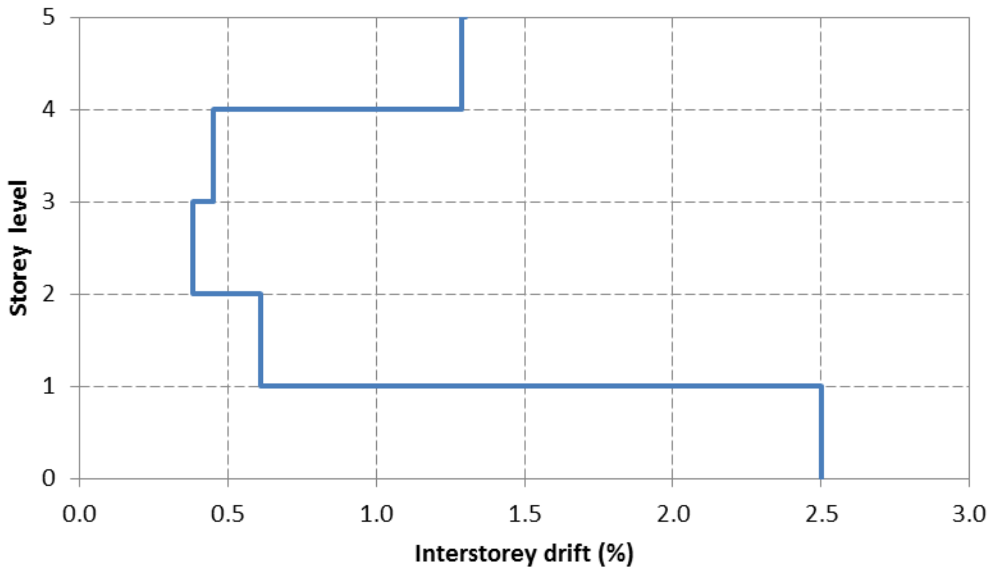

Fig. 4.14 Inter-storey drift of frame along the grid line X3 in X direction under input motion

Fig. 4.13 and Fig. 4.14 show vertical distribution of displacement of the selected internal frame in terms of storey displacement and inter-storey drift. It is observed that maximum storey drift (2.5%) occurs at the ground storey which indicates that the structure would collapse at the ground storey as per the performance level outlined in Table 3.4. The top floor will also be on the verge of collapse as the drift at top storey is around 1.3%. It happens as per column sway method in ground floor, because of there is less or ultimately no partition wall for the purpose of business, so the load bearing capacity in ground floor will be less compared to other floor. In top floor, as we consider fewer amounts of steel bar to reduce the gravity load or due to make economy but in resulting the stiffness of the top most floors will be reduced .Because of these two conditions, the damage occurs top and ground floor compared to other floors as shown in Fig. 4.14

4.4.2 RESPONSE IN Y-DIRECTION

The building in the longitudinal direction was analyzed for the input time history motion as presented in Section 4.2 by seismostruct-v5.2.2 and the analysis terminated at 15.8 second of the input motion, when the building was unable to resist further load reversal of the input motion. Fig. 4.15 shows the inelastic response (base shear vs. roof displacement) of the case study building in Y-direction. It is apparent from the Fig. 4.15 that there is a sudden drop of 34% of base shear (from 38 kip in time step 13.9 sec to 25 kip in time step 14.2 sec). So based on global collapse criteria, response corresponding to time step of 14.2 sec marks the beginning of collapse of the structure. Another important thing to note is that, the structure shows very unstable hysteris loop prior to the X direction collapse, which indicates less energy dissipation capacity.

