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INVESTIGATION OF SEISMIC BEHAVIOUR OF STEEL PALLET RACK FRAMES

by

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APPROVAL PAGE

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ABSTRACT

One of the most significant uses of cold-formed members is for steel storage racking structures, such as pallet, drive in, and drive through racking systems. In the current competitive industry, pallets and storage racks may support heavy loads that have the potential to injure workers and damage equipment if the pallets and racks fail and loads fall. Hence, storage racks must remain structurally sound. Additionally, when subjected to earthquake loading, they can exhibit very large transverse displacement. In spite of their complexity, racks are able to carry heavy loads, though they are designed as lightly as possible, and industries often rely on 3-dimensional Finite Element Analysis to achieve this objective. This study, presents a Finite Element model of a conventional rack structure modeled using the commercial software SAP2000. It deals with seismic behavior of cold-formed steel racking frames with different connection types. The objective of the study was to investigate the response of a cold-formed pallet framed racking subjected to earthquake ground motions through nonlinear time history analysis by employing three different earthquake records. In order to investigate the seismic behaviour of rack frames under real earthquake ground motions, the Time History Analysis was performed with rigid, semi-rigid and pinned connections. The results revealed that neglecting semi-rigidity cause stiffening of frames resulting in shorter fundamental period and larger lateral displacement which in turn results in a significant error in the evaluation of dynamic loads. It is also shown from the numerical investigations that semi-rigid frames exhibit ductile and stable behaviour and may be used effectively in earthquake-resistant design. With semi-rigid connection, the base shear could be considerably reduced to the level smaller than those of the frame with rigid and pinned connections. Hence, it is suggested that Semi-rigid connections should always be considered in structural analysis of pallet racks to obtain the most optimum results.

Keywords: Dynamic time history analysis, pallet racks, cold formed steel, earthquake, SAP2000.

PALETLI ÇELIK RAF ÇERÇEVELERININ DEPREM DAVRANIŞININ İNCELENMESI

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ÖZ

Soğukta şekillendirilmiş çelik elemanların en çok kullanıldığı yapılardan bazıları paletli ve içeri forkliftin girebildiği tipleri bulunan çelik depo raf sistemleridir. Mevcut rekabetçi endüstri koşullarında, depo raf sistemlerinin taşıdığı ağır yüklerin devrilmesi sonucunda ekipmanın zarar görmesi ve işçilerin yaralanması söz konusu olabilir. Bundan dolayı, depo raf sistemleri yapısal olarak etki bırakmalıdır. Ek olarak, deprem yükleri altında, geniş oranda enine yer değiştirme gösterebilirler. Kompleks olmalarının yanı sıra, ağır yükler taşıyabilirler, olabildiğince hafif tasarlanırlar, ve bu hedefleri gerçekleştirmek adına endüstriler 3 boyutlu sonlu eleman analizi kullanırlar. Bu çalışma, genel bir depo raf yapısının SAP2000 programı kullanılarak modellenen sonlu eleman modelini içermektedir. Farklı bağlantı tipleri içeren ince cidarlı çelik depo raf sistemi çerçevelerinin deprem etkileri altındaki davranışı ile ilgilenmektedir. Çalışmanın amacı, üç farklı deprem kaydı kullanılarak nonlineer zaman tanım alanında çözüm yöntemi ile bir depo raf sisteminin sismik davranışının incelenmesidir. Raf sistemi çerçevelerinin sismik davranışının gerçek depremler ile incelenmesi için zaman tanım alanında çözümde rijit, yarı-rijit ve moment aktarmayan bağlantılar kullanılmıştır. Sonuçlar, yarı rijitliğin ihmal edilmesinin, çerçevelerin dayanımının artmasına ve buna bağlı olarak daha kısa temel peryodun oluşmasına ve daha geniş yatay yer değiştirmenin ortaya çıkmasına sebep olduğunu ve bunların dinamik yüklerin belirlenmesinde ciddi bir hataya sonuç olduğunu göstermektedir. Nümerik incelemeler sonucunda yarı-rijit çerçevelerin düktil ve stabil davranış sergilediği ve deprem dayanımlı tasarımlarda kullanılabileceği görülmektedir. Yarı-rijit bağlantılar, taban kesme kuvveti rijit ve moment aktarmayan bağlantılara göre ciddi oranda düşürülebilmiştir. Sonuç olarak, en optimum sonuçların elde edilmesi adına depo raf sistemlerinin yapısal analizinde yarı-rijit bağlantıların kullanılmının dikkate alınması önerilmektedir

Anahtar kelimeler: dinamik zaman tanım alanında çözüm, paletli raf sistemleri, soğukta şekillendirilmiş çelik, deprem, sap2000.

This work is dedicated to my late sister

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LIST OF SYMBOLS AND ABBREVIATION

SYMBOL/ABBREVIATION

2D	2 Dimensional
3D	3 Dimensional
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
E	Modulus of Elasticity
FEM	European Federation of Materials Handling Associations
FEMA	Federal Emergency Management Agency
Fu	Ultimate Strength
FX	Base Shear along X-axis
FY	Base Shear along Y-axis
Fy	Yield Strength
IBC	Uniform Building Code
K_m	Stiffness
M_k	Characteristic Failure Moment
M_{Rd}	Design Moment for the Connection
M_{ti}	Observed Moment
n	number of test results in the group
NEHRP	National Earthquake Hazards Reduction Program
NFPA	National Fire Protection Association
PEER	Pacific Earthquake Engineering Research Center
SEISRACK	Storage Racks in Seismic Areas
T	Period of Vibration

UX	Displacement along X-axis
UY	Displacement along Y-axis
v	Poison Ratio
γ_m	Material Factor
Δ_a	Interstory Drift
$\Theta_k i$	Rotation

CHAPTER 1

INTRODUCTION

1.1 GENERAL

One of the most significant uses of cold-formed members is for steel storage racking structures, such as pallet, drive in, and drive through racking systems, (Bajoria et al., 2010). Storage racks are usually found in industry used for storing goods, mostly on pallets and made from cold-formed steel profiles. The most common type of rack, are separated by aisles and each pallets. In the current competitive industry, pallets and storage racks may support heavy loads that have the potential to injure workers and damage equipment if the pallets and racks fail and loads fall. Storage racks in particular must remain structurally sound. Additionally, when subjected to earthquake loading, they can demonstrate very large lateral displacements and are therefore susceptible to significant consequences of second-order geometrical effects.

In particular, steel storage racking systems are non-building structures that are well known by the fact that they carry live loads much larger than self- weight and rise to considerable height. Storage racking systems are as well differentiated by the great variety of typologies, shapes and sizes, ranging from large warehouses to small shelves for offices or shops. This imposes different importance from a design point of view, and a somewhat incompatible situation regarding which design codes to use and when to use them. Their behavior is also influenced by the geometry of their structural components (high slenderness elements, open section profiles hence prone to buckling problems) as well as by the non-linear behavior of their joints (beam-to-upright and base-plate). Thus, many difficulties arise in the prediction of their structural behavior or modeling problems of beam-to-upright and base-plate connections, (Stella, A., 2012).

Nowadays, in a conventional analysis of steel structures, beam-to-column connections are generally considered to be either hinged or completely fixed. These assumptions are not accurate. Still, they have been used in practice due their simplicity for use in analysis and design

However, a more economical design would result if the effects of semi-rigid connections were considered in the analysis of rack frames, these effects have been ignored due to the lack of information regarding the behavior of such connection coupled with complexity of the analytical method. In spite of their complexity, racks are able to carry heavy loads, though they are designed as lightly as possible, and industries often rely on 3-D Finite Element Analysis to achieve this objective. This study, presents a Finite Element model of a conventional rack structure modeled using the commercial software SAP2000 (version 12). The model is checked against rigid and pin-connections to obtain accurately semi-rigid behavior of the storage rack.

1.2 BACKGROUND AND MOTIVATION

The demand for a multiple, easier and efficient storage of goods is growing as the logistics are continuously developed (Adamakos, K et al., 2013). Unfortunately they do not follow the international norms applying for buildings because they are not conventional steel structures. For this reason nowadays numerous attempts are made for the publication of an independent and complete normative document. Presently, lack of sufficient design rules and bibliography makes the seismic design of pallet racks quite complex.

Even though these structures, made by thin-walled and many times cold-formed steel profiles, are very light and represent only a small percentage of the annual sales of steel profiles in the world, very large economic interests, as well as civil and penal Right liability problems might arise as a consequence of an earthquake event striking them, (SEISRACKS, 2007). For instance in 2003 estimated pan–European sale value for the racking industry exceeded 1.2 Billion Euro. Racking systems operated by industrial trucks represent approximately 70% of the total yearly racking industry market. The current estimated yearly loss due to accidental impact is 600 million Euros.

Moreover the losses due to consequent fires far exceed this value. Economical losses are expected to continue to rise due to competitive pressure in the logistic industry, resulting in higher driving speeds of industrial trucks within the racking environment.

On the other hand, modeling cold-formed steel, particularly through collapse, presents a strongly nonlinear problem with both material and geometric nonlinearity. Therefore, meaningful modeling requires more than a good nonlinear solution scheme and a robust element. Successful modeling requires in-depth understanding of the model inputs and their sensitivities, as well the limitations and strengths of the modeling tools themselves.

The research motivation for this study originates from the need to get a deeper understanding of the influence of seismic action on the structural behavior of thin-walled frames. As the special geometry of these thin-walled structures of high slenderness and their non-linear behavior require specific regulations for a successful and accurate modeling. This thesis aims at developing an accurate and efficient material geometric non-linear time history analysis for pallet rack structural behavior by the use of the SAP2000.

1.3 AIM AND OBJECTIVE

The main aim of the study is to investigate the range of response characteristics of pallet rack frames, having different connection types subjected to seismic loading. More precisely, the study deals with down-aisle frames and follows three definite objectives:

1. To obtain dynamic characteristics which are fundamental periods, mode shapes response to various strong ground motions of conventional pallet racking systems, made up of cold form sections.
2. To determine the maximum base shear at the time of collapse and maximum displacement for different connection types.
3. To investigate the effect of beam-column connections on structural behavior of rack system for various connection types under horizontal and vertical loads.

1.4 SCOPE OF THE STUDY

The study was limited to a 4-storey, 5-bay steel frame. The beams were all 140×50mm and the columns were 120×80×2.5 mm. All connections regardless of their location are assumed to behave in a similar manner which may not be the case in test situation.

1.5 METHODOLOGY

The most accurate method of seismic demand prediction and performance evaluation of structures is nonlinear time history analysis. However, this technique requires the selection and employment of an appropriate set of ground motions and having a computational tool able to handle the analysis of the data and to produce ready-to–use results within the time constrains of design offices (Bajoria, 2008).

In this research, modeling of conventional pallet racking systems was carried out using the finite element program. Nonlinear time history analysis found to be a useful analysis tool for the conventional pallet racking systems giving good estimates of the maximum roof displacement, base shears and time history graphs.

1.6 TYPICAL RACK CONFIGURATION OF PALLET RACK

Steel pallet racks are particular structures formed with specially designed cold-formed steel elements which allow easy installation and reconfiguration. The longitudinal direction is called down-aisle whereas the transversal is called cross-aisle direction.

In a typical storage rack structure, there two or three pallet loads between upright. The storage rack bays are typically 1.0 to 1.1 deep and 1.8 to 2.7 meters wide. The total height of the racking system is based on the limitations imposed by handling equipment and the building height. For instance, the overall height of a typical pallet racking system found in retail warehouses varies between 5 and 6 meters, while in industrial warehouses it can reach up to 12 to 15 meters.

The components of the upright frames consist of the steel uprights and cross braces which can be bolted or welded together. Bolted construction is more prevalent in Europe, while welded frames are more common in the United States. There are also two ways to connect beams and frames; the components can be bolted to frames or interlock using a slotted connection system.

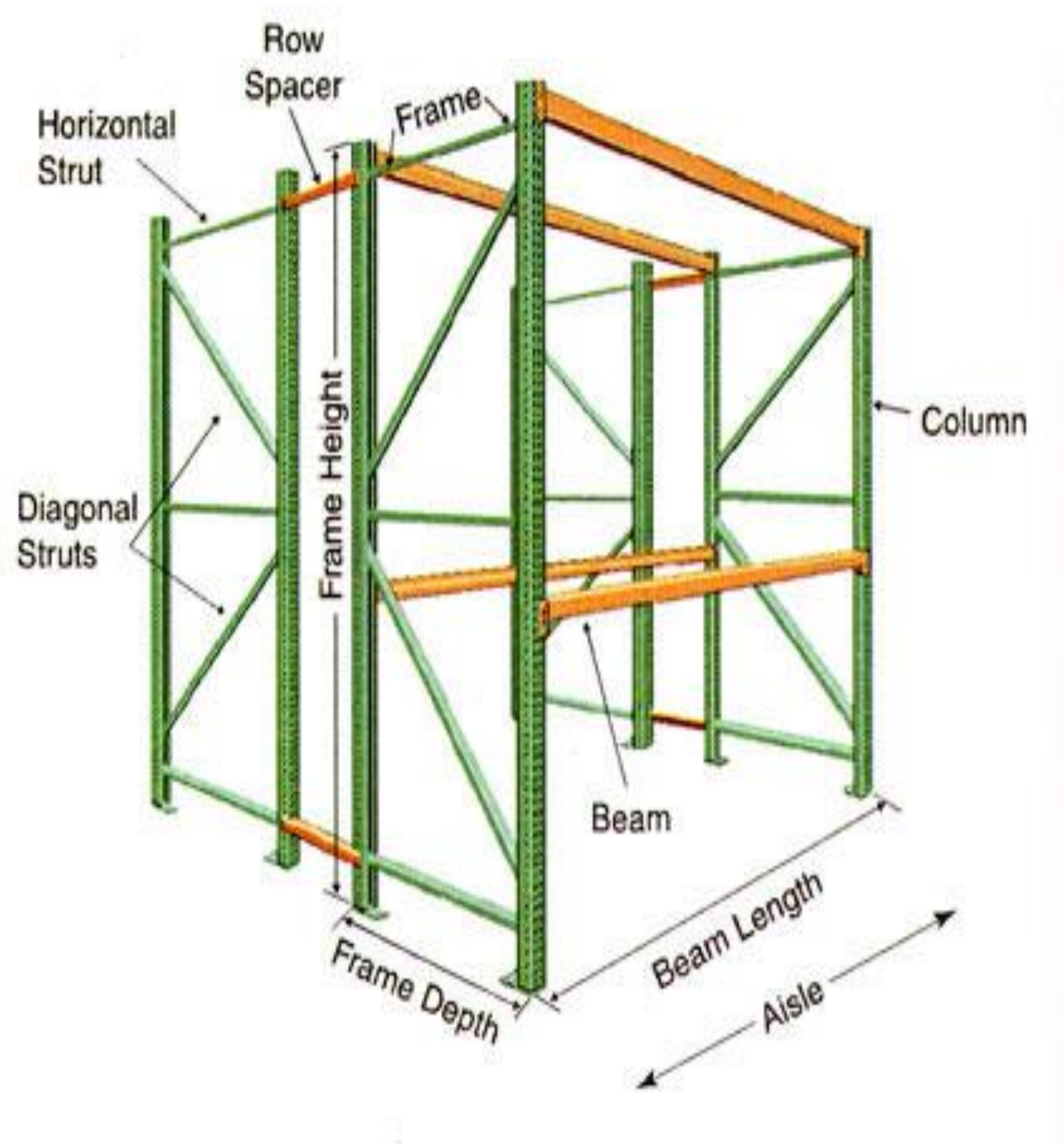


Figure 1.1 Typical Pallet Racking System.

1.6.1 Structural Elements of Racking System

1.6.1.1 Pallet

Typical storage pallets have plan areas of about one square meter and can weigh up to approximately 8 to 15kN. The Euro pallet for example has dimension (L×W×H) 8000x120x144 and it is four way pallet made of wood.

1.6.1.2 Bay

Refers to the space between two uprights frame spanning as many load level as permitted.

1.6.1.3 Upright Frames

The vertical components of the racks are called upright frames and they consist of two columns (front and rear) thin gauge diagonal trusses capable of working in tension or both intension and compression. The profile of the columns is defined by the need for high strength and easy connections in the two perpendicular vertical levels. The columns are perforated in order to facilitate the links, usually with oblique slots for the beams and circular holes for the diagonal stress. The strength of the column is affected by phenomena such as local buckling and global flexural bending.

1.6.1.4 Beams

They have closed built up sections made of cold-formed elements. Generally they are composed by two U-shaped members, hence forming box cross section with increased torsional stiffness.

1.6.1.5 Joints

The behavior of the beam-to-column connection is essential to the stability of the whole structure since it provides the frame action longitudinally. The connection is regarded as semi-rigid and can be described with a moment rotation diagram defined by experiments. The behavior of this links could also be approached by rotational spring with stiffness derived by moment-rotation diagram.

1.6.1.6 Spine bracing (vertical Bracing)

These vertical bracings are usually of the rear plane of the rack, thus rendering structural regularity in plan. They shift rack against horizontal loads in particular seismic forces. Transverse members (mainly compressive) and diagonal members working in tension only.

1.6.1.7 Plan bracing

Horizontal diagonal bracings on the level of the shelves are used occasionally, placed between beams, in order to transfer the horizontal actions from the unbraced vertical plane of upright to the braced vertical plane.

1.7 OVERVIEW OF THE THESIS

The essence of this thesis is to design and study the seismic characteristics of a cold-formed pallet rack for the various connection types when subjected to different ground motions. Intuitively, there no doubt that the choice varies in response to seismic events.

Chapter 1 focuses on the importance of steel storage racks as it's increasingly spread in warehouses by highlighting the importance of complying with design requirements for strength, economy, and safety of the user as well as of the stored goods. Description of a typical rack structure in a rather narrow perspective is also included in this section. Stressing on the peculiarity of the rack system and emphasizing on the needs for special attention on design and analysis of such structures.

The review of relevant design codes and standards are presented in Chapter 2. In addition, it does review the previous studies done on the cold-formed storage and research findings by different studies.

Chapter 3 outlines the research methodology adopted. This involves details of the simulation procedure in SAP2000 concerning the configuration of the models as well as material nonlinearity. It ends with the simulation of beam-to-column connection for rigid, semi-rigid, and pinned connection cases.

The results presented in Chapter 4 reveal the summary of the result obtained from the analysis, they were presented in the form of table and graphs to enable comparison among the various models. The second part encompasses the analysis and discussion of the result obtained in relation to the previous study.

In the final chapter, the conclusions of this study are assembled and some suggestions for future work are provided. Within the limitation of this study**,** Chapter 5 summarizes the findings and conclusions of the study. Some recommendations are also given for future studies regarding the cold-formed rack design.

CHAPTER 2

LITERATURE REVIEW

2.1 INRODUCTION

This chapter presents a brief overview of the design and current issues surrounding numerical modeling and experimental tests in steel racking system, which is strongly based on the researches within Europe and USA. The literature survey not only reviews but also summarizes recent research including the numerical modeling of cold-formed steel rack obtained within the scope of the available literature.

2.2 DEVELOPMENT OF CODES AND STANDARDS OF RACK SYSTEMS

2.2.1 RMI Standards

The Rack Manufacturers Institute (RMI) was established and incorporated in 1958 to deal with industry-wide issues. The first edition of an RMI standard, Minimum Engineering Standards for Industrial Steel Storage Racks was published in 1964. It was a short, simple, direct exposition of what had been developed and what was known by the members of the industry at that time. It represented the first step in developing seismic behavior of steel storage pallet racking systems specifications and other products designed to suit the needs of users, manufacturers, and the engineering and code-enforcement communities.

In the late 1960s, RMI engaged Professor, George Winter of Cornell University carry out analysis and testing to provide a sound foundation for the development of a more precise standard for the industry. The results of the work conducted by Professor Winter and his team provided the basis for a new RMI standard, Interim Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks, which was issued in 1972 and which required earthquake loads to be considered in a manner resembling the approach to building structures as stated in the Uniform Building Code (UBC). Similar to UBC, the K factors for ordinary moment frame building structures and braced framed structures were 1.0 and 1.33, respectively. These were the factors used to define the seismic forces in 1972 edition of the RMI standard in the down-aisle and cross-aisle directions, respectively.

2.2.2 Model Building Code requirements

Storage rack structures were mentioned for the first time in the 1973 edition of the UBC in the form of a footnote to a list of structures. The 1976 UBC referenced Standard 27-11 that handled storage racks specifically and included design seismic forces for storage racks. Shortly, the National Building Code and Standard Building Code added seismic provisions for storage rack structures to their codes.

2.2.3 NEHRP Recommended Provisions

The first edition of the NEHRP Recommended Provisions for Seismic Regulations for New Buildings was published in 1985 to serve as a resource document for the organizations involved in developing seismic requirements as well as providing an opportunity for ongoing improvement of these requirements. In 1987 after Provisions Update Committee meeting, since that time RMI representatives have served on the PUC (Provisions Update Committee) Technical Subcommittees responsible for requirements for steel structures; for architectural, mechanical, and electrical components and systems; and for non-building structure. The 1991 edition of the NEHRP Recommended Provisions introduced design values for storage racks in the chapter on architectural, mechanical, and electrical components. In the 1994 Provisions, design values for storage racks were extensively revised to be more consistent with the RMI seismic design criteria. R values of 6 and 4 were assigned for storage racks in the down-aisle direction and the cross-aisle direction, respectively. Furthermore, an importance factor of 1.5 was assigned for racks in areas accessible to the public. The R factor values of 6 and 4 were basically a translation of the UBC K values from the early 1970s. Starting in the 1997 NEHRP Recommended Provisions, non-building structures including storage racks as well as cooling and storage towers, which had been treated in the chapter on architectural, mechanical, and electrical components, were now covered in a separate non-building structures chapter.

2.2.4 ASCE 7 Requirements

As the NEHRP Recommended Provisions became more dominant with the various model code organizations, the American Society of Civil Engineers (ASCE) developed the first of a series of updates to the ASCE 7 standard, Minimum Design Loads for Buildings and Other Structures. The 1993 edition of ASCE 7 adopted the 1991 NEHRP Recommended Provisions as its seismic provisions and the 1995. Edition of ASCE 7 reproduced the 1994 edition of the NEHRP Recommended Provisions and covered storage rack structures under the category of architectural, mechanical, and electrical components and systems.

2.2.5 FEM 10.2.08

Presently in Europe there is no officially accepted design code for racks in seismic areas, but only the 2005 version of FEM10.2.08. Thus designers are compelled to operate with complete lack of reference to the Rack Manufacturers Institute (R.M.I) Specifications, while the European Racking Federation (F.E.M-ERF) is currently working in order to produce an official document.

Additionally, Eurocodes 1, 3, and 8 give insufficient information on many design issues. Therefore, in 2005 the FEM Seismic Design Standard FEM 10.2.08 titled The Seismic Steel Pallet Racks. It deals with all relevant and specific seismic design issues for racking system. The design procedure given in FEM 10.2.08 apply to all types of static pallet racks fabricated from steel members and supported by floors lying on the ground. It is based upon the philosophy of EN 1988 (Eurocode 8) while the design test and quality control of components materials refer to FEM 10.2.02.

In FEM 10.2.08, the seismic response is modified by means of two coefficients that estimate the effects of phenomena of the racking systems such as energy dissipation (due to the friction between the pallets and the beams), pallet damping (due to the movements of the stored product), in addition to pallet flexibility .

The pallet-beam friction coefficient to be considered is the static one and it depends on the materials in contact and the environment (wet/dry/warehouse conditions). It ranges between 0.05 and 0.25.

2.2.6 FEM CODES OF PRACTICE

FEM is the European Federation of Materials Handling Associations was formed in 1953. The first attempt to create European code of practice takes place in the beginning of the eighties as part of the research European Community sponsorship. However, it was not implemented by the national associations of racking and shelving manufactures at that time and it was reviewed in ninetieth. ER/FEM sponsored the development of FEM Codes of Practice which resulted in 2000 in the publication.

As stated earlier, in Europe, no official document is currently available for the seismic design of pallet racks and the designers are compelled to operate without references to commonly accepted European design rules. Present Eurocodes 1, 3 and 8 give insufficient information on many design issues. However, some of the legal European Directives necessary to comply with are given in Table 2.1.

Table2.1 Review of FEM Code's of Practice:

FEM	TITLE
FEM 10.2.05	Guidelines for working safely with lift truck in pallet racking installation
FEM 10.2.06	The design of loaded static steel shelving systems
FEM 10.2.07	The design of drive-in and drive through racking
FEM 10.2.08	Recommendations for static steel pallet racks under seismic conditions
FEM 10.2.09	The design of cantilever racking
FEM 10.2.10	Rail dependent racking and retrieval systems interfaces
FEM 10.2.11	Rail dependent racking and retrieval systems interfaces-Consideration of kinetic energy action due the faulty operation in a cross aisle direction in compliance with EN 528-Part1: Pallet Racking
FEM10.3.01-1	Basic calculations for storage racking and retrieval machines- tolerances, deformations and clearance in storage system- General, Single deep, Double deep Pallet Racking

2.2.7 Specific Modeling Requirements for the Analysis

I. The modeling and analysis rules must be according to FEM 10.2.02.

II. For beam-to-column connections the stiffness obtained from tests according to FEM 10.2.02 for static load must be used. The pallet beam must be checked under pallet load with pinned ends and load factor gL = 1.0

III. The shear stiffness of the upright frame must be evaluated according to FEM 10.2.02.

IV. When tension diagonals are provided, the bracing elements must be modelled in order to take into account the proper stiffness of the bracing considering the effect of the active component.

2.2.6 SEISRACKS Project

There have been limitations regarding the design of storage racks in seismic areas which are in principal due to lack of knowledge and hence lack of Standard Design Codes in Europe. Only one study was available in Europe, carried out within the ECOLEADER Research Program, for Free Access to Large scale Testing Facilities, known as Seismic Behaviour of Pallet Rack Systems (Castiglioni et al, 2008). To solve these limitations, the EU sponsored through the Research Fund for Coal and Steel, a research project titled Storage Racks in Seismic Areas (SEISRACKS) which initiated in 2004 and terminated in 2007.

In this project, four specimens were tested in full scale on the shaking table of the Laboratory for Earthquake Engineering at the National Technical University of Athens. The following activities were carried out:

1. Characterization of the component behavior
2. Assessment of the sliding conditions of pallets on rack beams and experimental determination of friction properties of pallets
3. Push-over tests on two full-scale racks models
4. Pseudo-dynamic tests on one full-scale rack model
5. Assessment of the actual service loading conditions of racks (in-situ monitoring)

6. Shake table tests on six full-scale rack models/ Experimental study of the cyclic behaviour of beam-to-upright joints and of base anchor-ages
7. Numerical modelling and study of the global dynamic structural behaviour of racks subjected to earthquakes including sliding of pallets.

Design is based upon the philosophy of EN 1998-1 (Eurocode 8), whereas the design, tests and quality control of components and materials refer to FEM 10.2.02. The FEM 10.2.08 Code of practice deals with all relevant and specific seismic design issues for racking systems such as:

1. The seismic response can be significantly different in down-aisle direction and in cross-aisle direction and can also be considerably affected by the size and the distribution of the masses
2. The natural damping of the structure without its pallet loads is very small, however in real conditions the damping can be significantly more than expected due to micro movements of the stored goods and sliding effects
3. Cyclic forces due to earthquake can progressively damage connections and/or other components thus affecting the response of the whole structure
4. In case of seismic isolation, the effectiveness of it must be guaranteed for all the loading conditions and during the whole expected life of the racking system.

For a second time, due to the further gap in knowledge of these structures, another research on Seismic Behaviour of Steel Storage Pallet Racking Systems (SEISRACKS 2) was initiated in 2011. The major objectives including examination of the out-of-plane of the beams and beam-upright connections, the investigation of the behaviour of the cross-aisle direction on the configuration and on the behaviour in the down aisle-direction in presence of eccentric vertical bracing, the use of nonlinear analysis for the behaviour of rack structures under seismic loads based on multi-modal spectral analysis, the study of the actual behaviour of the palletized goods depending on size and shapes.

The project focuses on steel selective pallet storage racks located in areas of retail warehouse stores and other facilities, eventually accessible to the general public (SEISRACKS1, 2007). The whole research project SEISRACKS has been an opportunity to analyze the current draft of the normative document FEM 10-2-08, also known as Recommendations for the design of static steel pallet racks under seismic conditions. In particular, a series of items have been identified as questionable and some are listed here with the corresponding sections of pr FEM 10-2-08 in its version of December 2005:

i. Regularity criteria and consequences on the behaviour factor (2.2 5),
ii. Effect of the actual position of the gravity centre of the masses, vertical eccentricity with respect to the beams (2.3.6),
iii. Methods of analysis (2.4),
iv. Definition of regularity criteria (3.1.4),
v. Modelling assumptions in the perspective of the structural analysis (3.3),
vi. Account for the different sources of energy dissipation (Viscous damping, friction of pallets, energy dissipation within the stored goods)
vii. Definition and values of parameters ED_1, ED_2 and RF (2.3.1, 2.3.2, 2.3.3, 2.3.4, 4.2.2, 4.2.3)
viii. Assessment of the structural ductility and associated behaviour factor
ix. Definition of ductility classes (3.1.1)
x. Material properties and overstrength coefficient (3.1.2)
xi. Definition of the q-factor according to the structural typology (3.1.3, 3.4)
xii. Impact of (ir-) regularity (3.1.4, 3.4)
xiii. Design rules for non dissipative vs. dissipative structures (3.1.5)
xiv. Identification of the resisting system (3.2)
xv. Detailing of dissipative elements and overstrength criteria (5)

2.3 COMPARISON OF RMI 2008 and F.E.M 10.2.08

The comparison with ANSI-RMI-2008 edition and the FEM 460 was carried out considering mainly low ductile design concept of FEM 10.2.08 which is the most relevant.

Table 2.2 Comparison between RMI 2008 and F.E.M 10.2.08.

	F.E.M 10.2.08	**ANSI-RMI-2008**
Behavior factors	The behavior factors q defined by FEM 10.2.08 are mainly related to the structure and don't take into account the interaction with the unit loads	An effect is included in the response of modification factor (R) approach of RMI-2008
Method of analysis	FEM 10.2.08 assumes the modal response spectrum of analysis (MRSA) allowing the LFMA as a simplified procedure under conditions ensuring that modes with lower periods have negligible relevance	The reference method of analysis for RMI-2008 is lateral forced method of analysis (LFMA). RMI-2008 allows also displacement-based method referenced in FEM-460, and MRSA appears to be used in the design practice even if not mentioned in the Code.
Second order effects	Second order effects are considered by FEEM 10.2.08 in all cases in Ɵ exceeds 0.1 either directly or with a simplified method	RMI-2008 require, in the reference design procedure, to consider the 2^{nd} order amplification only for the evaluation of the rotational demand of the connections for unbraced racks in down aisle direction
Seismic action combination	while it is mandatory for FEM 10.2.08.	the effects of the seismic action occurring in the 2 main directions is not required to be combined by RMI-2008
Rotational capacity of the connections	uses a quite simple testing protocol for the beam-end connector, but nothing is specified for the baseplates	The control of the rotational capacity of the connections: it is required by FEM 10.2.08 only with ductile design concept
Ductility	In low ductility design approach FEM 10.2.08 allows using the test protocols of the EN15512, while cyclic tests are required to assess the rotational capacity of beam-end and floor connectors for ductile design, the testing protocol is not well specified and difficult to apply	
Effects of friction	FEM 10.2.08 provides a detailed procedure for the design of pallet beams under seismic actions, taking into account the effects of friction between unit loads and beams	no specifications are provided by RMI-2008 for seismic conditions
Beam-end connector stiffness	For the beam-end connector stiffness, FEM 10.2.08 in low ductility design approach allows using the values obtained from tests according EN15512	Requires using the connection secant stiffness derived from the moment-rotation curve obtained from static test consistent with the applied base shear and resulting displacements (this implies design procedure).

(Castiglioni, C.A, 2013)

2.4 REVIEW OF PAST SEISMIC RESEARCH ON STORAGE RACKS

Available results from experimental and analytical investigations on the seismic response of storage racks are briefly reviewed here, and gaps in knowledge requiring further research studies were identified.

2.4.1 Experimental researches

Experimental research related to the seismic behavior of storage racks can be categorized into different types of testing procedures:

i. Cantilever testing of subassemblies in which quasi-static cyclic loads are applied to beam-to-upright connections.
ii. Portal testing of subassemblies in which beam and uprights portal subassemblies are loaded laterally to simulate seismic loading.
iii. Quasi-static testing of storage rack systems for which completely loaded storage racks are loaded laterally to simulate seismic loading.
iv. In-situ dynamic testing of storage rack systems with small shakers or under ambient vibrations, in order to obtain their dynamic characteristics (e.g., natural periods and damping).
v. Shake-table testing of storage rack systems, with completely loaded storage racks, excited by recorded or artificially generated ground motions.
vi. Testing of cold-formed steel members and structures, from which most storage racks are built.

2.4.1.1 Cantilever Testing of Storage Rack Subassemblies

The lateral stiffness of storage rack systems in the down-aisle direction is significantly affected by the distortions that occur at the beam-to-upright connections. For analytical modeling purposes, these distortions often are represented by simple rotational spring elements inserted between the beam ends and the upright center line. The rotational spring constant to be used in a numerical model can be obtained from moment-rotation relationships between a beam end and an upright using the so-called cantilever test method (RMI 2002).

Krawinkler et al. (1979) carried out cantilever tests on 20 different beam-to-upright subassemblies of standard pallet racks. In all connections, the beam ends were welded to angle connectors that, in turn, permitted connection to the perforations on the upright through either hooks (Type A) or button grips (Type B). The experimental results indicated that, because of local deformations at the beam-to-upright connections, moment-rotation hysteretic loops have a pinched shape similar to that obtained for reinforced concrete elements with high shear load. Low cycle fatigue phenomena also may affect the strength and ductility of beam-to-upright and upright-to-floor (base plate) connections. The strength of the Type A subassemblies was limited by the capacity of the hook-type grips that started to pull out of the upright perforations. In Type B subassemblies, fractures of the fillet weld between the beam and the connection angle limited the moment capacity.

In another research, Bernuzzi and Castiglioni (2001) performed a series of 11 monotonic and 11 cyclic tests on two different types of beam-to-upright connections used in Europe. The experimental results obtained from the monotonic tests indicated that the connections were characterized by significant ductile behaviour. None of the test specimens failed before the maximum rotation achievable by the testing equipment was reached. This maximum rotation was way beyond practical design values. The results of the cyclic tests exhibited, with increasing number of response cycles, pronounced pinching behaviour associated with slippage and plastic deformations of the connectors leading to significant reduction of energy dissipation capacity.

Quasi-static testing was recently conducted on 22 different types of interior beam-to- upright subassemblies (Higgins 2004). The test data indicate that beam-to-upright connections exhibit very ductile and stable behavior, with rotational capacities beyond the values observed during shake-table tests and expected from a design seismic event. The hysteretic responses of some of the tested beam-to-upright connections, however, exhibited significant pinching similar to those tested by Bernuzzi and Castiglioni (2001).

Sarawit and Peköz (2003) recently proposed a new beam-to-upright connection test to replace the cantilever test method. They concluded that the actual frames bending moment-to-shear force ratio is better represented in this proposed test method than the current cantilever test.

2.4.1.2 Portal Testing of Storage Rack Subassemblies

In this testing procedure, a portal assembly of generally one beam connected to two uprights is loaded by constant static gravity loads on the beam and variable lateral loads on one of the upright at the elevation of the beam. Moment-rotation at both beam-to-upright connections can be monitored during the tests.

Krawinkler et al. (1979) have done six portal tests on three different beam-to-upright subassemblies of standard pallet racks. Four types of beam-to-upright connections were investigated. In all connections, the beam ends were welded to angle connectors which in turn permitted connection to perforations on the upright through either hooks (Type A) or button grips (Types B and C). In Type D connection, additional devices were used to join the connector angles to the uprights. When the moment-rotation hysteretic loops of the beam-to-upright connections were compared to that of the cantilever tests described in the previous section, it was found that the loops from the portal tests exhibited a significantly higher initial stiffness. This result verified that moment-rotation characteristics of beam-to-upright connections depend on the bending moment-to-shear force ratio that is considerably higher in the portal tests due to the presence of the vertical merchandise loads.

2.4.1.3 Quasi-Static Cyclic Testing of Complete Storage Rack Systems

Quasi-static cyclic testing of complete storage racks represents an efficient experimental procedure to study the interaction between beams, uprights and connections under merchandise loads and simulated seismic lateral loads. The beams of the racks are loaded by either concrete blocks on pallets or real merchandise, and hydraulic actuators apply lateral loads to the uprights at the various beam levels. With this testing procedure, racks can be tested separately in their down-aisle or cross-aisle directions.

Krawinkler et al. (1979) performed four quasi-static tests of complete three-story storage racks. Two tests were performed in the down-aisle direction and two others were performed in the cross-aisle direction. The first two rack specimens contained hook beam-to-upright connectors, while the two others incorporated button grip connectors. The lateral load was applied only at the top level of the racks. It was found that for constant lateral displacement amplitudes the second load cycle led to a significant decrease in energy dissipation capacity while the third cycle was practically identical to the second one. Failure in the down-aisle direction typically instigated by weld cracking between the beam ends and the connector angles. This weld cracking in the cyclically loaded racks occurred at smaller lateral displacements than in the monotonically loaded racks. Up to the point when weld cracking was adverted, the ductility of storage racks in their down-aisle direction depended strongly on the axial load ratios in the uprights. For small axial load ratios, a very ductile behavior characterized by flexural plastic hinging in the uprights was achieved. It also was found that second order (P-Δ) effects greatly affected the lateral strength and stiffness of storage racks in the down-aisle direction. Finally, the ductility and energy dissipation capacity of storage racks resulted much larger in the down-aisle moment-resisting direction than in the cross-aisle braced frame direction.

2.4.1.4 Dynamic In-Situ Testing of Storage Rack Systems

The first published in-situ dynamic investigation of storage racks was performed in the mid 1970s at various distribution centers in the San Francisco Bay Area , John A. B, et al. (1973). Ambient and man-made vibration measurements were applied to representative steel industrial storage racks of standard pallets, drive-in and drive-through, cantilever, and stacker crane types to obtain a range of natural frequencies and damping ratios. The ambient vibration and man-made excitation measurements generated average response accelerations at the top of the racks on the order of 0.005g and 0.015g, respectively. The experimental results showed that the fundamental translational period of storage racks obtained by means of the empirical formula for building periods of the 1973 edition of the Uniform Building Code, was not reliable measured fundamental periods over a range of actual merchandise loading conditions averaged 0.6 sec in the down-aisle direction, and 0.2 sec in the cross-aisle direction. Torsional periods averaging 0.4 sec were identified in many of the rack configurations. It was noted that these period values would increase at least by 20 percent under response amplitudes representative of a strong earthquake. Measured structural damping ratios for storage racks averaged 2 to 3 percent of critical at rootmean- square response acceleration levels of 0.01-0.02g. It was noted that these damping values would increase at least by a factor of 2 under response amplitude representative of a strong earthquake for which significant energy dissipation would occur due to rocking, slippage and interaction of stored merchandise. Thus, it was concluded that a damping ratio of 5 percent of critical would be a reasonable value for storage racks under seismic excitations.

Krawinkler et al. (1979) subjected two full pallet rack assemblies with gravity loads to forced and free vibration tests to obtain information on natural frequencies, mode shapes and damping characteristics in the down-aisle and cross-aisle direction.

Measured fundamental periods averaged 0.7 sec in the down-aisle direction, and 0.5 sec in the cross-aisle direction. The vibration decay obtained from the free-vibration tests in the down-aisle direction exhibited a textbook example of Coulomb-type friction decay. At large amplitudes, the friction between the grip-type connectors and the perforations in the uprights caused significant damping (on the order of 2.5 to 3.5 percent of critical). Once the connectors locked up at smaller amplitudes, the damping dropped drastically to a very small value (on the order of 0.7 percent of critical). The damping characteristics in the cross-aisle direction were more constant with vibration amplitudes (on the order of 1 to 2 percent of critical).

2.4.1.5 Shake-Table Testing of Storage Rack Systems

Shake-table testing complete storage rack systems loaded with real merchandise represents the most direct experimental procedure to assess their seismic behavior. However, this type of testing is expensive compared to other testing procedures and only a very limited number of shake-table studies on storage racks have been performed to date.

The first published shake-table studies on storage racks in the United States, was performed in the late seventies on the 20-ft-square shake-table at the University of California, Berkeley, Chen et al. (1981). Four types of full-scale industrial steel storage racks were subjected to scaled ground motions of 1940 El Centro and 1966 Parkfield earthquakes . The ground motions were scaled so that the resulting base shear coefficients approximately equaled the design base shear coefficients of the 1979 edition of the Uniform Building Code for ordinary moment frame buildings (with K= 1.0) in the down-aisle directions and ordinary brace frame buildings (with K=1.33) in the cross-aisle direction. Generally, the storage racks performed well during the tests, with the exception of the drive-in stacker racks in the cross-aisle direction, for which considerable buckling was observed in the first story diagonal members. The fundamental periods of vibration ranged from 2 to 3 sec for the standard pallet and drive-in racks in the down-aisle direction and 0.5 to 1.0 sec for the standard pallet, drive-in, and stacker racks in the cross-aisle direction. They concluded that the racks could undergo significant inelastic deformations without suffering major damage in the down-aisle direction, but could only develop limited amount of inelastic deformations in the cross-aisle direction. Second order (P-D) effects contributed significantly to the response of the racks in the down-aisle direction.

Later on, Filiatrault (2001), five different back-to-back pallet racks loaded with real merchandise were tested on a uniaxial shake-table under a single component, scaled at various amplitudes, of the ground motion recorded at Canoga Park during the 1994 Northridge earthquake in California. Three of the tests were conducted in the cross-aisle direction, while the two other tests were conducted in the down-aisle direction. In general, the racks performed well. Significantly more flexibility, ductility, and energy dissipation capacity were observed in the down-aisle direction than in the cross-aisle direction. The fundamental periods of vibration averaged 1.4 sec in the down-aisle direction and 0.6 in the cross-aisle direction. No structural damage occurred in any of the rack configurations for peak ground motion amplitudes less than 0.42g.

Castiglioni et al. (2003) performed shake-table tests on four full-scale steel storage pallet racks loaded by concrete blocks mounted on pallets simulating content merchandise. The four specimens were chosen among six structures designed by two different European manufacturers based on Eurocode 8. The experimental results indicated that sliding of pallets occurred for ground motion intensities less than the considered design levels. Also, the diagonal bracing configuration in the down-aisle and cross-aisle directions has a significant influence on the seismic response of steel storage pallet racks. In particular eccentric bracing configurations can lead to significant torsional response. The authors stressed out the importance of a regular configuration of bracing systems.

Filiatrault et al. (2004) conducted a shake-table test at the University of Buffalo on four different pallet rack configurations, incorporating bolted beam-to-upright connections. All racks were tested in the down-aisle direction. The main ideas of the tests were to find out the variations of in-plane dynamic characteristics of the industrial storage racks during service life, plane \fundamental period of racks and assess the response of storage racks under strong ground motions. The test results revealed that the rotational stiffness of beam-to-upright connections is the main factor influencing the down-aisle seismic response of pallet racks. Furthermore, very ductile seismic behavior was observed in the down-aisle direction with peak interstorey drifts exceeding 7 percent without any sign of incipient collapse.

2.4.2 Experimental Research on Cold-Formed Steel Members

The lateral load-resisting systems of storage racks often include cold-formed steel bracing members. Therefore, research information related to the behavior of cold-formed steel structural members and systems have influenced the design of storage racks.

Cheng (1973) performed axial load tests on cold-formed steel open sections used as primarily load carrying structural members in storage racks. It was observed that local flexural-torsional buckling is the primary mode of failure for axially loaded perforated open section segments. An analytical expression was proposed to predict the axial load carrying capacity of these members.

Kotha and Peköz (2000) studied the behavior of cold-formed pallet storage racks with semi-rigid beam-to-upright connections and with flexible upright bases through Seismic behavior of steel storage pallet racking systems. A general moment-rotation relationship was established to model the beam-to-upright connection stiffness of pallet storage racks. Also, the upright base flexibility caused by base plate bending was quantified. Guidelines were provided to carry out nonlinear finite element analysis of storage racks accounting for these influencing parameters.

Table 2.3 Lists of Experimental Investigations Documented in The Public Literature, and the Various Testing Techniques Extracted From Castiglioni Book and other Researches.

Year	Authors	Testing Types (Number of Specimens)
1973	John A. Blume & Associates	In-situ dynamic tests (19)
1979	Krawinkler et al.	Cantilever tests (20), portal tests (6), quasi-static tests of storage rack systems (4), dynamic tests (2)
1980	Chen et al.	Shake-table tests (4), merchandise tests (2)
2001	Bernuzzi and Castiglioni	Cantilever tests (22)
2001	Filiatrault	Shake-table tests with real merchandise (5)
2003	Castiglioni et al.	Shake-table tests (4)
2004	Higgins	Cantilever tests (22)
2004	Filiatrault	Shake-table tests (4)
2004	Bernuzzi et al.	Cyclic tests(2)
2008	Bernuzzi et al.	Monotonic tests (61)
2008	Siders et al.	Shake table tests (4)
2010	Bajoria et al.	Cantilever tests(18)
2010	Prabha et al.	Double cantilever tests(18)
2012	Sangle et al.	Double cantilever tests(2)

(Castiglioni C.A,2008)

2.4.3 Review of analytical and numerical researches

As confirmed by the experimental research previously reviewed, the seismic response of storage racks in the down-aisle direction is strongly affected by the nonlinear moment-rotation response of the beam-to-upright connections. In the cross-aisle direction, contrast, the seismic response of storage racks relies on the characteristics of the bracing members used in the truss configuration. Therefore, numerical models that have been used to predict the seismic response of storage racks include these different lateral load-resisting Seismic behaviors of steel storage pallet racking systems. The analytical and numerical research associated with the seismic behavior of storage racks can be classified into two different types of models.

2.4.2.3 Linear Modelling

In linear modeling, the moment-rotation response of beam-to-upright connections is liberalized by simple linear rotational springs representing secant properties at the anticipated response level of the storage racks. For dynamic analysis, a corresponding linear viscous damping model is also used to represent the energy dissipation of these connections during inelastic actions.

John A. Blume et al. (1973) developed and analyzed equivalent lumped mass numerical models exemplifying selected storage racks in order to compare their predicted fundamental periods to measured values. Pinned upright bases were assumed for all rack configurations except for the cross-aisle direction of cantilever racks. Rigid beam-to-upright connections were assumed in the down-aisle direction. Reasonable agreement was attained between measured and computed storage fundamental periods.

Chen et al. (1981) conducted frequency analyses of linear mathematical models to evaluate calculated periods of vibration and mode shapes with those observed during low amplitude shaking table tests and pull-release free-vibration tests that they had earliar conducted. These calculated periods and mode shapes were then used to perform response spectrum analyses. The calculated fundamental periods of vibration were also used to find out the base shear coefficients according to the 1973 edition of the Uniform Building Code and to the ATC-3 procedure (ATC 1978). The results revealed that two-dimensional models with minimum net section properties and centerline dimensions were adequate for practical purposes. Modeling parameters such a semi-rigid beam-to-upright and base connections should be taking into account in theoretical predictions of rack response. It was also found that in the down-aisle direction, the lateral forces determined by the 1973 edition of the Uniform Building Code were approximately equivalent to those obtained from response spectrum analyses with intensity levels slightly more than 50% of the 1940 El Centro and 1966 Parkfield earthquake records. In the cross-aisle direction, however, the code lateral forces were approximately equivalent to 25% to 50% of the El Centro and Parkfield records. In the cross-aisle direction the lateral forces predicted by the UBC were higher than those predicted by the ATC-3 (ATC 1978) procedure. Opposite results were obtained in the down-aisle direction.