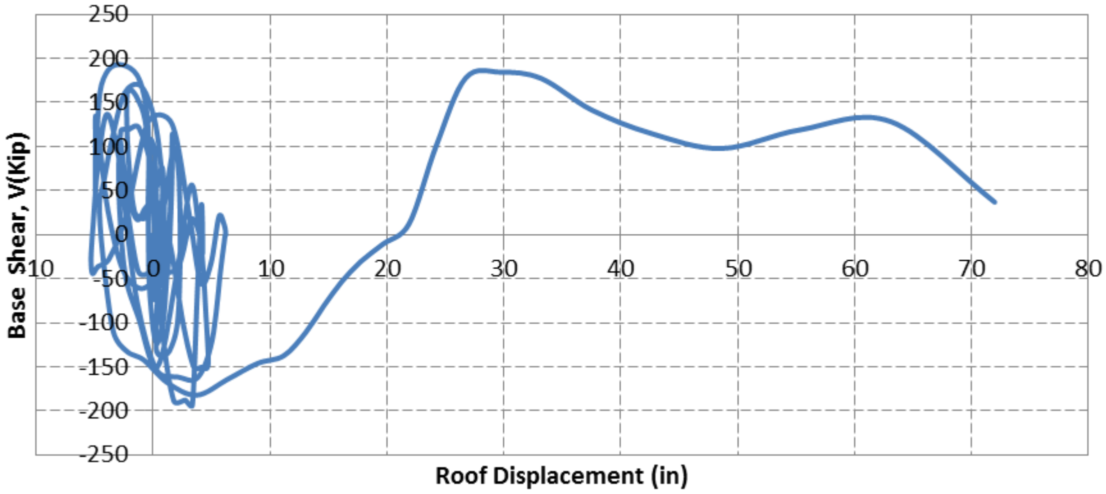

Fig. 4.15- Inelastic Response of building in Y-direction

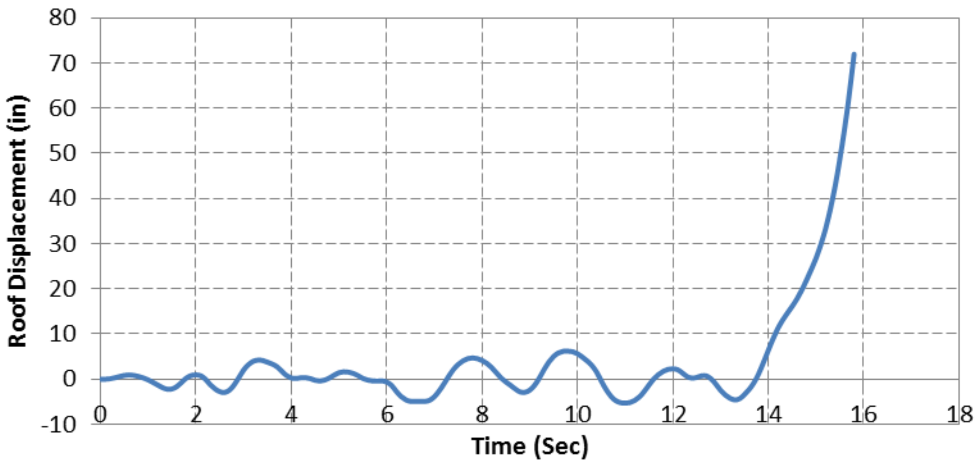

Fig. 4.16- Roof Displacement with input time history record in Y-direction

Fig. 4.16 represents the monitored roof displacement with the input motion in Y-direction. Consistent with the observation of Fig. 4.15, it is observed that roof displacement shoots up from 3.5 inch at time step 13.9 sec to 11 inch at time step 14.2 sec, which amounts to about 73% increase in displacement. This sudden increase in deformation as well as strength degradation is due to the substantial loss of stiffness of different building elements (beam, column, walls) due to load reversal of input earthquake motion.

Fig. 4.17 and Fig. 4.18 show vertical distribution of displacement of the selected internal frame in terms of storey displacement and inter-storey drift. It is observed that maximum storey drift (5.8%) occurs at the top storey which indicates that the structure would collapse at the top storey as per the performance level outlined in Table 3.4.

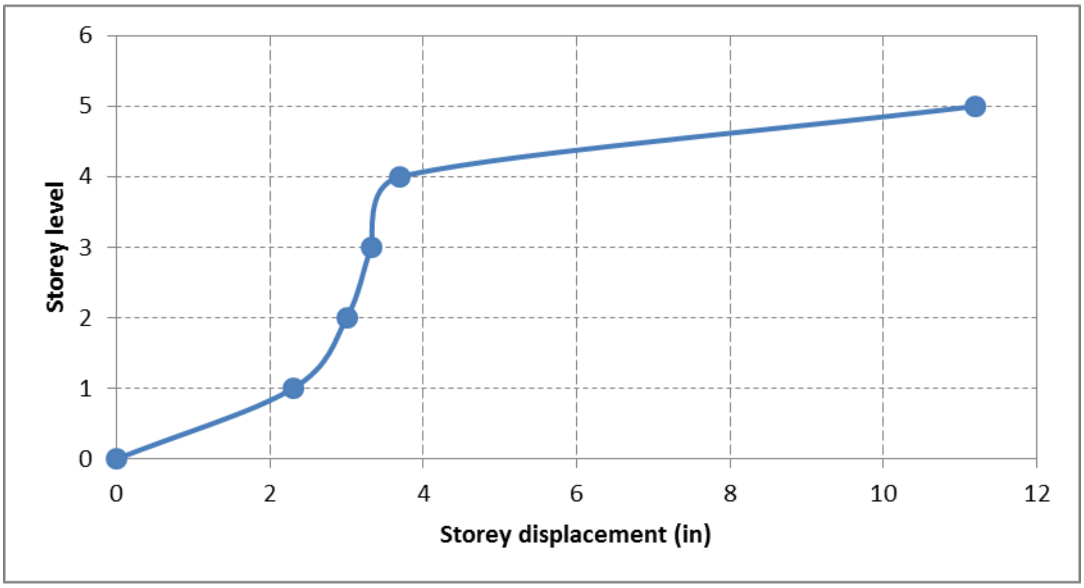

Fig. 4.17- Storey displacement of frame along the grid line Y_9 in Y direction under input motion