John A. Blume et. al. (1987) performed static and response spectrum analyses to investigate the applicability of the eccentric braced frame concept (Roeder and Popov 1978) to storage racks in order to improve their seismic behavior in the cross-aisle direction. The results of the study showed that, aside from a considerable savings in steel material, the eccentric bracing system could undergo significantly more inelastic deformations without structural instability than conventional bracing systems. Although the analytical results were promising, the authors recommended also that experimental investigations needed to be conducted before implementing the eccentric bracing system in storage racks. Such experimental results are not available to date.

Pekoz and Karakaplan (2010), have used the current design approach known as Linear Idealization of the moment -rotation relationship based on an empirically by taking the secant to the nonlinear moment –rotation curve. Analytical approach FEM based on linear analysis program LARSA 4D. However, the results show that care must be taking when treating the non-linear relationship as linear. This linearization moment-rotation relationship as linear. This linearization approach found to be too conservative for certain range of parameters

2.4.3.2 Non-linear models

The nonlinear response of beam-to-upright connections is followed over the time-history response of storage racks by the use of nonlinear moment-rotation hysteretic rules. This nonlinear modeling is used mostly for research purposes and rarely in design situation.

Baldassino and Bernuzzi (2000) have done a numerical study on the lateral-load response of the steel storage pallet rack systems commonly used in Europe. The results confirmed that the nonlinear rotational behavior of beam-to-upright connections influenced significantly the response of storage rack systems in the down-aisle direction. The numerical investigation confirmed the significant influence of the base plate connections on the overall rack response in both directions. They pointed out the need for test data on the non-linear moment rotation behavior of base upright connections. While Carlos Aguirre (2005) performed non-linear analyses of the rack structure under different seismic conditions, considering the measured moment-rotation curves. Results showed that non-linear calculated displacements were more than twice the displacements predicted with the classical linear analysis

2.5 CURRENT SEISMIC REQUIREMENTS FOR STORAGE RACKS

The 2003 edition of the NEHRP Recommended Provisions (FEMA 450) considers storage rack structures as non-building structures and references the seismic provisions of the RMI standard but also requires the use of mapped ground motions from the NEHRP Recommended Provisions for the design, and sets limitations on minimum base shear and seismic displacements. These requirements are meant to assure comparable results from the use of the RMI standard. It also helps to clarify and coordinate the multiple references to rack structures in the NEHRP Recommended Provisions and the different means by which rack structures are analyzed and designed. Of particular interest to rack designers is the minimum base shear coefficient of about 10 percent of the weight of the rack in areas of high seismicity. According to the 2009 edition of the NEHRP Recommended Provisions (FEMA 750), for the force and displacement requirements the reference should made to ANSI/RMI MH 16.1 standard.

The 2002 and 2005 editions of ASCE 7 allow the use of the RMI standard subject to the Provisions requirements on ground motions, limitations on minimum base shear and seismic displacement or drift default values, again distinguishing between the approaches for storage racks supported at the base (which are to be designed as nonbuilding structures) and those supported above the base (which are to be designed as architectural, mechanical, and electrical components and systems).

The 2003 edition of the IBC references the 2002 edition of ASCE 7 for its seismic requirements, thereby invoking the use of the RMI standard subject to the requirements and limitations imposed by ASCE 7. The 2006 edition of the IBC is expected to reference the 2005 edition of ASCE 7. According to 2003 edition of NFPA 5000, like the IBC, states that the design, testing, and utilization of industrial steel storage racks shall be in accordance with the RMI standard, and subject to the requirements and limitations imposed by the Section 9 (Earthquake Loads) of the 2002 edition of ASCE 7

2.6 CURRENT STORAGE RACK SEISMIC DESIGN PRACTICES

The 1972 edition of the RMI standard introduced seismic requirements for storage rack structures; it was renewed and published in 1979, 1985, 1990, 1997, 2002, 2004, 2008 and 2012. Each new edition was an expanded version of the previous one and each represented an effort to reflect the seismic provisions articulated in the most current editions of the NEHRP Recommended Provisions, ASCE 7, and the model codes. To provide higher levels of safety in locations open to the general public, the 1997 edition of the RMI standard, which was based on the 1994 NEHRP Recommended Provisions, introduced a higher importance factor for storage rack installations in places such as retail warehouse stores that increased the magnitude of the design seismic base shear. As noted above, the 2002 edition of the RMI standard remains largely based on the 1994 edition of the NEHRP Recommended Provisions and the 1997 UBC, both of that are now considered out of date. In producing the 2002 edition, RMI followed the ANSI canvassing process and the document is designated ANSI Standard MH16.1-2004, Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks

Single selective steel pallet storage racks are typically designed for seismic forces using the equivalent lateral force procedures found in model building codes and in the RMI standard. Storage rack structural systems generally are moment frames in the down-aisle (longitudinal direction) and braced frames in the cross-aisle direction (transverse direction). Storage racks placed in the middle of a floor area usually are attached back to back, whereas single rack configurations are used near building walls. Storage racks in store areas accessible to the public typically are loaded with pallets; however, in some merchandising situations, merchandise is stored directly on the shelves. Intermediate shelf heights vary depending on merchandising needs.

Also, there are presently no ductility type prescriptive provisions for connection designs. P-D effects are typically considered by a moment magnifier for member design. The procedures currently used to compute rack seismic loads vary depending upon whether the prevailing requirements are from the model building codes, the NEHRP Recommended Provisions or ASCE 7, or the RMI standard. In some cases, there is more than one acceptable method of calculating seismic loads.

2.7 RECOMMENDATION BASED LITERATURE SURVEY

At present, as for the design of storage rack structures few code of practice like draft Australian code AS4084 (1993), AISI (2001), SEMA (1985), FEM 10.2.0.8 and the specifications published by the Rack Manufacturer's Institute (RMI -2010) serves as guidelines for analysis and design of rack structures**.**

A look at the current state-of- the-art, as pointed out in the literature survey, shows that there is a need for new and better information to be incorporated in the future new edition of RMI, FEM and Australia Standards to more closely represent the behaviour of rack structure during seismic events which, in turn will allow the determination of more realistic and accurate requirements related to displacements, base shear, beam-to-column connector of the period, drift, and the overall rack structural behaviour and performance during seismic events. On the base of the knowledge acquired during the research project and on engineering judgment, many of these items could be tackled.

CHAPTER 3

NUMERICAL SIMULATION

3.1 INTRODUCTION

The main focus of this study is to investigate into the behavior of pallet storage rack under different ground motion records. To achieve this objective, a numerical simulation using finite element software methodology is adopted for this study. SAP2000 version 12 was used as a tool for simulation software and analysis of the response of these models under dynamic non linear time-history analysis. The key reason, for which the specific programme was chosen as an analysis tool, was its applicability to FE modeling, static, dynamic, nonlinear analysis and design a wide range of structural forms; buildings, bridges, towers, and other structures. It also includes the facility for different structure types, such as trusses. This section includes the necessary steps to quantify pertinent characteristics of the model. The details of the research design are outlined in this chapter.

Figure 3.1 SAP2000 Program Display.

3.2 GEOMETRY, PROPERTIES AND LOADING

3.2.1 Material Properties

The material properties of the frame members are given in Table 3.1

Table 3.1 Material Properties.

Modulus of Elasticity, E	24856 MPa
Poisson's Ratio, U	0.2
Minimum Yield Strength, Fy	228 MPa
Minimum Tensile Stress, Fu	310 MPa

3.2.2 Frame Section Properties

The tables below contain section properties as obtained from the mass properties using AUTO CAD software. This data was used for input into the SAP 2000 model

Table 3.2 Properties of Upright Section.

Area	921.9723 mm^2
Perimeter	620.6482 mm
Moments of inertia about X-axix	672688.3139 mm^4
Moments of inertia aboutY-axis	1854787.3294mm^4
Radii of gyration about X-axis	27.0115mm
Radii of gyration about Y-axis	44.8527mm
Section Modulus about X	672688.3mm^3
Section Modulus about X	30913.122mm^3
Torsional Constant	5585.8338

Table 3.3 Properties of Beam Section.

Area	561 mm^2
Moments of inertia about X-axis	1362200.8 mm^4
Moments of inertia about Y-axis	273020.75 mm^4
Shear area in X-direction	420 mm^2
Shear area in Y-direction	150 mm^2
Section Modulus about X-axis	19460 mm^3
Section Modulus about X-axis	10920.83 mm^3
Plastic Modulus about X-axis	24464.25 mm^3
Plastic Modulus about Y-axis	11841.75 mm^3
Radius of gyration about X-axis	49.2764 mm
Radius of Gyration about Y-axis	22.0606 mm
Torsional Constant	723873.5

Table 3.4 Bracing Section Properties.

Area	561 mm^2
Moments of inertia about X-axis	1362200.8 mm^4
Moments of inertia about Y-axis	273020.75 mm^4
Shear area in X-direction	420 mm^2
Shear area in Y-direction	150 mm^2
Section Modulus about X-axis	19460 mm^3
Section Modulus about X-axis	10920.83 mm^3
Plastic Modulus about X-axis	24464.25 mm^3
Plastic Modulus about Y-axis	11841.75 mm^3
Radius of gyration about X-axis	49.2764 mm
Radius of Gyration about Y-axis	22.0606 mm
Torsional Constant	723873.5

3.2.3 Elements and profiles

The selection of cross-section size and lengths employed was designed to ensure that the specimen member slenderness covered most practical range. The uprights as well as the pallet beams of the structures were simulated as beam elements whereas all the bracings were simulated as truss elements. In Sap2000 all the above elements are referred to as Frame Sections. The profiles of the elements were defined in SAP's General Section were the geometry and the materials can be specified. The 3D frames refers to modeling the entire pallet rack frame and open-section beam elements are used to model and braces.

The dimensions of the beam and column used in this study are shown in Figures 3.1 and 3.2

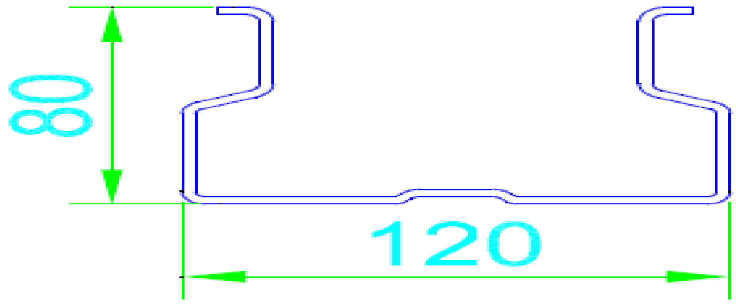


Figure 3.2 Upright Section and Dimensions.

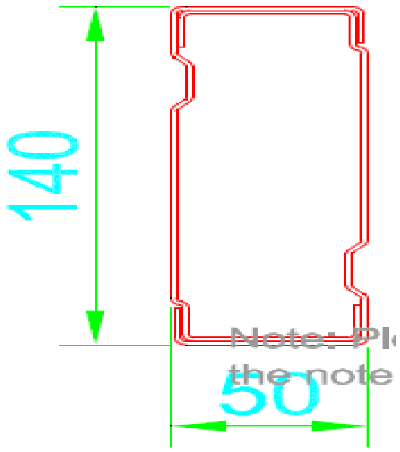
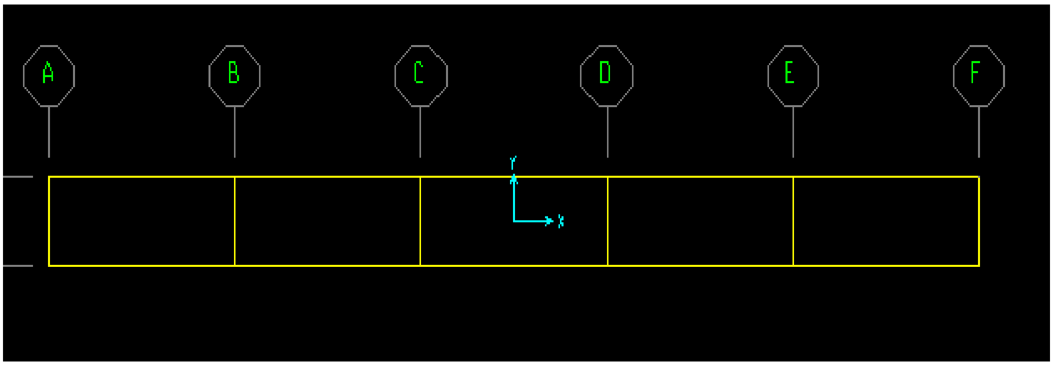


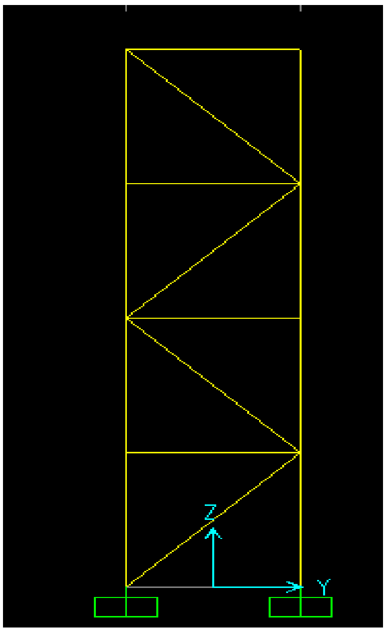
Figure 3.3 Beam Section and Dimension.

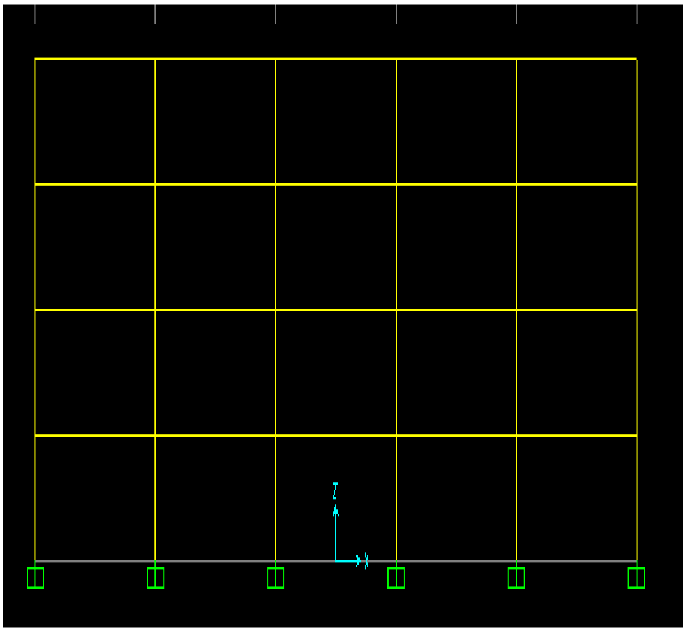
3.3 GEOMETRY AND MODEL CONFIGURATION

All the configurations examined were according to a X-Y-Z grid defining five bays, a front and a rear vertical level and four floors. The grid was divided in a primary grid and a secondary grid. Global axes are axis X is parallel to picking bays (down-aisle direction), axis Y is parallel to upright frames (cross-aisle direction), and axis Z is the vertical direction all the secondary nodes in general. The 4-story, 5-bay frame is 5.6m high and 15m wide was selected from the frames studied to represent a typical rack used for merchandise storage.



(a) Plan View.





(a) Side Review

(c) Front View

Figure 3.4 2D View of the Modal Configuration.

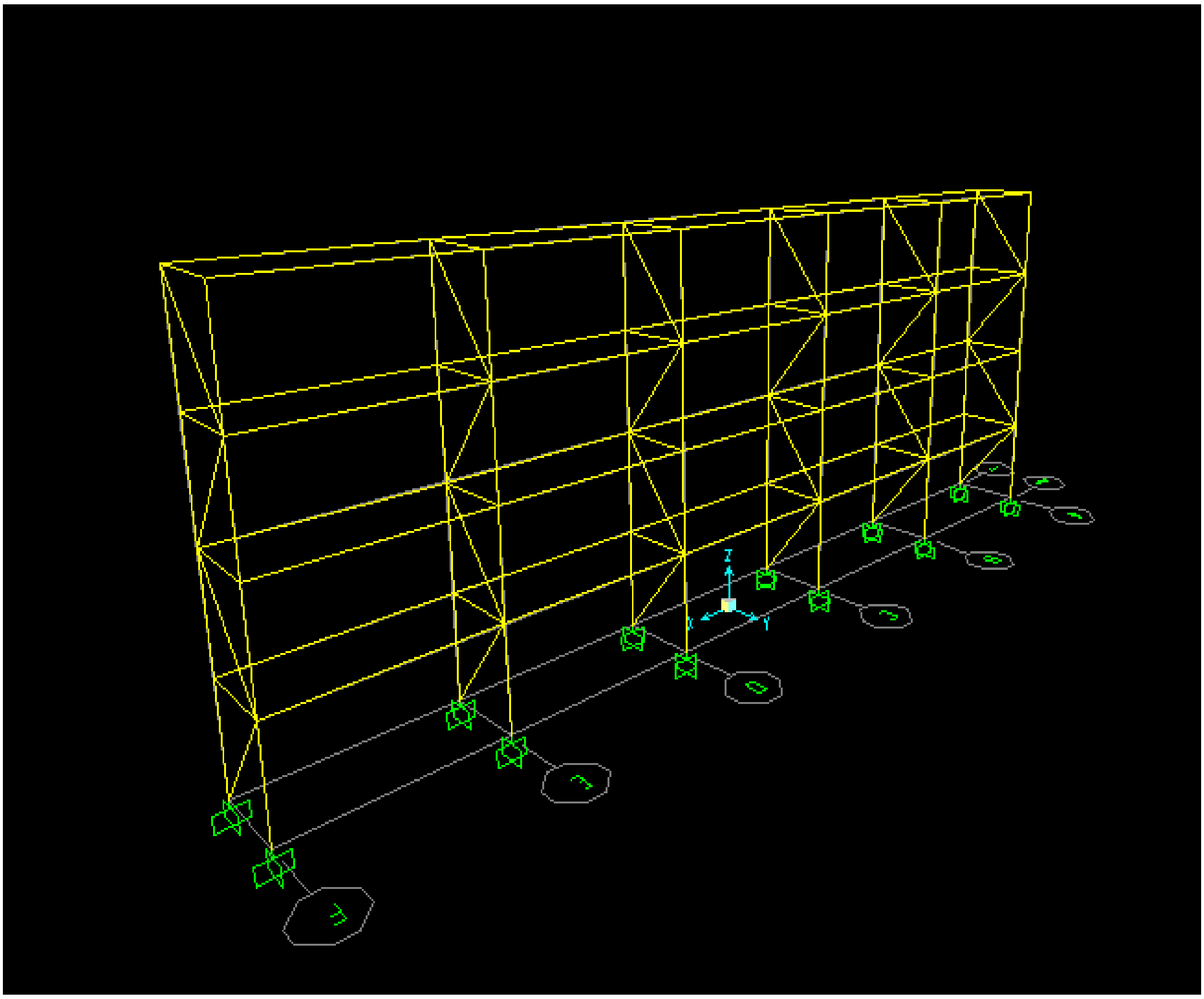
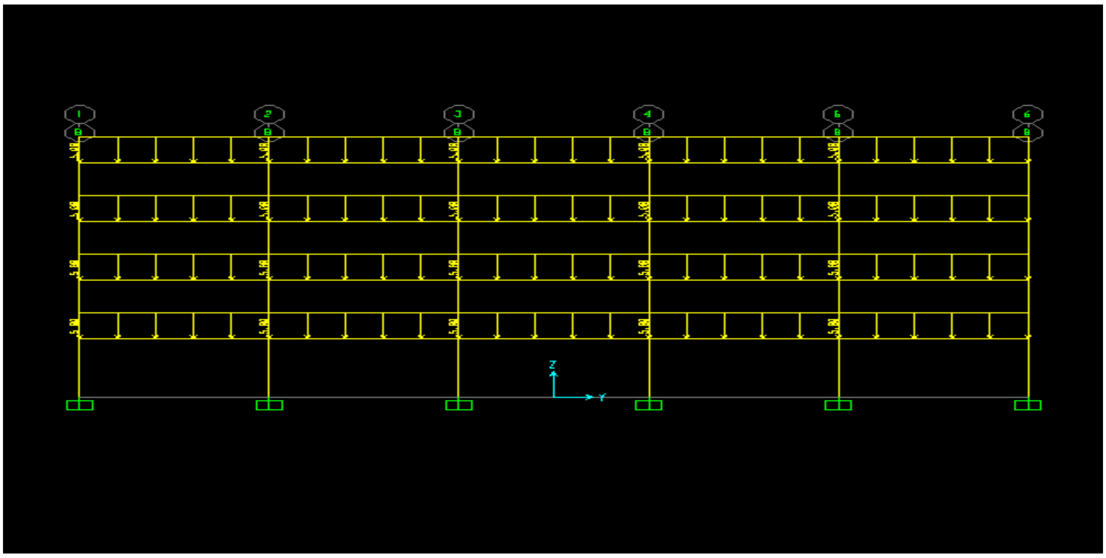


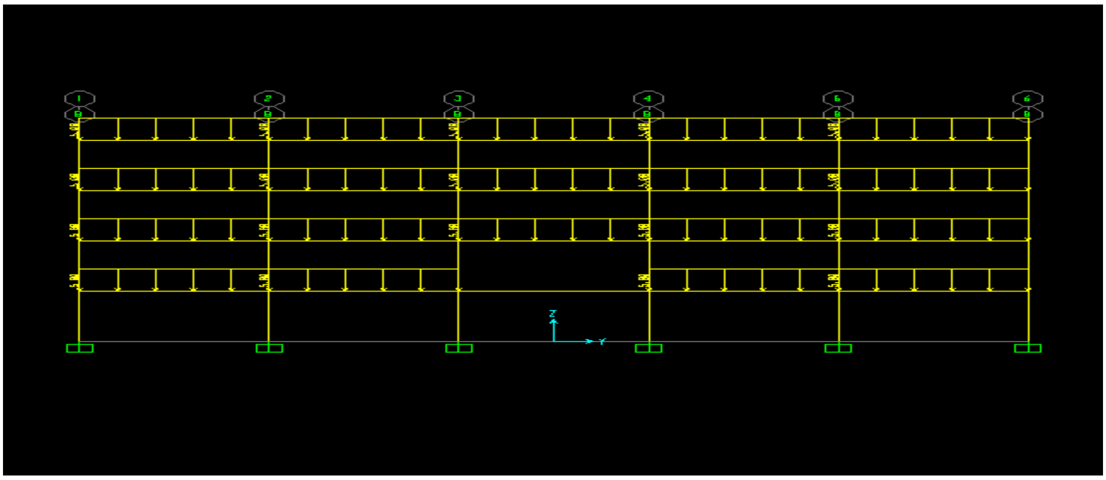
Figure 3.5 3D View of the Model Configuration

3.4 LOADING PATTERN

The applied loads are vertical and horizontal. The vertical loads consist of the dead load of the structure as well as the pallets. Pallet loads were applied in the form of distributed loads of equal magnitude (5kN/m) of each bay of the bays of the frame. To get the pallet effective weight, the 0.67 coefficient for the pallet weight comes not from the average load but from an evaluation of the amount of load that participates in developing the dynamic seismic force. FEM 10.2.08 stated that experience has shown that the whole mass of the merchandise stored on the storage rack system does not participate entirely to the inertia generated from the ground motion. There is some friction inducing energy dissipation for the relative movement between the storage racks. To get the pallet effective weight, the 0.67 coefficient for the pallet weight comes not from the average load but from an evaluation of the amount of load that participates in developing the dynamic seismic forces



(a) Case 1 Loading Condition



(b) Case 2 Loading Condition

Figure 3.6 Loading Pattern.

3.5 BEAM-TO-COLUMN CONNECTION BEAM END CONNECTORS

The behavior of the beam end connector is crucial for the stability of the whole structure since it provides the frame action (moment resistance) longitudinally. These are hooked, and their calculations are only experimental in order to specify the rotational stiffness of the connection and its strength. Moment-rotation curve were used to simulate, they were input in the models using partial fixity releases (Konstantinos et. al, 2013). Taking account two extreme conditions, namely rigid and pinned have been considered for beam-to-column connections. For the joint at each end of the beam, three different forms of behavior were considered:

3.5.1 Pinned Connection

Pinned connections, no end moment is developed and the bending moment diagram can be determined by statics, with a maximum moment as shown in the Figure 3.7(a)

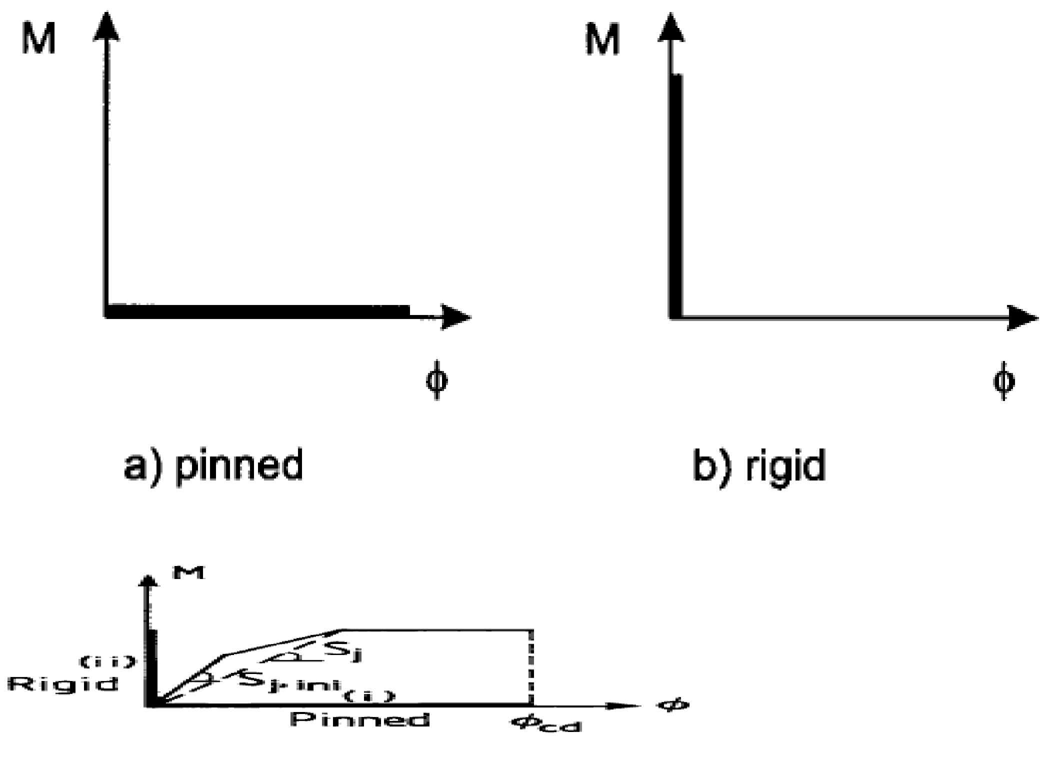


Figure 3.7 Major Connection Characteristics (Elnashai et. al , 1994).

3.5.2 Rigid Connection

In rigid connections, no rotation occurs at the ends of the beam. In conventional design of continuous structures, the connections are proportioned to resist whatever end moments result from the global analysis of the structure, and the connections resistance provided is therefore as great as that of the connected beam.

3.5.3 Semi-rigid Connection

For very high values of stiffness, the behaviour resembles that of the rigid connection. In such cases the connection can be assumed to be rigid for the global analysis. Similarly, a very flexible joint may be assumed to be pinned. However in the interests of economy the designer need to choose a form of connection whose stiffness does not approximate to either rigid or pinned behaviour. In this way arrangements, classification by rotational stiffness therefore to model the structural frame in a realistic manner whilst providing freedom to choose the connection stiffness most suited to the particular rack connection.

A well known method of allowing for semi-rigid connections action in global analysis is to modify the beam stiffness to an effective value. For similar reasons, the acceptable boundaries for the rigid and pinned idealisations are expressed in terms of beam stiffness related to initial joint stiffness. Determined in terms of acceptable errors resulting from the assumption of fully-rigid or truly pinned behaviour. Therefore the beam–column connections were semi-rigid and the experimental moment-rotation curves were incorporated into the connection behaviour (Abdel-Jaber et al., 2005).

Since the moment-rotation response at the beam-to-column connection is non-linear and affected by the looseness of the connections. To achieve an accurate semi-rigid joint, the inelastic beam to column connection means considering the connection to be semi-rigid, setting the moment and rotation relationship as elastic plastic. In this study, bending test was used to determine the moment-rotation behaviour of the semi-rigid connection of racks. and it was found to be stiffness K_{Θ}=153.68 kN-m/radian.

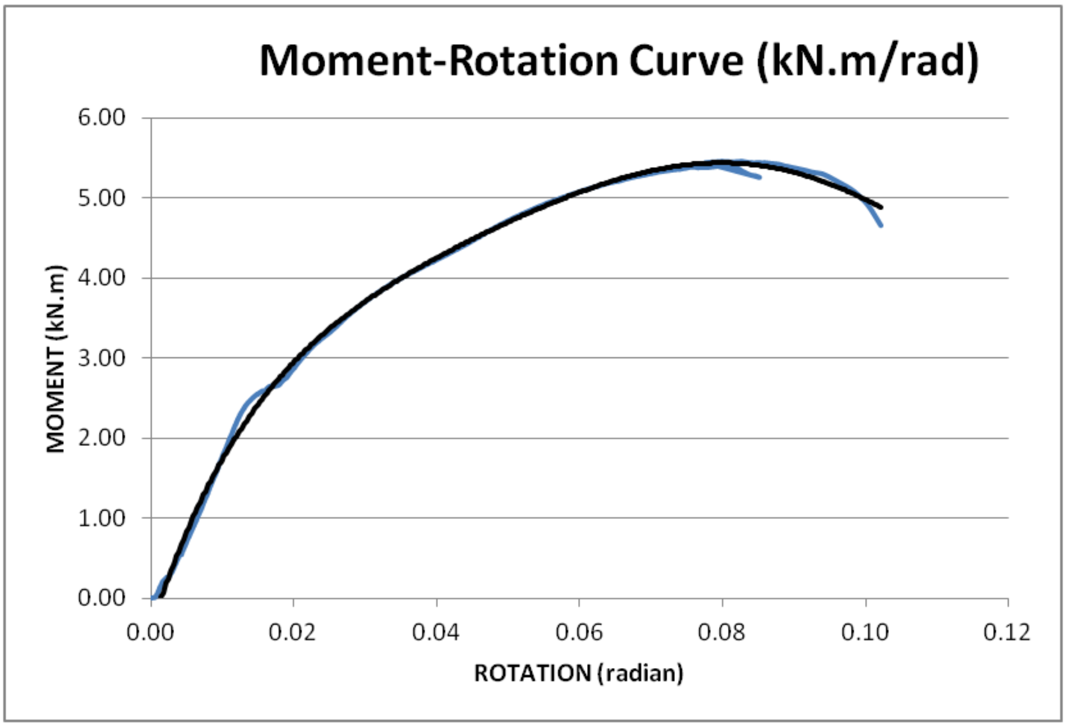


Figure 3.8 Experimental Moment–Rotation Curve for Beam End Connection.

3.5.4 Column Base

For this study the column base connection is assumed to be fixed in all six degrees of freedom, as base plates are fixed with two or more bolts normally (Bajoria, 2009).

3.6 TIME HISTORY ANALYSIS

Linear methods, such as modal analysis and response spectrum technique can not accurately simulate the structural behavior of such a highly non-linear Structure to seismic ground motions. An accurately simulate the structural response of non-linear time history analysis. It appears to be most accurate and rational method of seismic evaluation is the time history analysis. As verified by previous studies, the most accurate method of seismic demand prediction and performance evaluation of structures is nonlinear time history analysis. However, this technique requires the selection and employment of an appropriate set of ground motions and having a computational tool able to handle the analysis of the data and to produce ready-to–use results within the time constrains of design offices. Nonlinear time history analysis found to be a useful analysis tool for the conventional pallet racking systems giving good estimates of the overall displacement demands, base shears and plastic hinge formation.

3.6.1 Earthquake ground motions

The nonlinear Time history analysis of the frames was subjected to three different ground motions obtained from the Pacific Earthquake Engineering Research Center (PEER) Ground Motion Database. These are Sakarya (Turkey), Loma Prieta (California), and Kobe (Japan) Earthquake ground motions. The rack models were analyzed both in down and cross aisle directions. Model of conventional pallet racking systems was carried out using the SAP2000 finite element program. Three connection cases were considered for each of the ground motions.

3.7 MODELING ASSUMPTIONS

In order to study the dynamic behaviour for the down aisle of pallet racks, the following assumptions similar to Bajoria (2010) were made:

1. Uniform beam to upright connection is used throughout the frame.
2. The beams are spaced uniformly along the height of frame
3. All connections of the racks experience simultaneously similar rotations at all times. This assumption implies that the connection rotational stiffness is smaller than the rotational stiffness of the beams and uprights.

CHAPTER 4

NUMERICAL RESULTS AND DISCUSSION

4.1 INTRODUCTION

The major objectives of the investigations reported in this thesis were to increase understanding of the behavior of pallet rack under the effect of ground motions; the nature and type of beam-column connections on structural systems through nonlinear time history analyses. The primary findings of this thesis are presented in the following subsections.

4.2 PRESENTATION OF THE RESULT

Results showing the characteristics of five story rack frame with different connection configurations are summarized in proceeding Tables. For comparison, the pinned and rigid-connection cases are also included in the studies. The results obtained from time-history analysis are compared with respect to the fundamental period, the maximum displacement, maximum and base shear under seismic loads.

4.2.1 Result from the Modal Analysis

Model analyses are used to determine a structure's vibration characteristics such as natural period, mode shapes, mode participation factors (how much a given mode participates in a given direction). Most fundamental of all the dynamic analysis types benefits of modal analysis. It allows the design to avoid resonant vibrations or to vibrate at a specified frequency by giving engineers an idea of how the design will respond to various types of dynamic loads. Since a structure's vibration characteristics determine how it responds to any type of dynamic load, always perform a modal analysis first before trying any other dynamic analysis.

The fundamental period of vibration (T) is the amount of time, in seconds, the structure will take to undergo one complete cycle of motion when it is laterally displaced and released. Table 4.1 compares the values of period of vibration obtained from the modal analysis. For the three connection types, lowering the natural frequency (increasing natural period) decreases the effect of seismic force.

Table 4.1 Comparison of Fundamental Period Results.

Mode	Rigid Connection		Semi-Rigid Connection		Pinned Connection	
	CASE I	CASE II	CASE I	CASE II	CASE I	CASE II
1	2.797466	2.796493	3.35102	3.350081	6.86404	6.863118
2	1.14217	1.141685	1.171322	1.170888	1.250283	1.249506
3	0.81508	0.812973	0.903422	0.90115	1.115341	1.113067

The variation of the first three periods of vibration of rack fames with various connection types is given in Table 4.1. Clearly, the stiffness of the connections affect the periods of the frames significantly. Modeling racks as pinned frame leads to higher period of vibration. For rigid frames the first, second and third period are 2.7, 1.14 and 0.81seconds respectively. As semi-rigid is taken into account the first, second and third period become 3.35, 1.17 and 0.9 seconds. It is interesting to see that the relationship between the periods of vibration of the rack frames is almost linear.

Table 4.2 Maximum Modal Displacement

Connection Types	Load Case	Maximum Top Displacement (m)	
		UX(mm)	UY(mm)
Rigid	Case 1	106.778	0.061
	Case 2	106.827	0.062
Semi-rigid	Case 1	110.319	0.049
	Case 2	1103.59	0.049
Pinned	Case 1	121.203	0.014
	Case 2	121.221	0.014

From Table 4.2, it is seen that assuming rigid connections in the analysis is slightly unconservative. A maximum horizontal displacement of 106 mm is determined for rigid frame compared to 111 mm for the semi-rigid frame determined and 121mm for pinned connection.

4.1.2 Maximum Displacement under Earthquake

Table 4.3 Maximum Displacements Comparison.

	Connection Type	Load Case	Maximum roof Displacement	
			UX(m)	UY(m)
Kobe	Rigid	Case 1	0.058831	0.000453
		Case 2	0.058902	0.000442
	Semi-rigid	Case 1	0.081107	0.000794
		Case 2	0.080899	0.000786
	Pinned	Case 1	0.29871	0.001247
		Case 2	0.298156	0.001199
Loma Prieta	Rigid	Case 1	0.138713	0.000609
		Case 2	0.138303	0.000609
	Semi-rigid	Case 1	0.140153	0.000837
		Case 2	0.141103	0.000843
	Pinned	Case 1	0.134476	0.002478
		Case 2	0.132825	0.002368
Sakarya	Rigid	Case 1	0.30341	0.000906
		Case 2	0.300606	0.000884
	Semi-rigid	Case 1	0.287893	0.001447
		Case 2	0.285704	0.001435
	Pinned	Case 1	0.102741	0.002401
		Case 2	0.102599	0.002339

From table 4.3, it could be seen that the maximum roof displacement under seismic effect of various ground motions is showing a similar trend as that obtained from the modal analysis. However, the maximum displacement obtained from Sakiria earthquake is having semi-rigid frame with highest value. This may be as a result of the connections, ability to undergo inelastic deformation without collapsing and higher energy dissipation as confirmed by the value of the base reaction obtained.

4.1.3 Maximum Base Shear

The table shows comparison of the maximum base shear development for the various models. Changing the stiffness of the connection increases the fundamental period of the frames. This is due to the reduction in the base as shown in the table above.

Table 4.4 Comparison of Maximum Seismic Base Shear.

Ground Motion	Connection Types	Load Case	Maximum Base Shear (kN)	
			FX(kN)	FY(kN)
Kobe	Rigid	Case 1	106.2	0.0001369
		Case 2	101.961	0.00009731
	Semi-rigid	Case 1	93.481	0.085
		Case 2	90.12	0.085
	Pinned	Case 1	57.264	0.0001101
		Case 2	56.15	0.0001231
Loma Prieta	Rigid	Case 1	140.537	0.0001917
		Case 2	139.73	0.0001596
	Semi-rigid	Case 1	147.553	0.208
		Case 2	139.325	0.213
	Pinned	Case 1	153.694	0.000109
		Case 2	151.982	0.00012
Sakarya	Rigid	Case 1	237.54	0.0001489
		Case 2	234.539	0.0001946
	Semi-rigid	Case 1	145.9	0.148
		Case 2	143.2	0.151
	Pinned	Case 1	169.877	0.0001748
		Case 2	167.03	0.0001968

The maximum values of base reaction of 4 storey frame when semi-rigidity is provided for Kobe Earthquake, Loma Prieta Earthquake, and Sakarya Earthquake, load are given in Table 4.4. It can be observed that maximum base shear decrease effectively from 237kN to 57kN for different earth-quake load cases.

4.3 COMPARISON OF THE RESULT

A summary of the main results obtained from the numerical analysis is presented with the aim of allowing understanding of the actual dynamic behavior of steel pallet racks under seismic conditions.

4.3.1 Period of Vibration

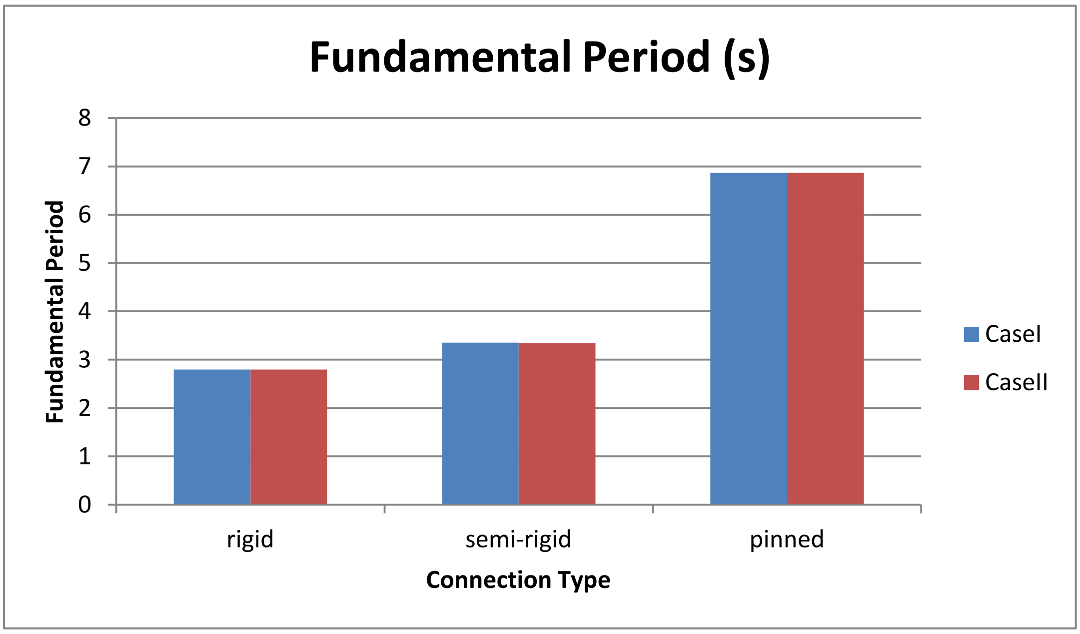


Figure 4.1 Comparison of Fundamental Period Result.

The natural periods of the first three modes of all the three connection cases are compared in Figure 4.1. From this figure, it is seen that the natural periods of the frames were almost the for a particular connection type regardless of the loading cases. Comparing the values of the period for semi-rigid frames with those of rigid and pinned frames, the first mode period is about 1 second higher than the two connections. It can be seen that fundamental period for a semi-rigid frame is longer than that of rigid but lower than pinned connections. This may be due to the effect of structural stiffness altering the time periods. Longer period produces lower frequency which in turns reduces the effect of seismic event .Thus; flexibility cause by semi-rigidity of the rack frame serves as to dissipate the energy imparted to the structure from the earthquakes.

However, the movement of the rack system should be enough to achieved increasing period to a desired level, while at the same time not exceeding an acceleration threshold over which product will fall off the shelves.

4.3.2 Comparison of Displacement Result

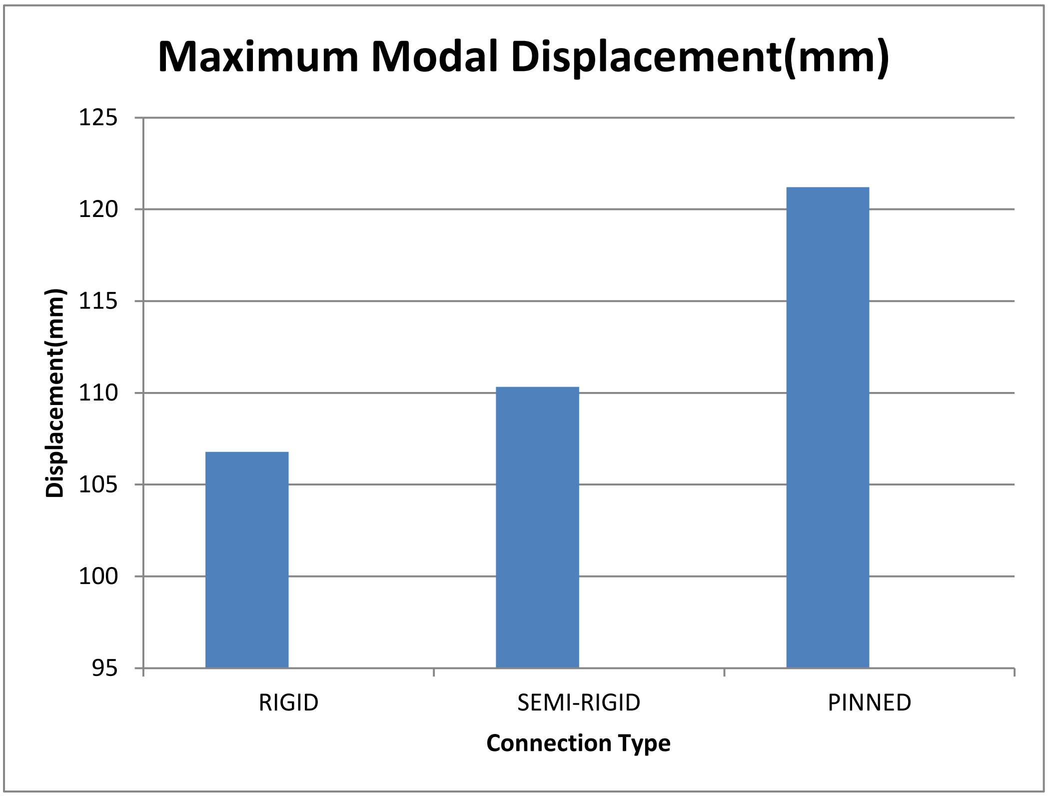


Figure 4.2 Variations in Modal Displacement

Figure 4.2 depicts the variation of maximum modal displacement response of the top level obtained for the numerical model with different connections based on stiffness. As expected the larger the flexibility of the connection, the larger the top maximum displacement. It clearly shows that, under the above semi-rigid, constitutes an optimum of 111mm which is 5% higher than rigid frame. For pinned frames, displacements are slightly overestimated (about 8%) higher than the semi-rigid case.

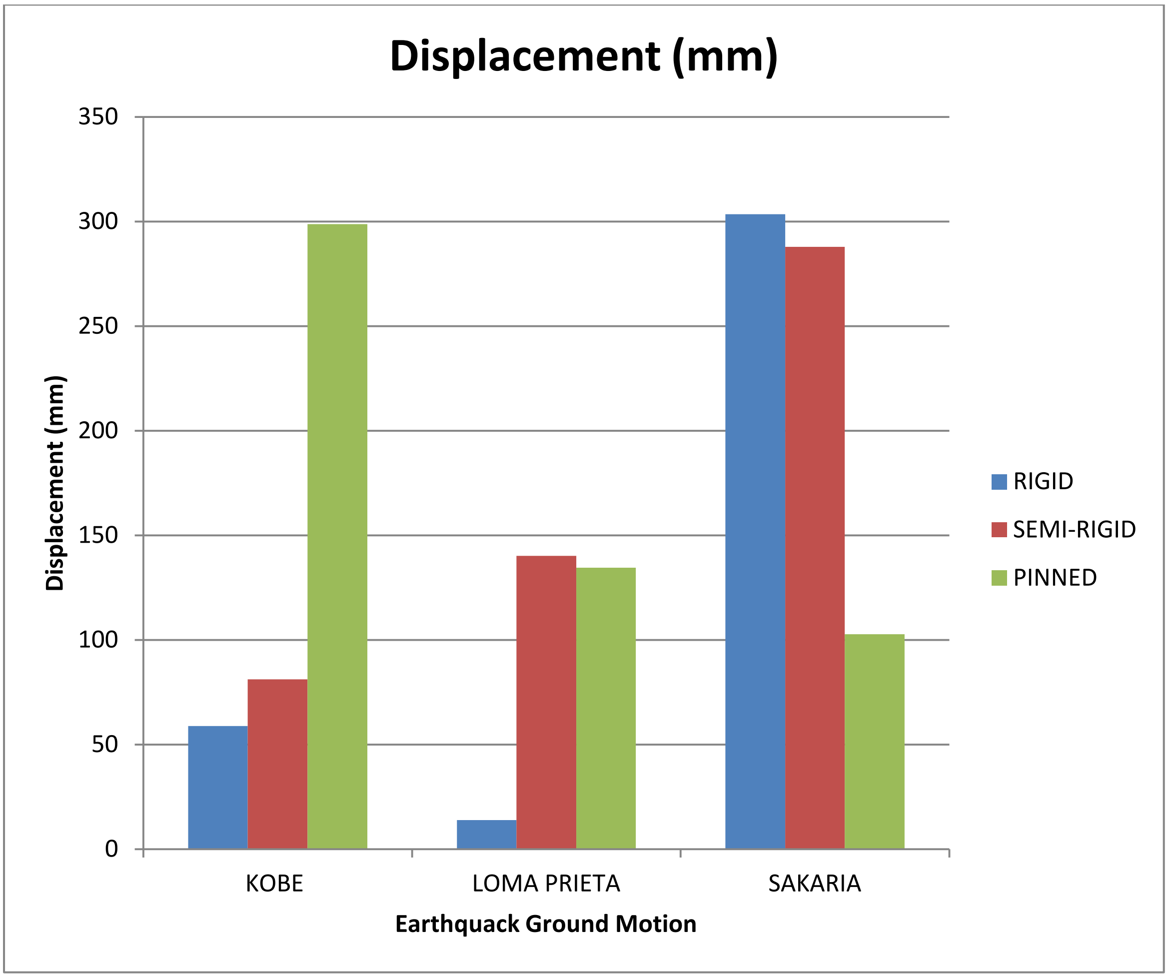


Figure 4.3 Variation of Displacement response time History at the maximum displacement.