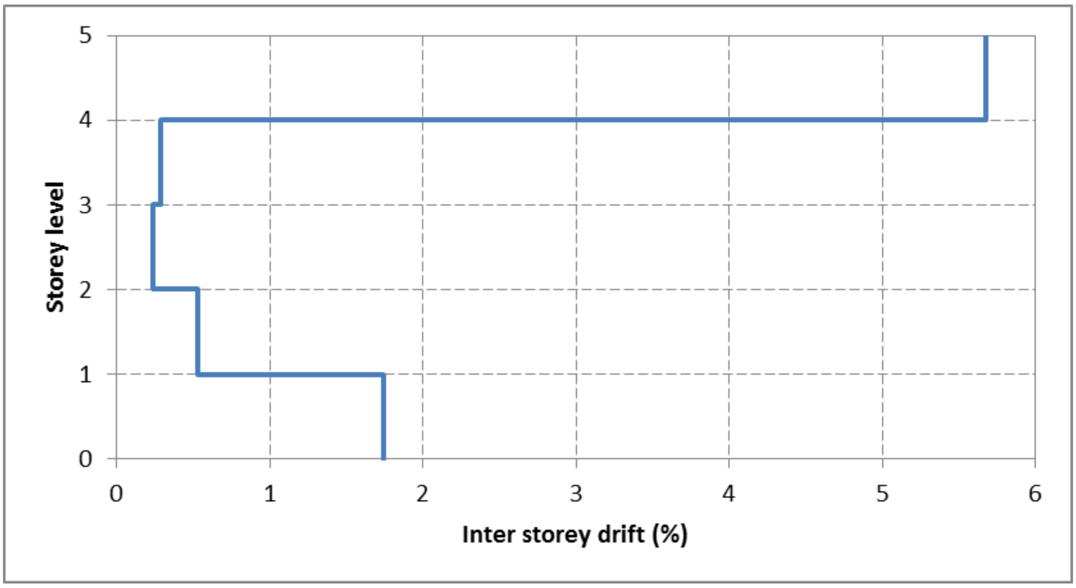

Fig. 4.18- Inter-storey drift of frame along the grid line Y9 in Y direction under input motion

5 CONCLUSION

5.1 SUMMARY

A full time history will give the response of a structure over time during and after the application of lateral dynamic loading. Time history analyses are required to define real seismic response of structure especially for irregular, highly ductile, critical or higher modes induced structures. With advances in seismic analysis and design of structures, nonlinear time-history analyses are becoming more common in civil engineering area.

It is very important to understand how buildings move before, during, and after an earthquake. Time History graphs allow engineers to study the structure's behavior over a specified amount of time. The main difference between Equivalent Lateral Force Procedure (ELFP) and Time History Analysis (THA) is the type of load used to simulate an earthquake. In ELFP, the base shear is the main load, and the analysis is static. In THA, simulations are done by incorporating real earthquakes recorded in the past. The first step in performing a Time History Analysis is to decide what accelerogram to use. But unfortunately there is no strong motion record available in the literature which can be applied for Dhaka city buildings performance evaluation. So, artificial time history record has been generated from the normalized response spectra available in BNBC, 1993 for analysis purpose.

The inelastic time history analysis reveals that the hysteresis curve has large area in X direction compared to Y direction, so the energy dissipation capacity of the building in X direction is higher than the energy dissipation capacity in Y direction. In X & also in Y direction, the story drift at top & bottom floor was found to be greater than 1% , which lead to the development of soft storey mechanism at those floors. The main reason for development of soft storey in top floor is due to reduction in steel area from gravity load consideration. On the other hand, the soft storey in ground floor is due to the reduced stiffness caused by open ground storey for parking purposes.

5.2 FUTURE STUDY

It is possible to observe the performance of individual elements (beam, column) in time history analysis which leads to the identification of the weakest element of the structure. But due to lack of time, it was not possible to monitor the performance of individual elements & hence there remains greater scope to monitor the individual elements performance in future under the input artificial time history record.

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