The comparison between the rigid, the semi-rigid and the pinned connections for the two loading cases considered is presented in Table 4.20 while a typical output graphs from the program are given in the Appendix B.

The displacement at the top of the frame (joint 10) is response quantity of interest. It is seen that characteristics of the seismic response are almost similar among the two load cases. It is also notable from the Figure 4.4 that the time-history record employed in the seismic analysis influences the maximum displacement due to the differences found in the Kobe, Loma Prieta and Sakarya. It is noted that the three chosen ground motions featured significantly different maximum displacement as observed in their respective responses. The displacement of semi-rigid connection is larger than that of rigid frames with the exception of Sakarya Earthquake. This could be possibly due to the inherent modelling assumptions made in the course numerical design.

4.3.3 Maximum Base Shear

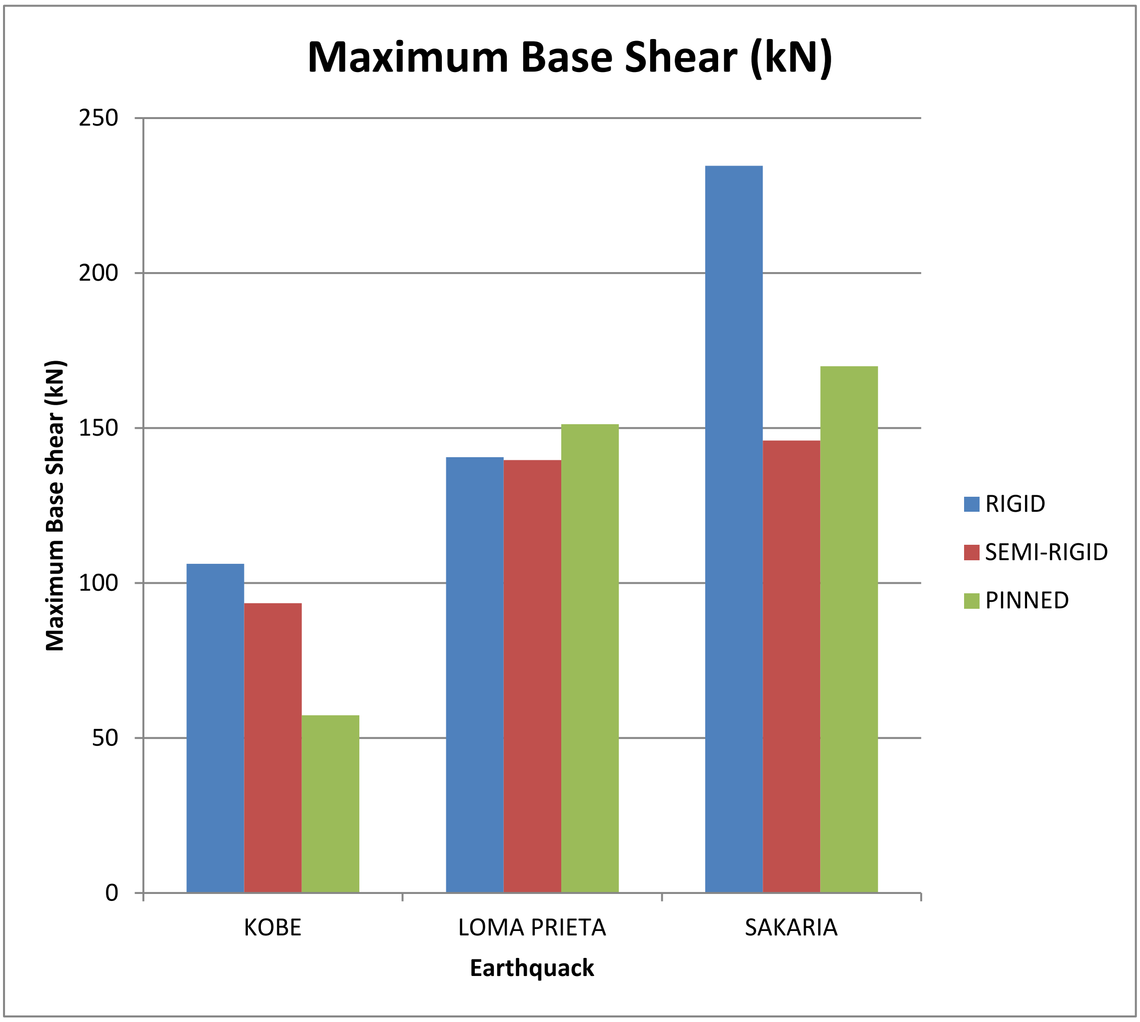


Figure 4.4 Bar Chart Showing Variations in Seismic Base Shear.

Figure 4.4 shows the variation of the base shear of the frame at the base. It is seen that rigid connection can result in the large base shear force response for all the three earthquake considered in this study. For semi-rigid connection, the maximum base reaction becomes the smallest. For instance, the base shear for Sakarya earthquake with perfectly rigid connection is 235kN, with pinned connection is 169kN, leaving 28% reduction of the base shear response. With semi-rigid connection the base shear further reduced to145kN. Thus, the overall earthquake resistance of the pallet rack under study could be significantly enhanced through the use of semi-rigid connections.

4.3.4 Maximum Interstory Drift Ratio

Interstory drift is a measure of how much one floor or roof level displaces under load relative to the floor level immediately below. It is generally expressed as a ratio of the difference in deflection between two adjacent floors divided by the height of the story that separates the floors.

Table 4.5 Maximum Interstorey Drift.

Connections	Rigid	Semi-rigid	Pinned
Maximum Drift Ratio	0.01557	0.02071	0.0431

According to the NEHRP Recommended Seismic Provisions sets maximum permissible interstory drift limits based on a structure's Occupancy Category and construction type Δa, varies from 0.007 to 0.025 depending on the structure's Occupancy Category and construction types. Hence the intertorey drift for rigid and semi-rigid are within the acceptable limit

4.4 OVERALL DICUSSION

A four story rack structure was studied, using three types of connections (rigid, semi-rigid, and pinned) excited by three different earthquakes. From the results obtained, it was observed that as the structure moved from rigid to semi-rigid connections, the response shifted from a shear to a bending mode of deformation. In addition, although the interstory drift ratios increased as the frame became more flexible. Furthermore, since the earthquakes considered had predominant periods in the low period range, the base shear of the structures was reduced, potentially leading to a more economical design if a semi-rigid frame was used instead of a rigid or pinned frame

All three models are producing very close results for the same connection type, which translates the fact that, the two loads cases are not playing an important role and connections are behaving in a similar manner. There is no apparent difference between Case 1 and Case 2 in terms of period of vibration for all the three connection types considered. These results provide confirmatory evidence that load case 2 cannot be considered as the critical loading contrary to the FEM 10.2.02: (The Design of Static Steel Pallet Racking) as such static and dynamics design cannot be treated in the same way.

Regarding the mode shapes, as anticipated the first mode was lateral either lateral or transitional while the second and third modes were either transitional or torsional. It can be seen that, when modifying the connection stiffness, the periods associated to the vibration modes differ but the modal shapes remain more or less unchanged. Therefore, the only difference is in the period of vibration. For example the first period torsional mode of vibration for rigid frames looks similar to that of semi-rigid but having higher period of vibration.

The first lateral mode displacement can be seen at approximately 106 mm, corresponding to a fundamental period of vibration of 2.8 seconds. Likewise in the semi-rigid frames, the first mode of lateral vibration can be seen at approximately 111mm, corresponding to a period of 3.4 seconds. In the pinned frame, the maximum lateral displacement found to be approximately 121 mm. Therefore, seismic action on a given rack structure depends on the dynamic properties of this structure particularly on its first natural period of vibration. The geometrical second order effects (non-linear) tend to decrease the lateral stiffness of the pallet rack structure, leading to increased values of the period.

The comparison of the results discussed and analysed, base shear as the maximum expected lateral force that will occur due to the seismic ground motion found be dependent on the probability of the ground motion, the frame joint connections associated with rack structural configuration and the natural period of vibration.

From the numerical results, time history data related to the story displacement of each top of the structure were measured. The maximum displacement for rigid observed at the top story was 138 mm , while the maximum displacement for semi-rigid and pinned frames were 140mm and 298mm respectively. Also, the variation of the seismic base shear with time was measured, with a maximum base shear of 237 kN

According to modern building codes structures should be design and detail to develop inelastic ductile behavior under extreme earthquake. This could be achieved through the use of carefully detailed seismic resisting systems capable of withstanding huge inelastic deformation without degradation. The result indicates good behavior of the semi-rigid frame under seismic loads, it reveals that the structure utilizes its capacity lying in the inelastic zone. Therefore semi-rigid connection is more effective than the usual, and that they can deform in a ductile manner, dissipating more seismic energy..

Similarly, economy studies in many countries have indicated possible benefits from the use of the concept of semi-rigid connections. Their advantages, in terms of lower construction costs and simple fabrication, are therefore not till now utilized in seismic design. Conversely, dependence on the rigidity of fully-welded connections under earthquake loading has recently come under question, particularly in Japan, as a consequence of difficulties associated with quality control of welding processes (Elnashai et. al **,** 1999).

The study concluded that semi-rigid connections did account for precise stiffness of the rack frames, also adding a considerable dissipation of seismic energy and generally providing significant reductions in the base shear a structure experienced. The real response of the semi-rigid frame was found to be intimately dependent upon the dynamic properties of the frame and the characteristics of the ground motion, requring detailed analysis for each semi-rigid frame prior to the construction of the frames. The present study provided a strong case for the possible benefits of semi-rigid steel frames and motivated the need for detailed, accurate, and reliable analytical models of the connection.

CHAPTER 5

CONCLUSION AND RECOMMENDATION

5.1 CONCLUSION

The objective of the study was to obtain basic knowledge on the response behaviors that govern the seismic behavior of cold formed pallet racks. Although the study was limited to pallet rack configuration, it is expected that much of what has been learned can be applied to other rack of similar configurations. On the basis of present study and reviewed literature the following conclusions can be drawn:

1. The elements which control the seismic response of storage racks are the beam-to-column connections. Seismic performance of pallet rack can be improved by accurate modeling the beam-to-column connection as semi-rigid which absorbs the input energy during earthquake.
2. For the semi-rigid frames, the base shear effectively reduces, hence making the structure cost effective.
3. The overall performance of rack frame under study is significantly enhanced through the use of semi-rigid connection. It is apparent that connection flexibility and stiffness affects the fundamental frequency, Shear force distribution and deformation in frames, and must be considered in a dynamic structural analysis.
4. The behavior of the beam-to-column connections can be represented by partial releases in SAP2000 and the stiffness characteristic value could be determined experimentally. The determination of the response characteristics of beam and upright frames require tests of rack assemblies which allow proper simulation of boundary and loading conditions.

5.2 RECOMMENDATIONS AND FUTURE WORK

A non-linear time history analyses of the storage rack frames with semi-rigid stiffness is worthy to be investigated in the time domain in the future. Furthermore, as a result of this research it is intended that adequate design recommendations will be given to allow structural designers to push the limits of the material and connection configuration further and increase the economy of the pallet rack frame design. The following are some of the aspects that would increase the strength of the study:

1. The present study considers only nonlinear time history analysis. It may be extended to P-delta dynamic analysis and response spectrum dynamic analysis.
2. Further research need to be carried out integrated full scale experiment to validate the numerical simulation. As the use of both analytical and experimental techniques simultaneously to study the frame behavior harnesses the economy of analytical studies, while also capturing real experimental data that can be used to validate the Finite Element model.
3. There is also need for effective research on how to reduce the effect of other dynamic loading like wind load, bombard load, vibration load and also seismic load when subjected to pallet racking system.
4. Similar to other studies such as Rasmussen, K.J.R et al., (2009), the study does not consider perforations for the sake of simplicity. However, it would be of great interest to extend the study to include upright cross-sections with perforations.

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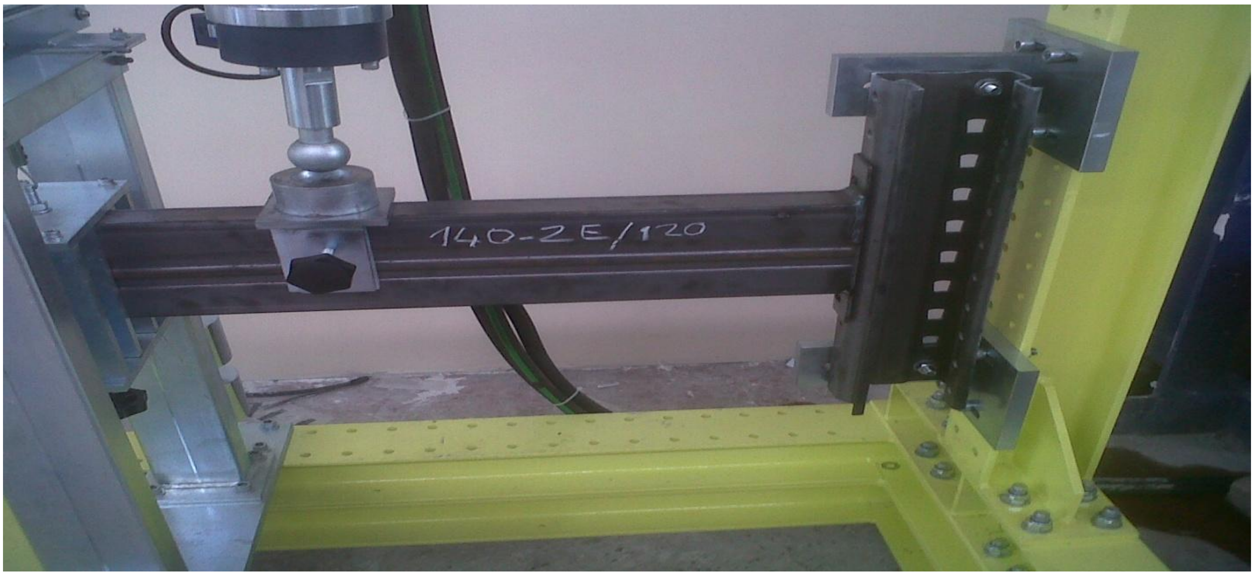
APPENDICES

APPENDIX A: DETAILS OF BENDING TESTS (ALTERNATIVE 1 THE CANTLIVER TEST)



Setup for Beam End Connector Bending Test According to the European Standard (European Committee for Standardization, 2009)

Close Up

Failed loading joint.

APPENDIX B: DETAILED CALCULATION OF STIFFNESS

n	K_s
3	3.15
4	2.68
5	2.46
6	2.33
7	2.25
8	2.19
9	2.14
10	2.10
15	1.99
20	1.93
30	1.86
40	1.83
50	1.81
100	1.76
∞	1.64

n= number of test results in the group

Moment vs. rotation values used as connection stiffness/Beam-end connector test results

Test Number	Obeserved Moment Mti kNm	Stiffness Kti kNm/radian	Results from curves for stiffness calculation	
1	4.88	177.14	MRd	ϴki
2	5.47	148.11	3.85	0.026
3	5.74	132.79	3.85	0.029

$$S = \sqrt{\frac{1}{(1-n)}\sum_{i=1}^{n}(R_n - R_m)}$$

M_m(mean)= 5.364 kNm

S(Stdev)= 0.358 kNm

Material factor γ_m	Ultimate limit state	Serviceability limit state
Resistance of class1, 2 or 3 cross-section	1.0	1.0
Resistance of upright and class 4 cross-sections	1.1	1.0
Resistance of member to buckling	1.1	1.0
Resistance of connections	1.25	1.0
Resistance of connections subject testing and quality control (e.g beam end connectors	1.1	1.0

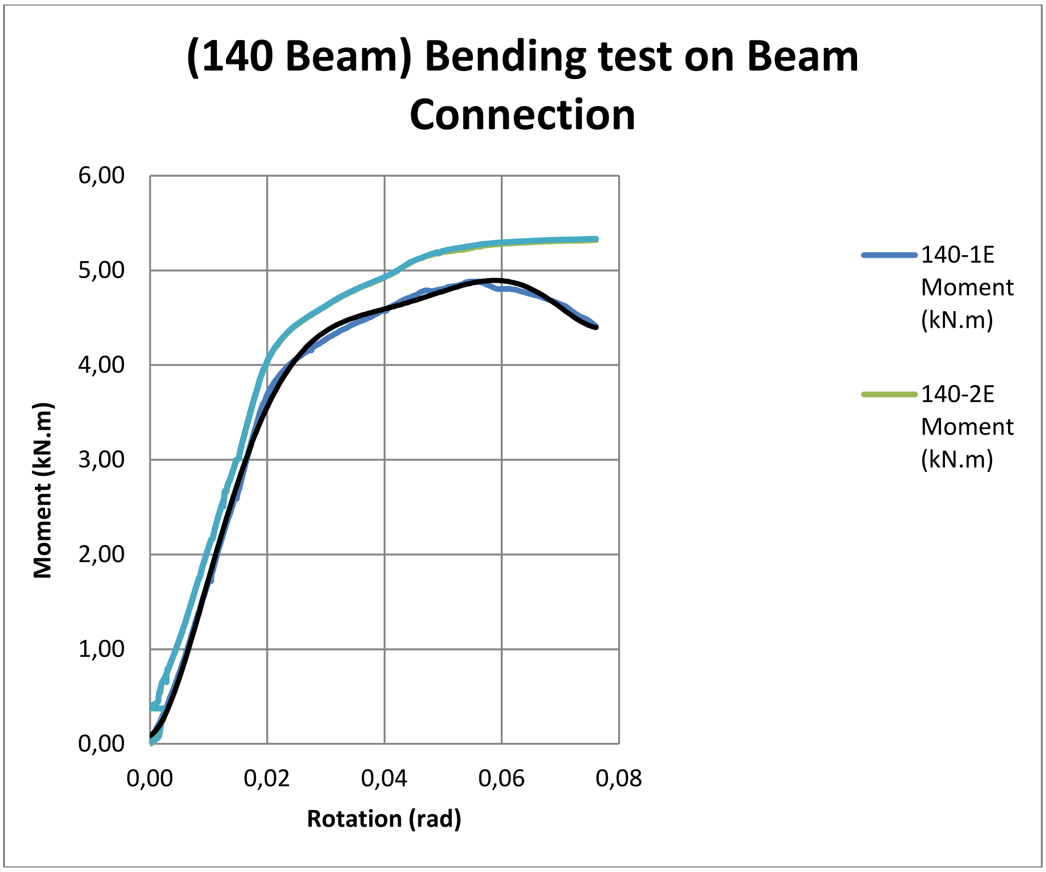
The design moment for the connection M_{Rd} =3.85 kNm

The characteristic failure moment Mk is $M_K = M_m - K_{s.s}$ = 4.24kNm

when γ_m = 1.1

K_s= 3.15 kNm/radian

K_m=152.68 kNm/radian



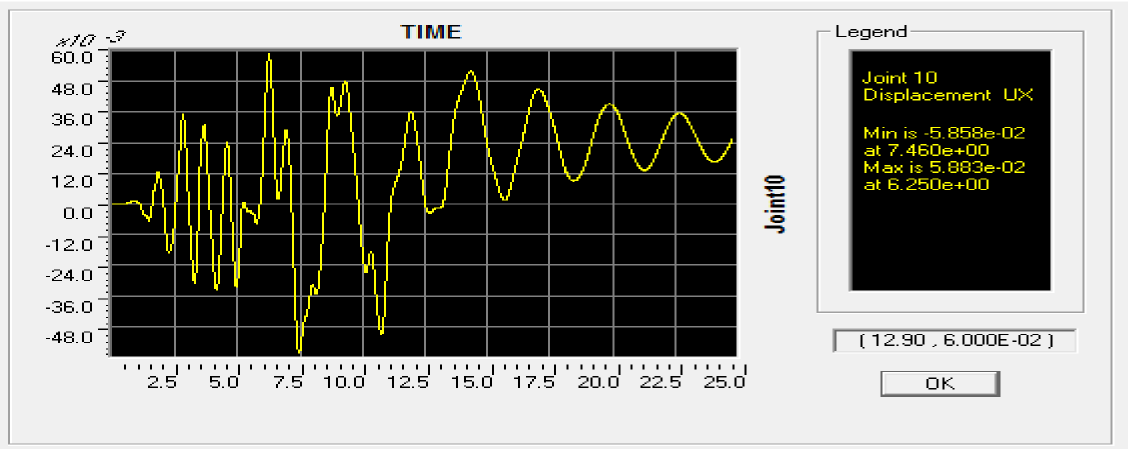
From the Moment-Rotation curve for a beam and connector we can get the rotation Ѳki at the moment M_{Rd} and the insert it in the following equation to get the connector stiffness Kti while $Ѳ_{ki}$ = 0.025radians = 177.144kNm, K_m = 152.68 kNm/rad

$$M_{RD} = \frac{M_K}{\gamma_m} = 177.144\text{kNm}$$

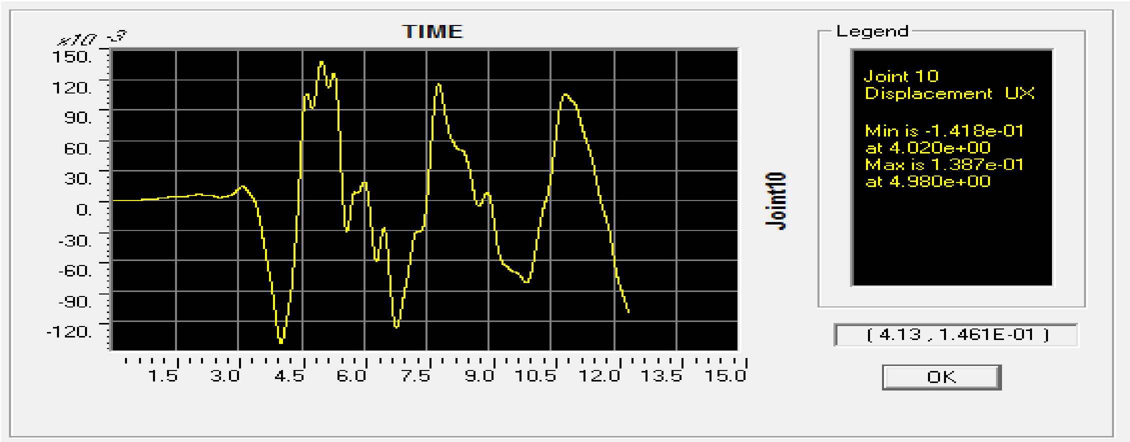
Km = 152.68kNm/rad

APPENDIX C: MAXIMUM DISPLACEMENT TIME HISTORIES

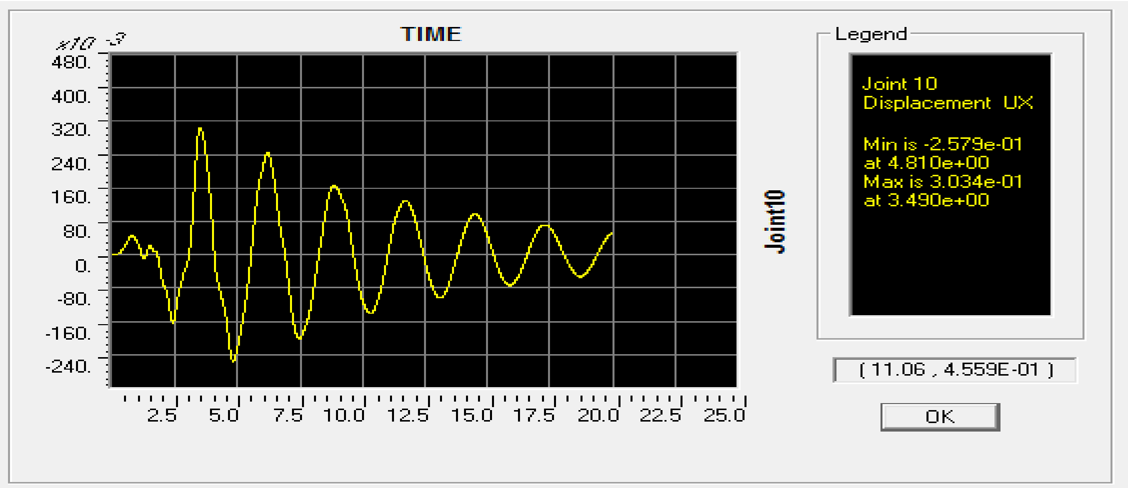
FRAME 1



Kobe Top Displacement Time Histories

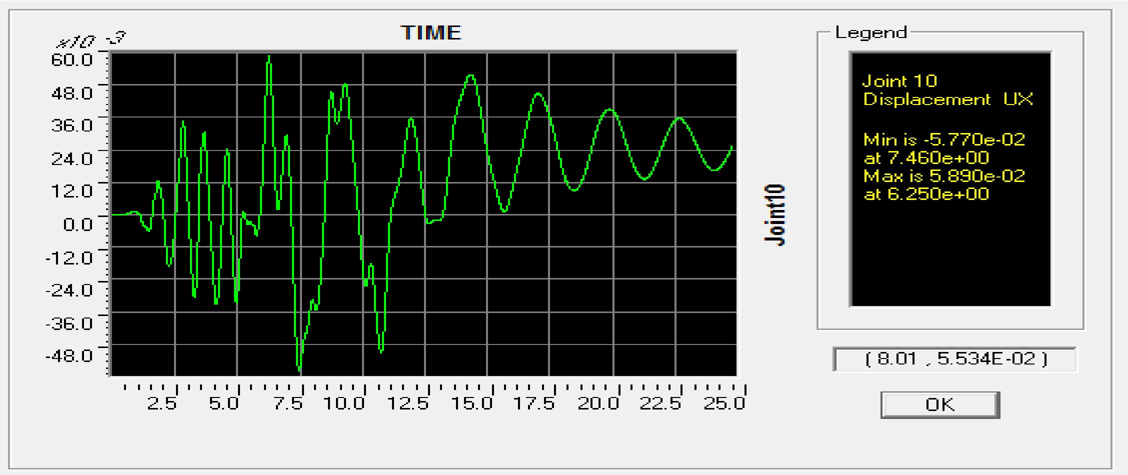


Loma Prieta Top Displacement Time Histories

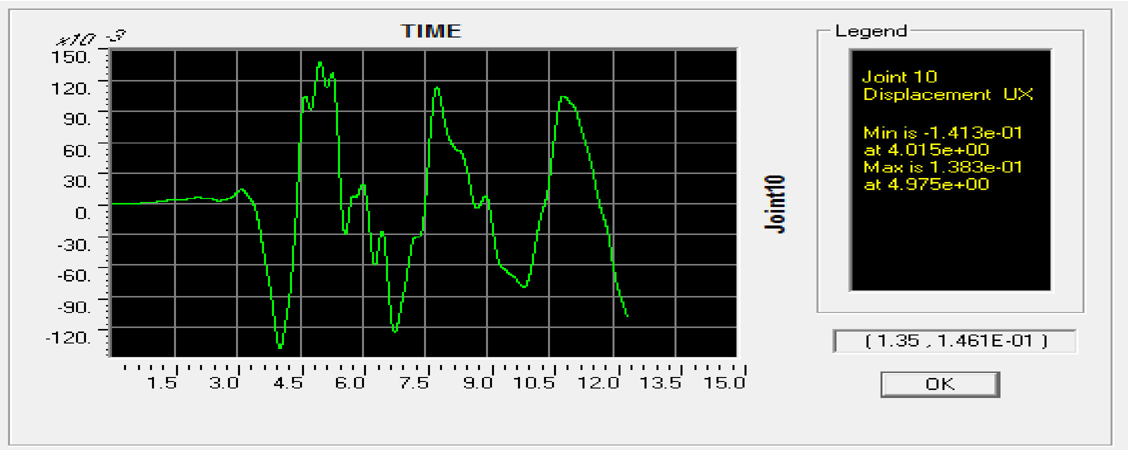


Sakarya Top Displacement Time Histories

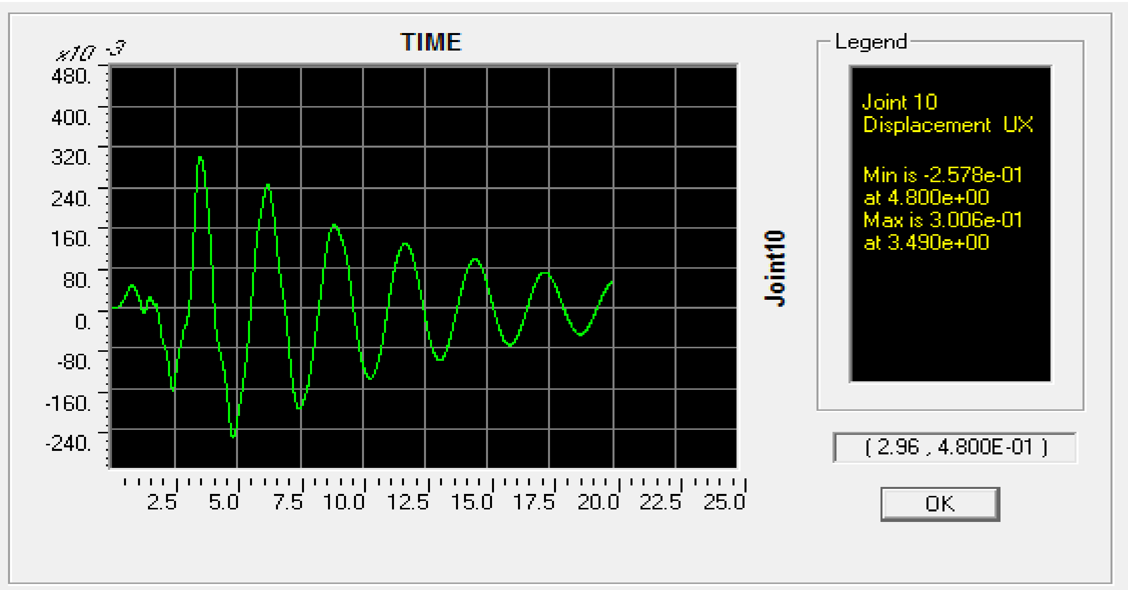
FRAME 2



Kobe Top Displacement Time Histories

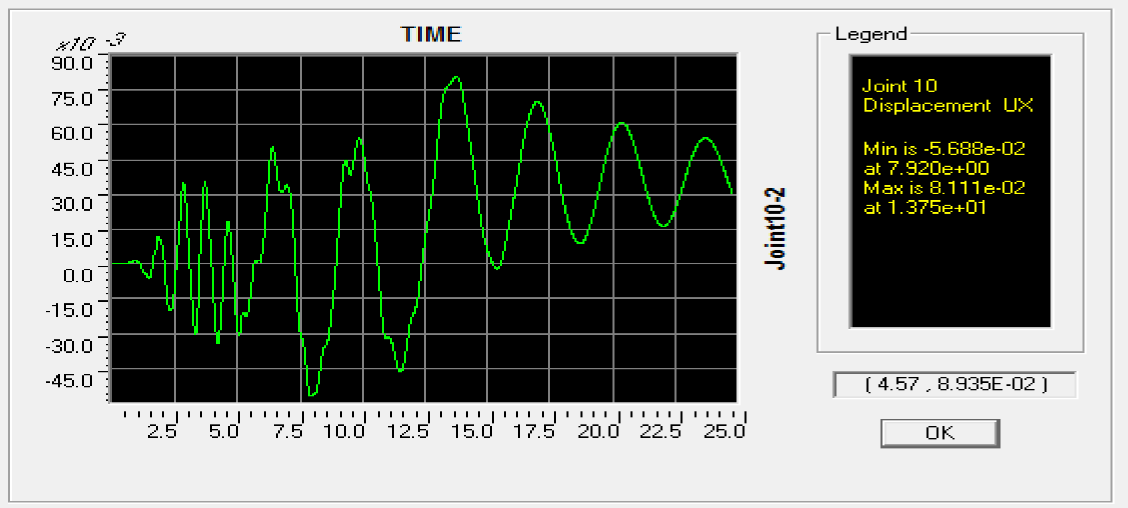


Loma Prieta Top Displacement Time Histories

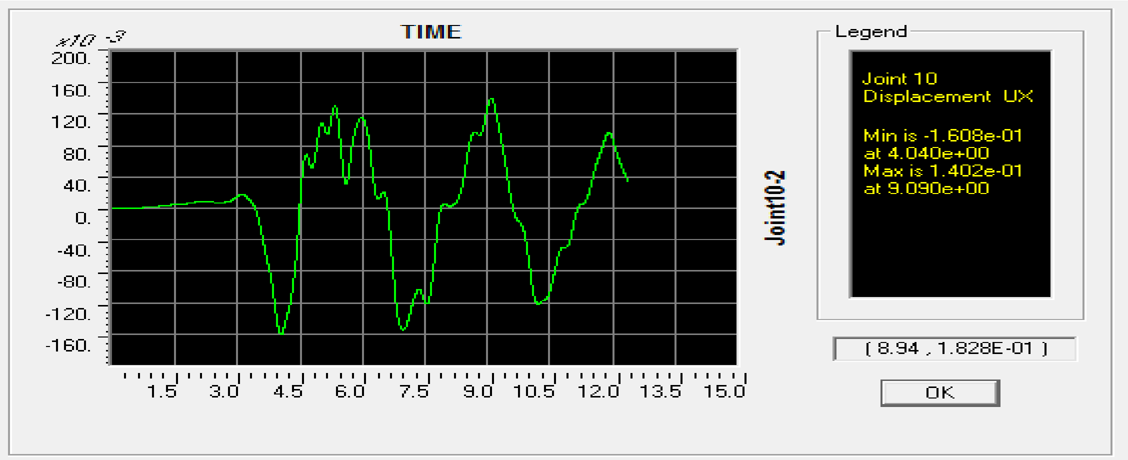


Sakarya Top Displacement Time Histories

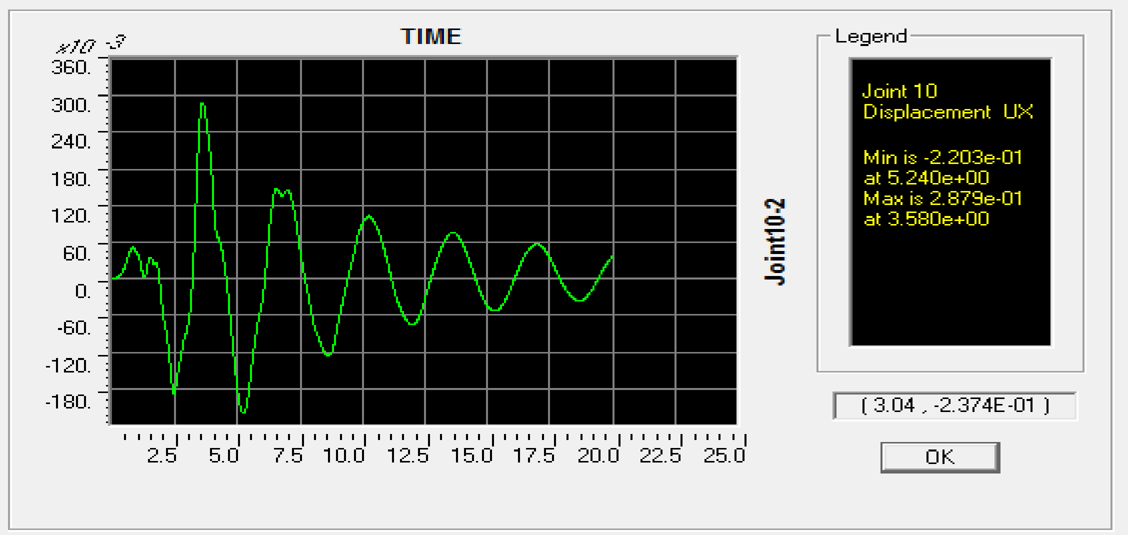
FRAME 3



Kobe Top Displacement Time Histories

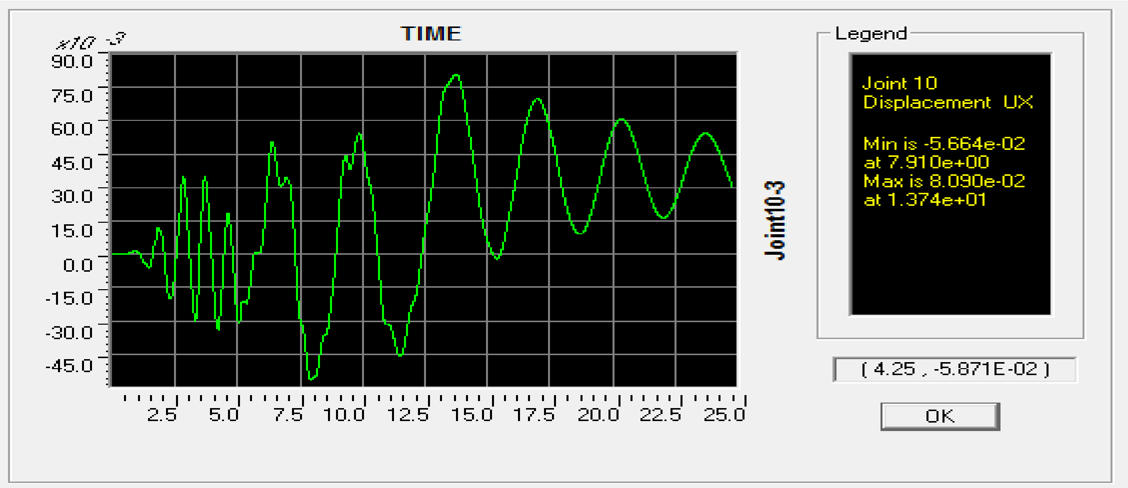


Loma Prieta Top Displacement Time Histories

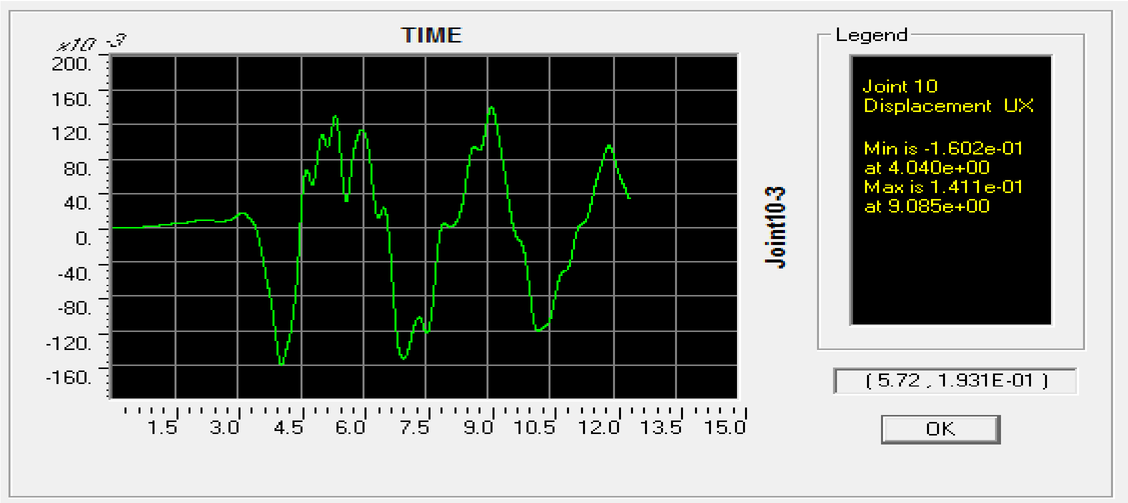


Sakarya Top Displacement Time Histories

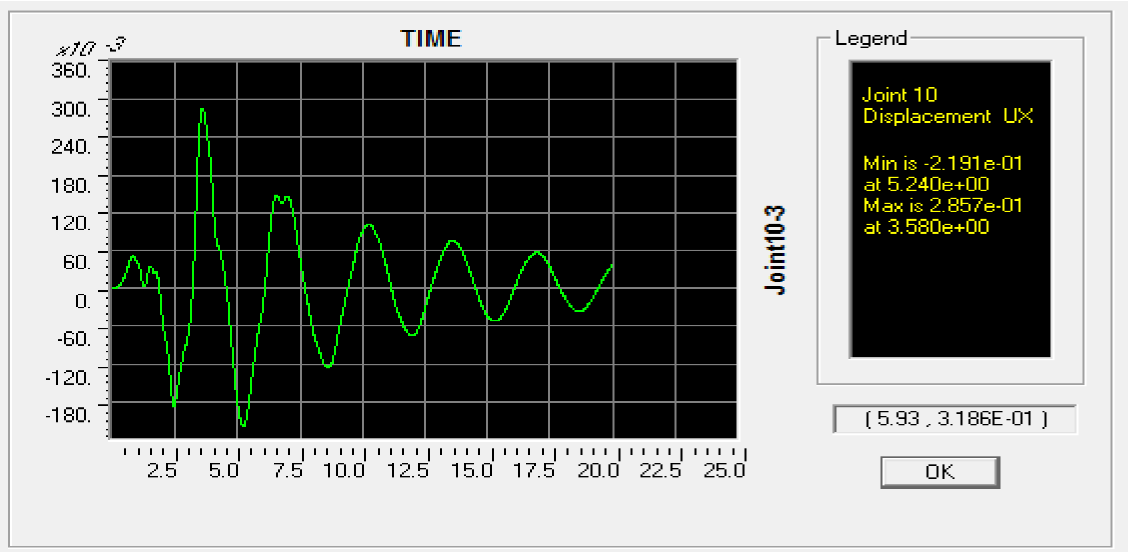
FRAME 4



Kobe Top Displacement Time Histories

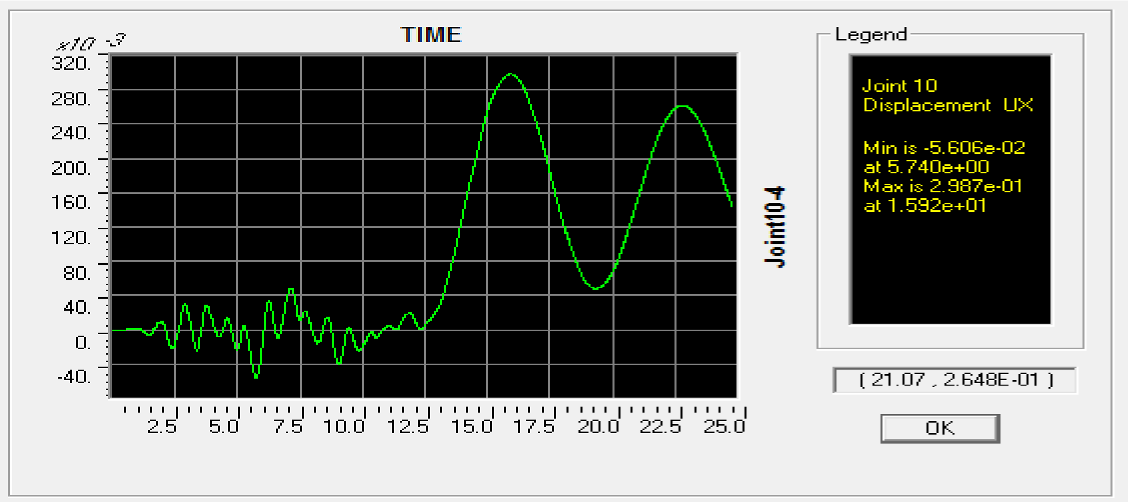


Loma Prieta Top Displacement Time Histories

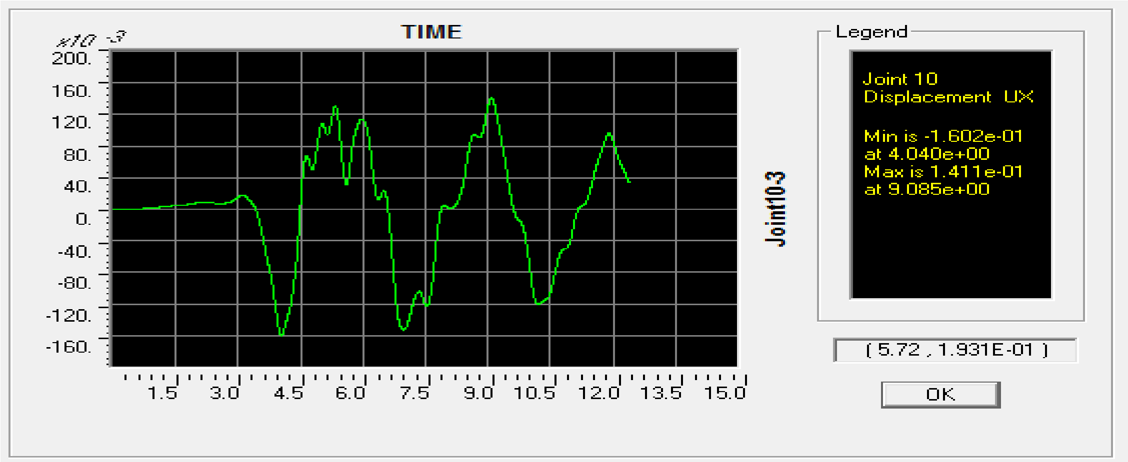


Sakarya Top Displacement Time Histories

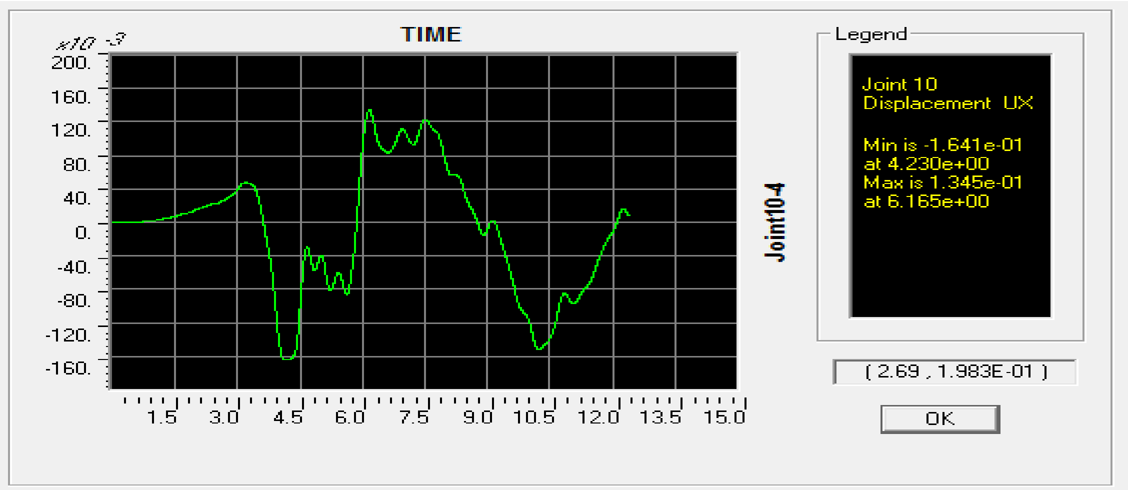
FRAME 5



Kobe Top Displacement Time Histories

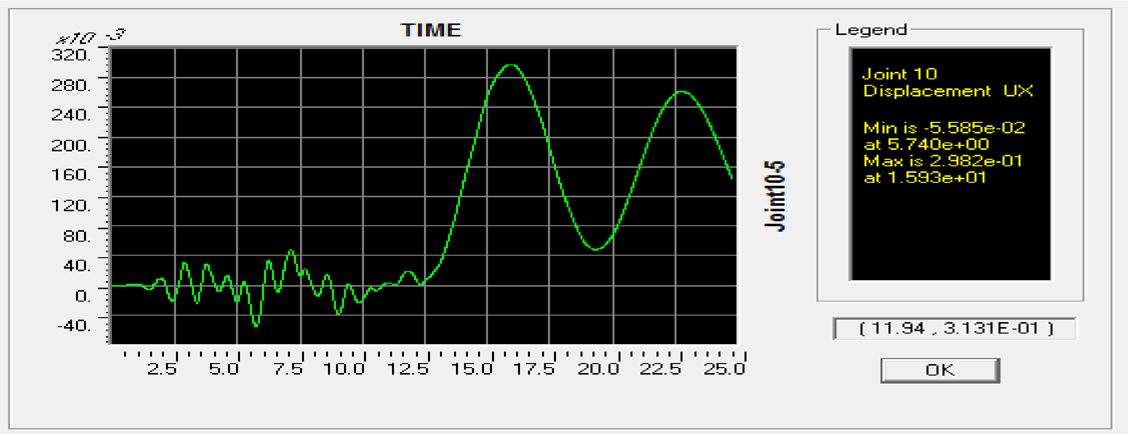


Loma Prieta Top Displacement Time Histories

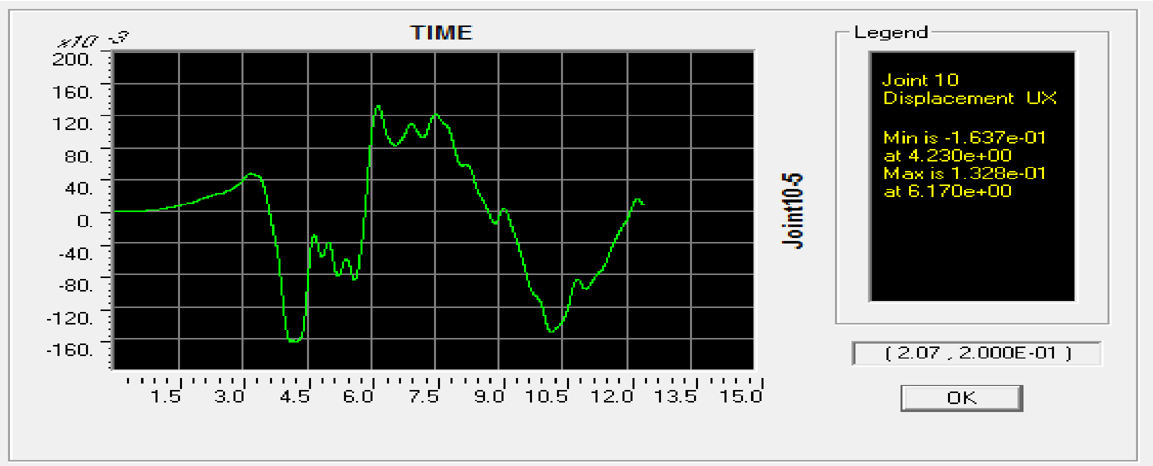


Sakarya Top Displacement Time Histories

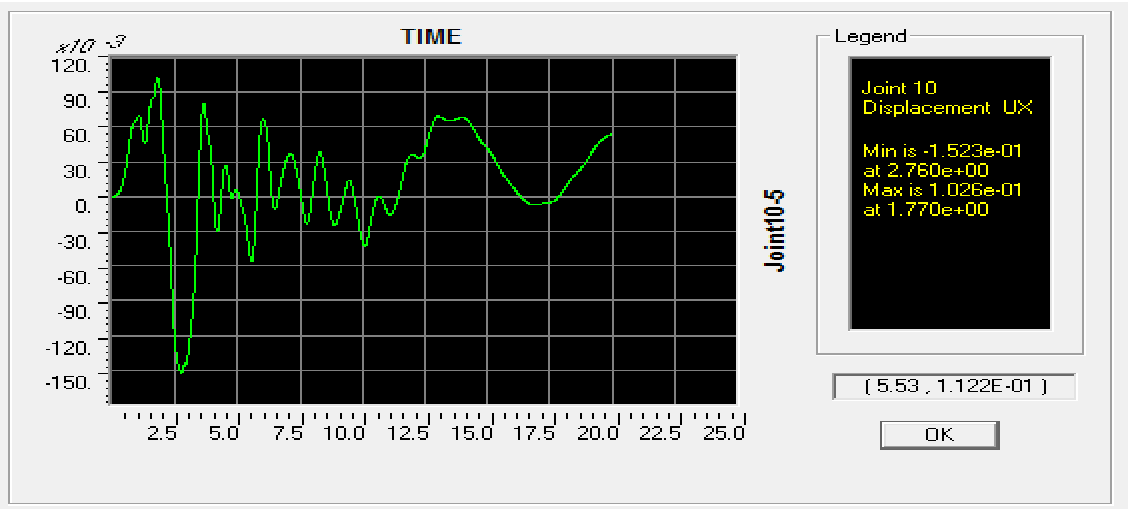
FRAME 6



Loma Prieta Top Displacement Time Histories



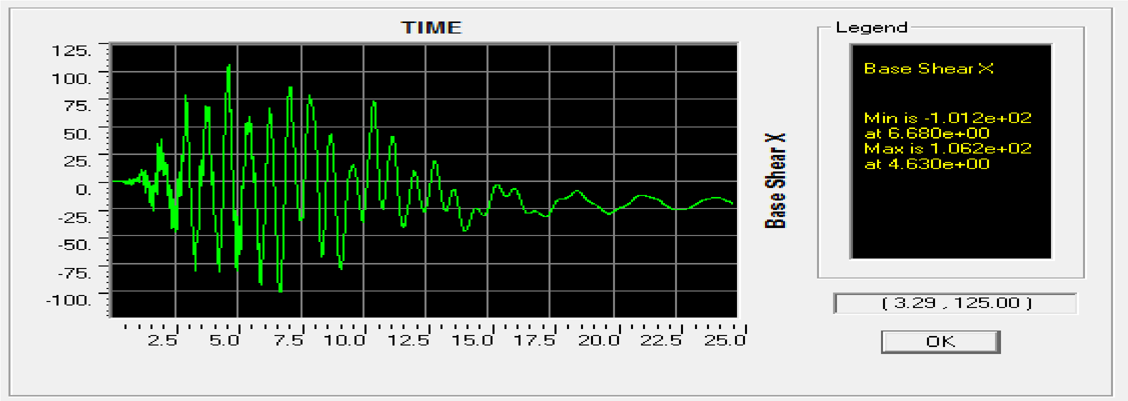
Loma Prieta Top Displacement Time Histories



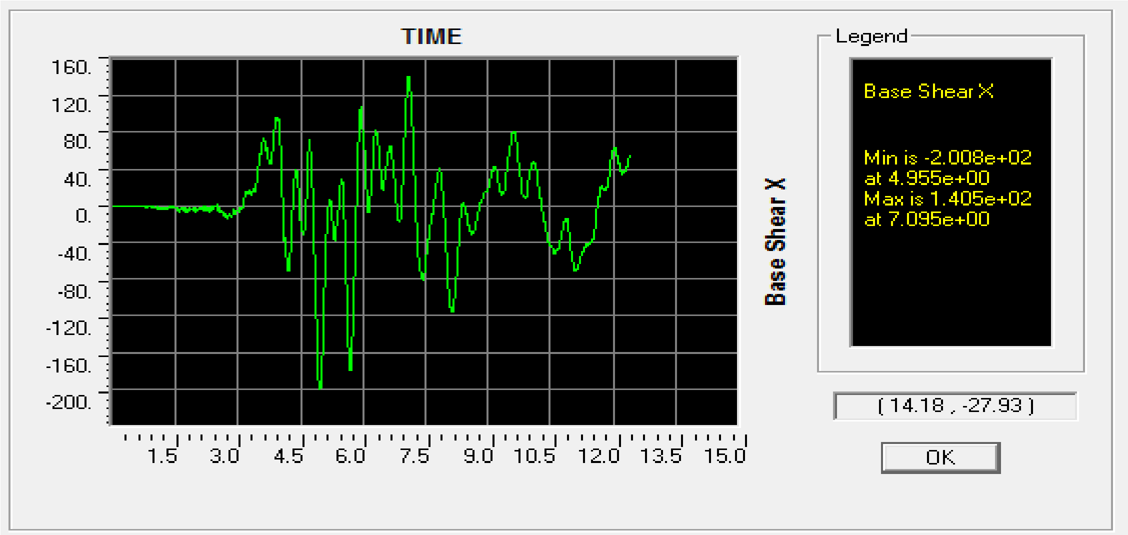
Sakarya Top Displacement Time Histories

APPENDIX D: MAXIMUM BASE SHEAR TIME HISTORIES RESPONSE

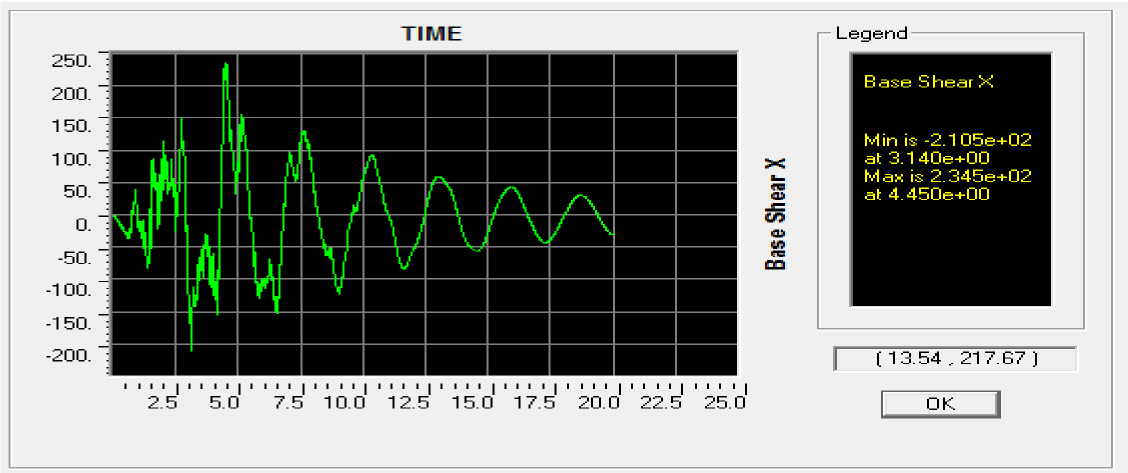
FRAME 1



Kobe Base shear Time Histories

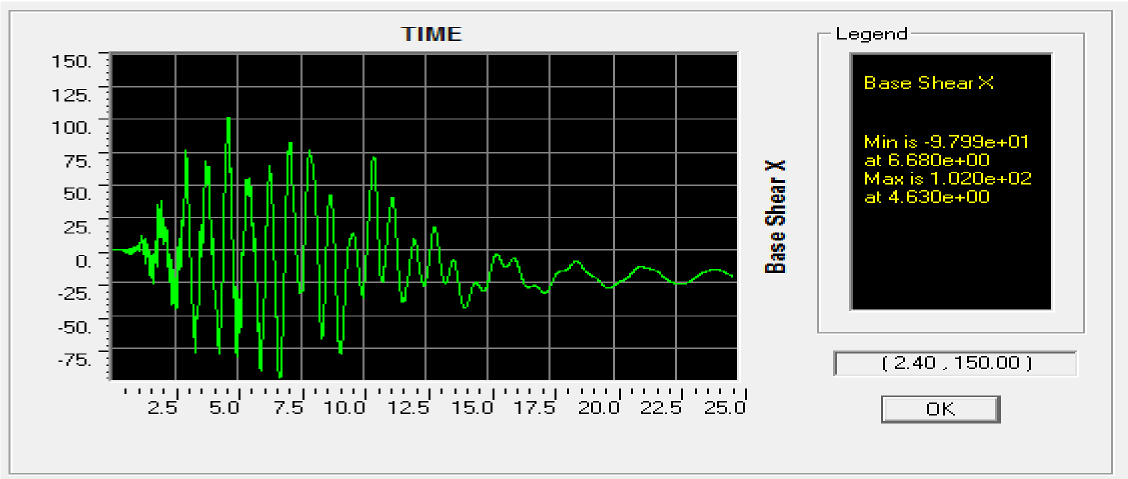


Loma Prieta Base Shear Time Histories

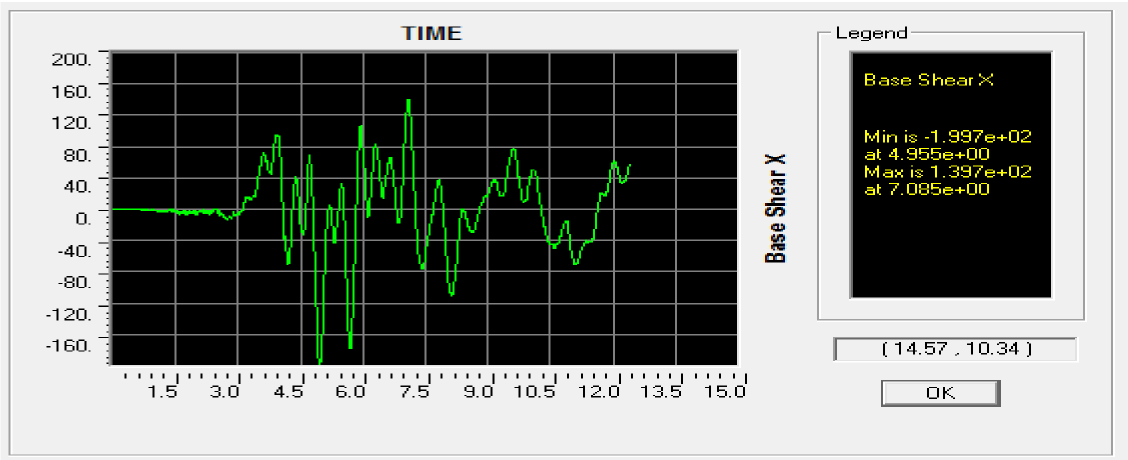


SakaryaTime Histories

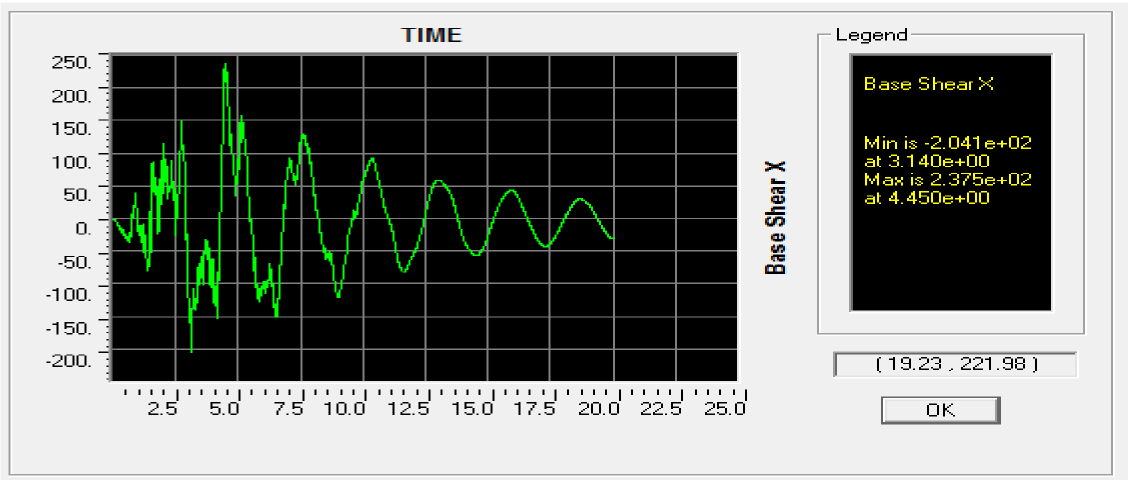
FRAME 2



Kobe Base Shear Time Histories

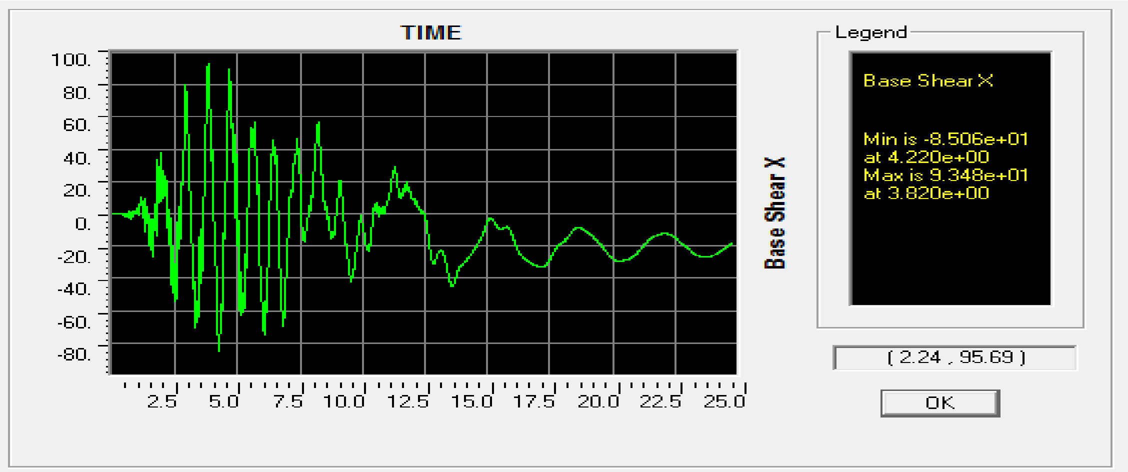


Loma Prieta Base Shear Time Histories

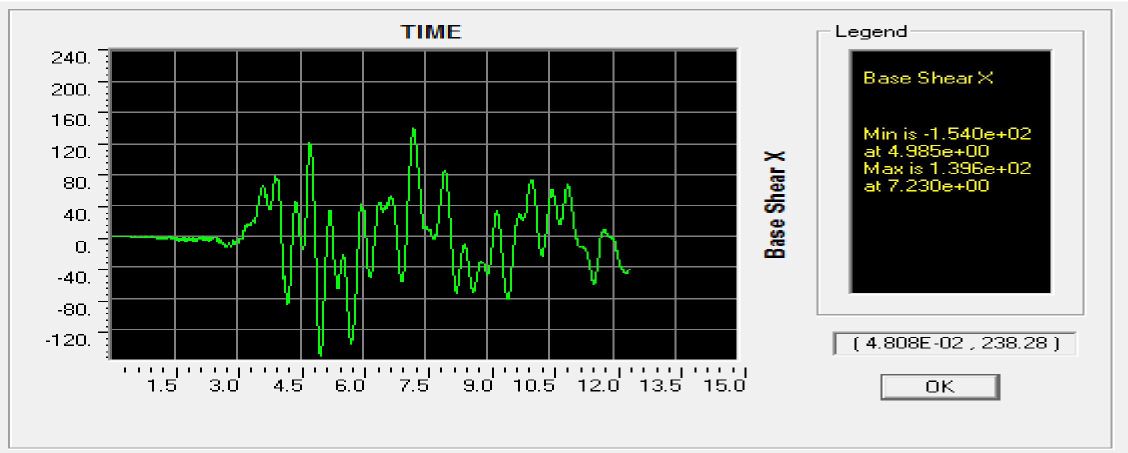


Sakarya Base Shear Time Histories

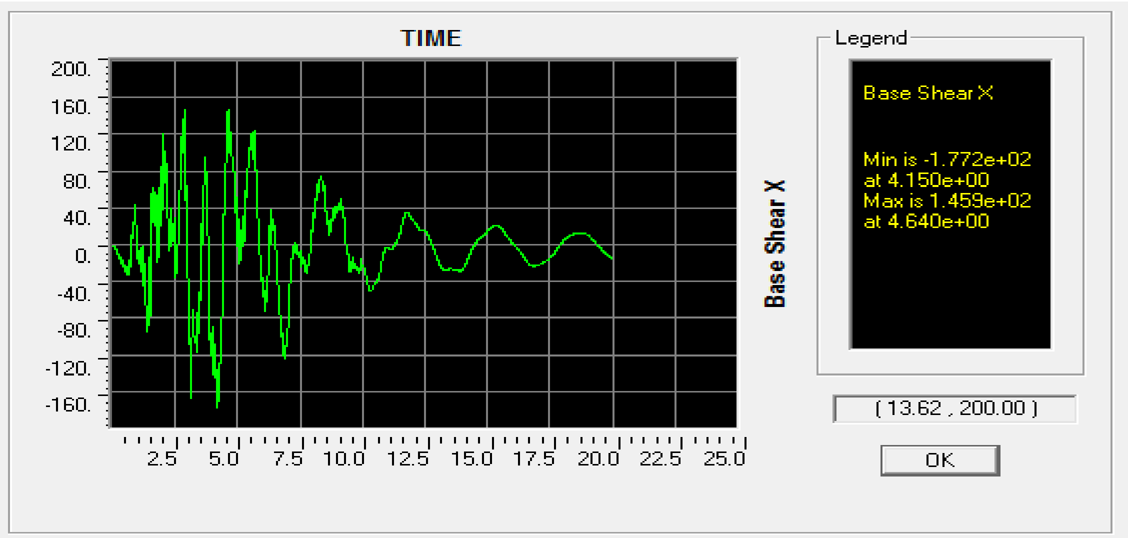
FRAME 3



Base Shear Time Histories

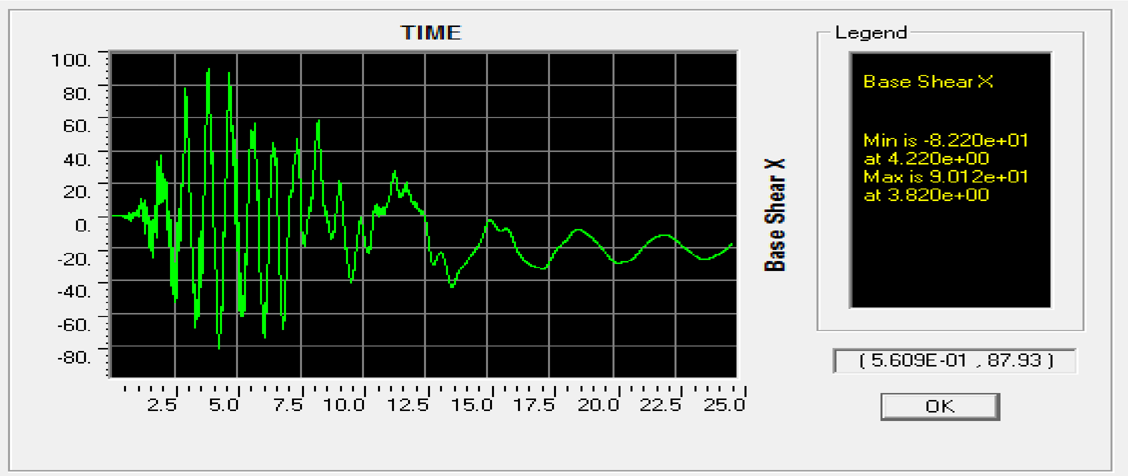


Base Shear Time Histories

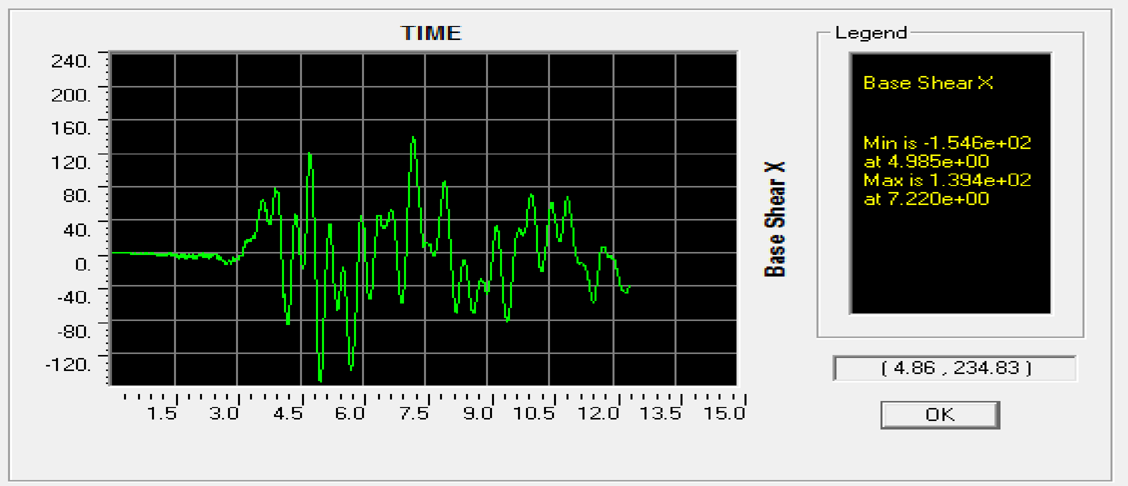


Base Shear Time Histories

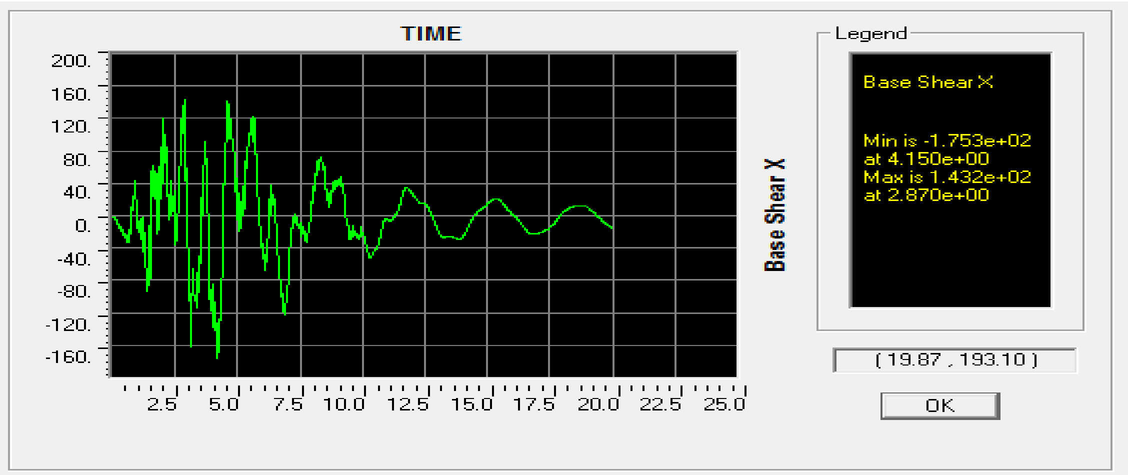
FRAME 4



Kobe Base shear Time Histories

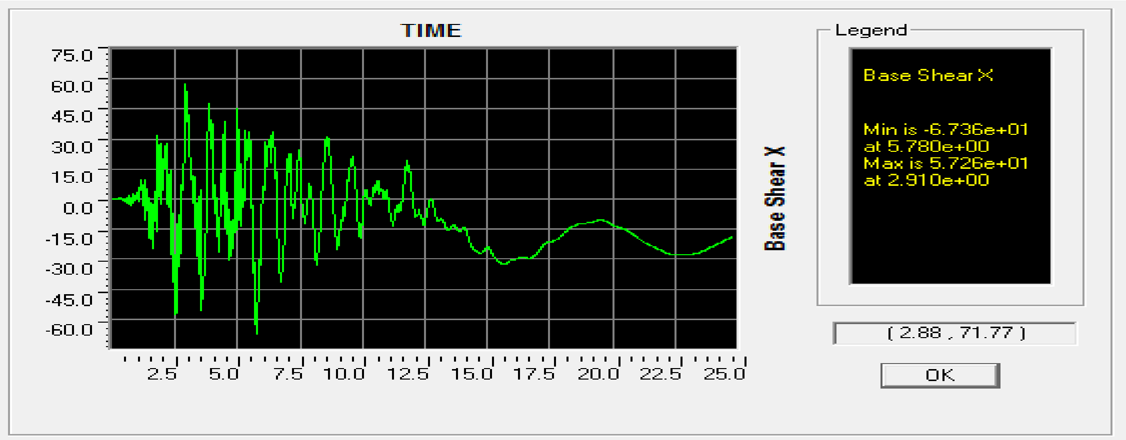


Loma Prieta Base Shear Time Histories

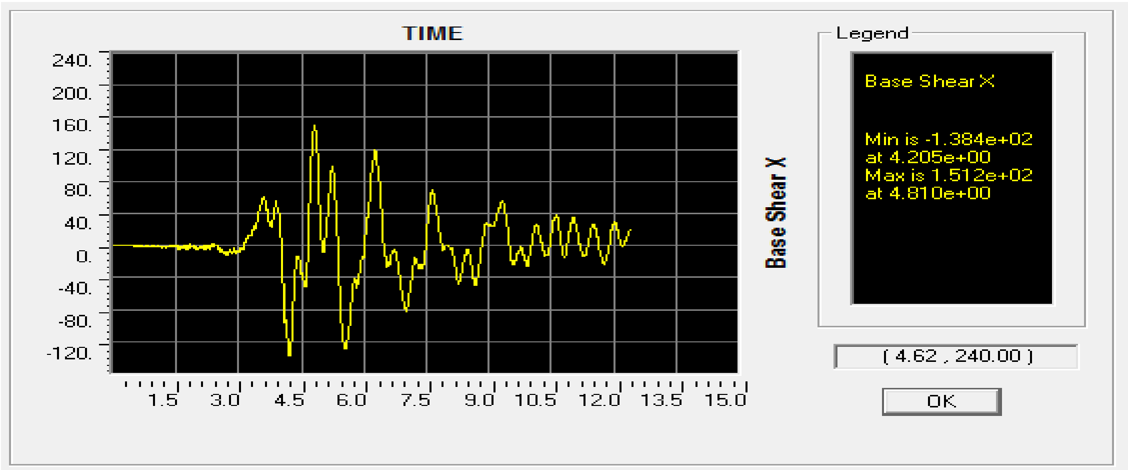


Sakarya Base Shear Time Histories

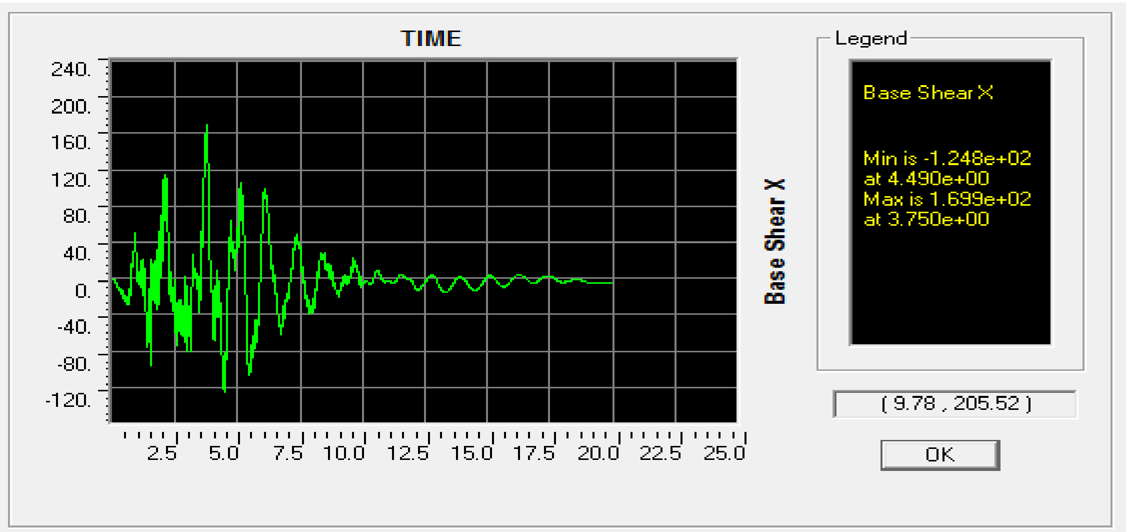
FRAME 5



Kobe Base Shear Time Histories

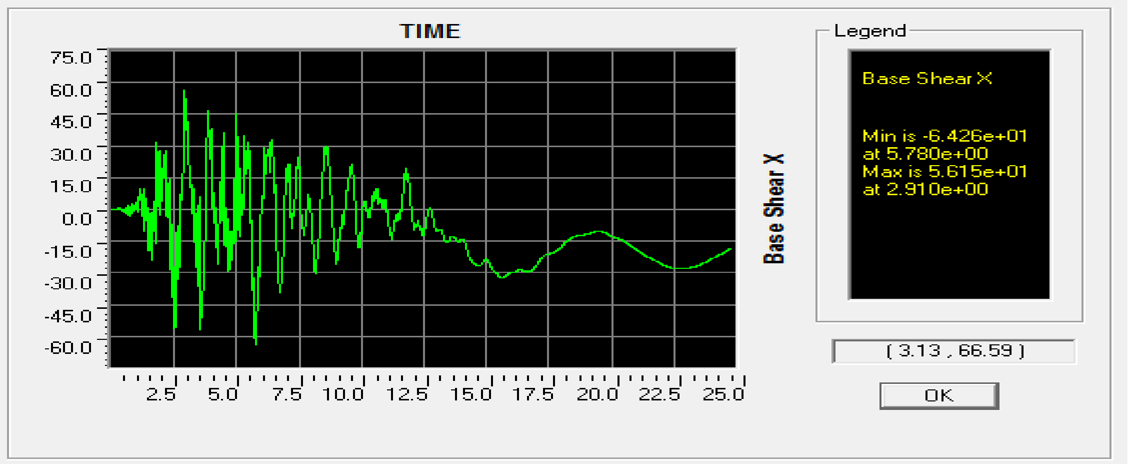


Base Shear Loma Prieta Time Histories

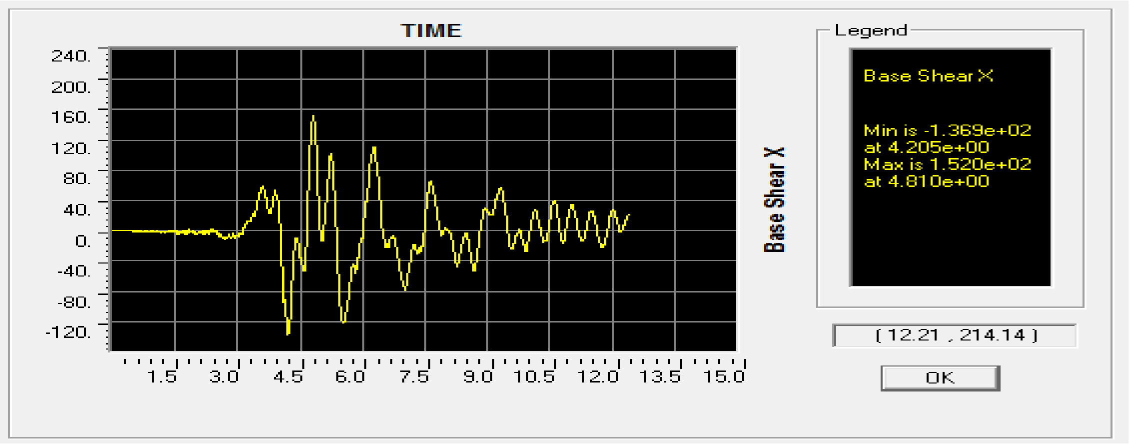


Sakarya Base Shear Time Histories

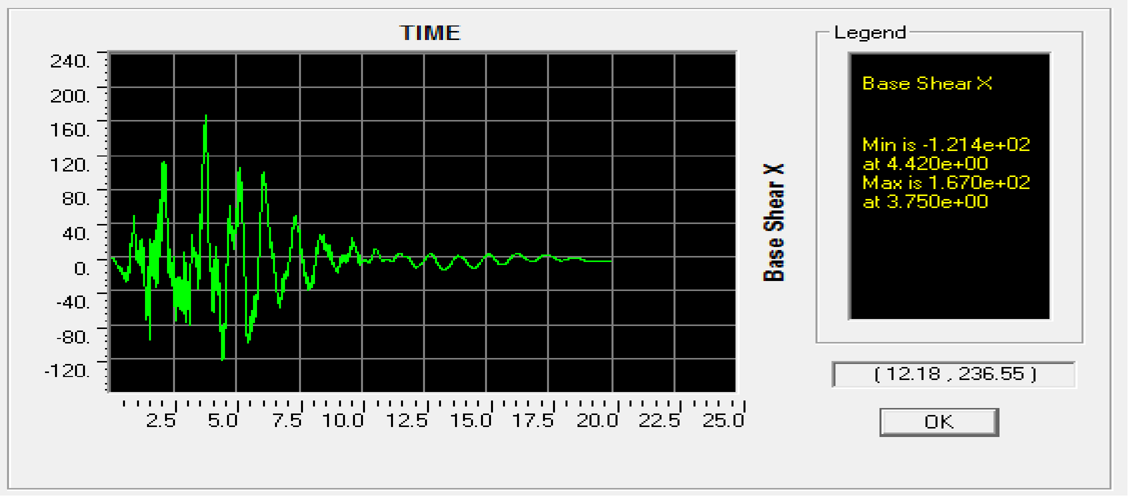
FRAME 6



Kobe Base Shear Time Histories



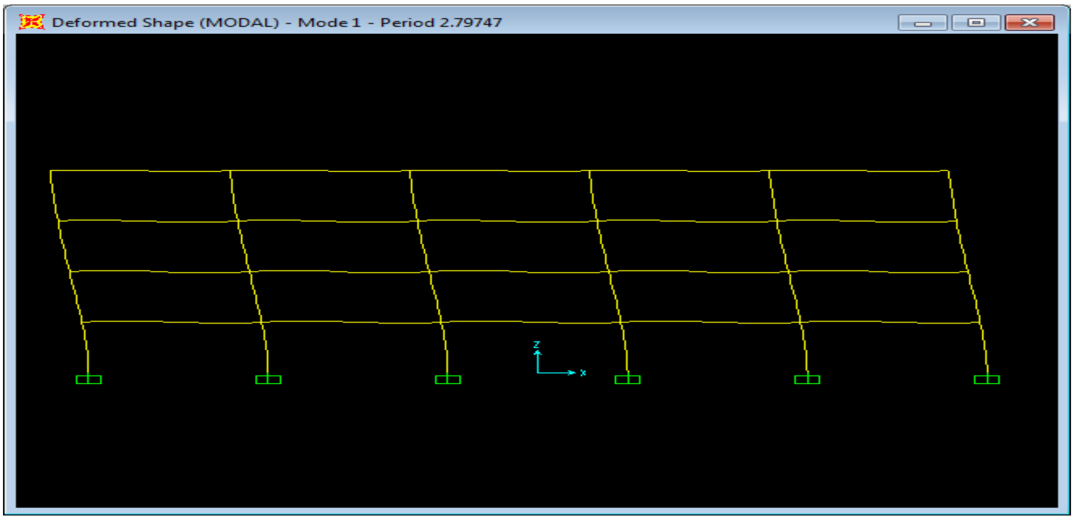
Loma Prieta Base Shear Time Histories

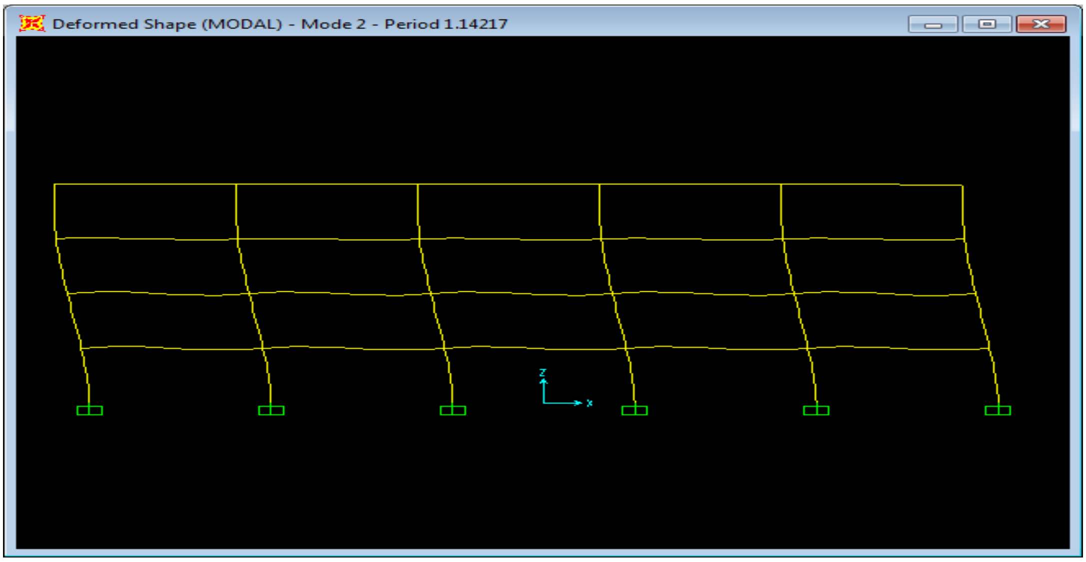


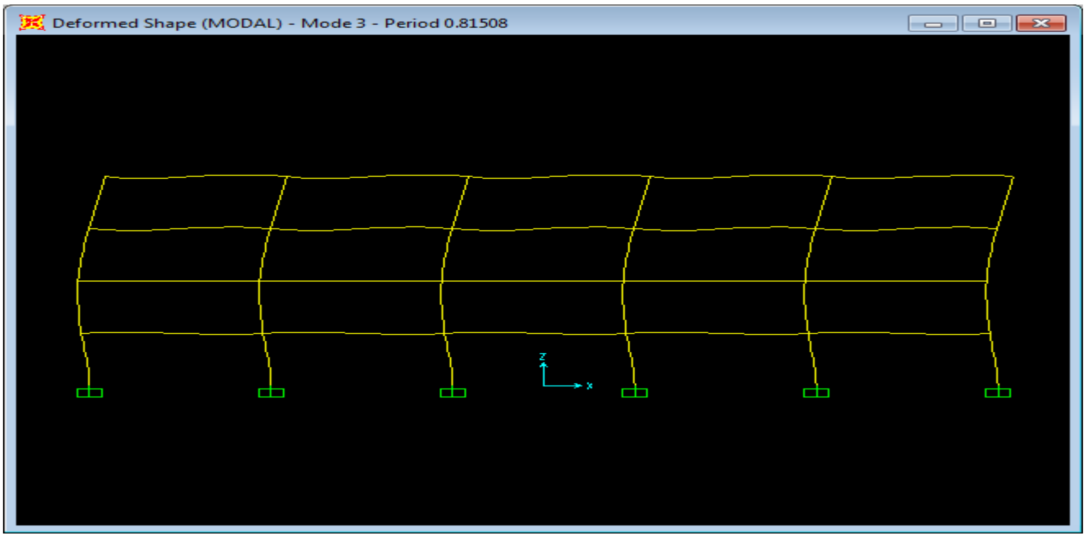
Sakarya Base Shear Time Histories

APPENDIX E: MODE SHAPES AND NATURAL PERIODS

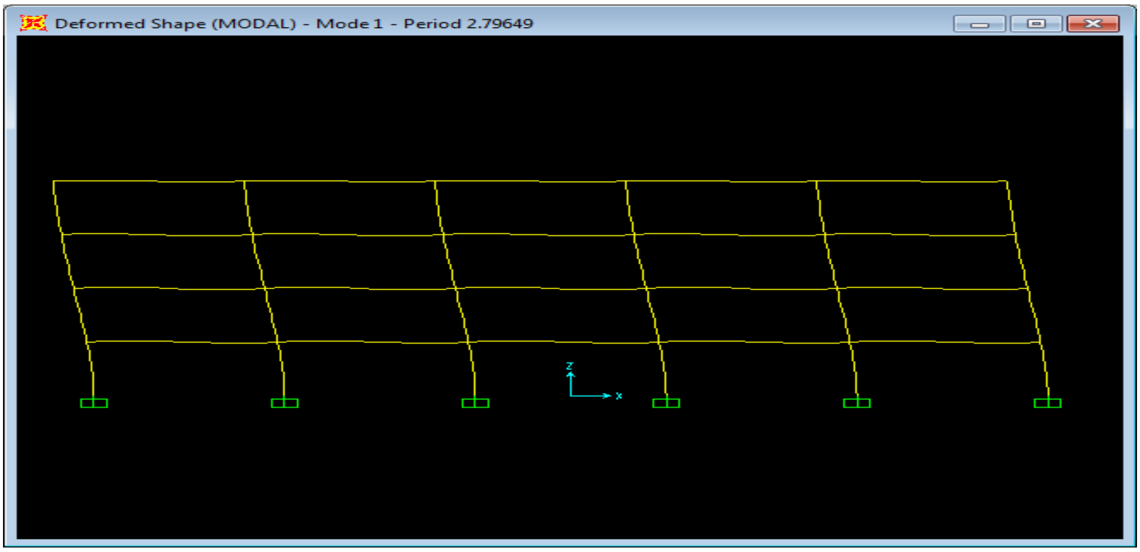
FRAME 1

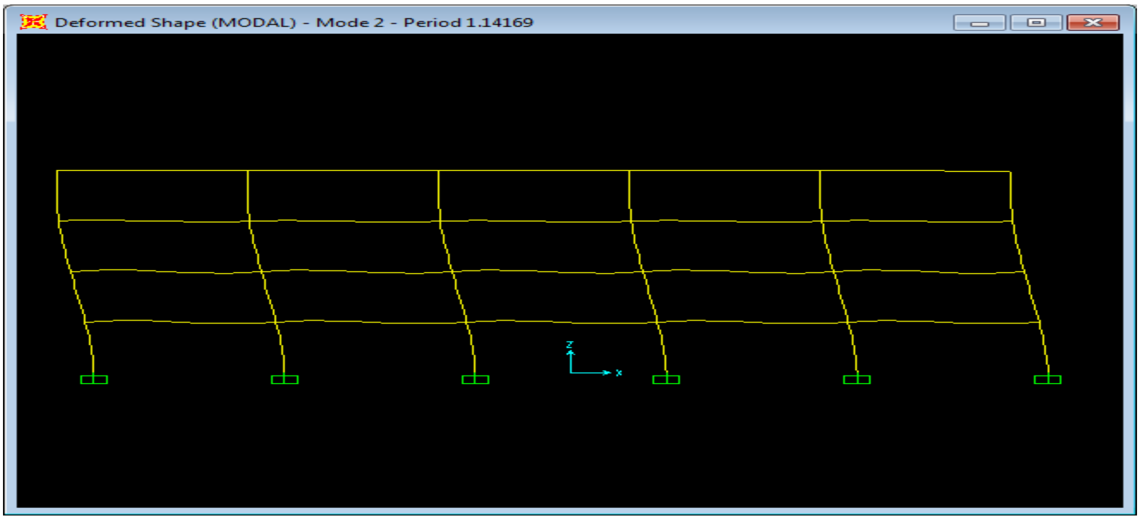


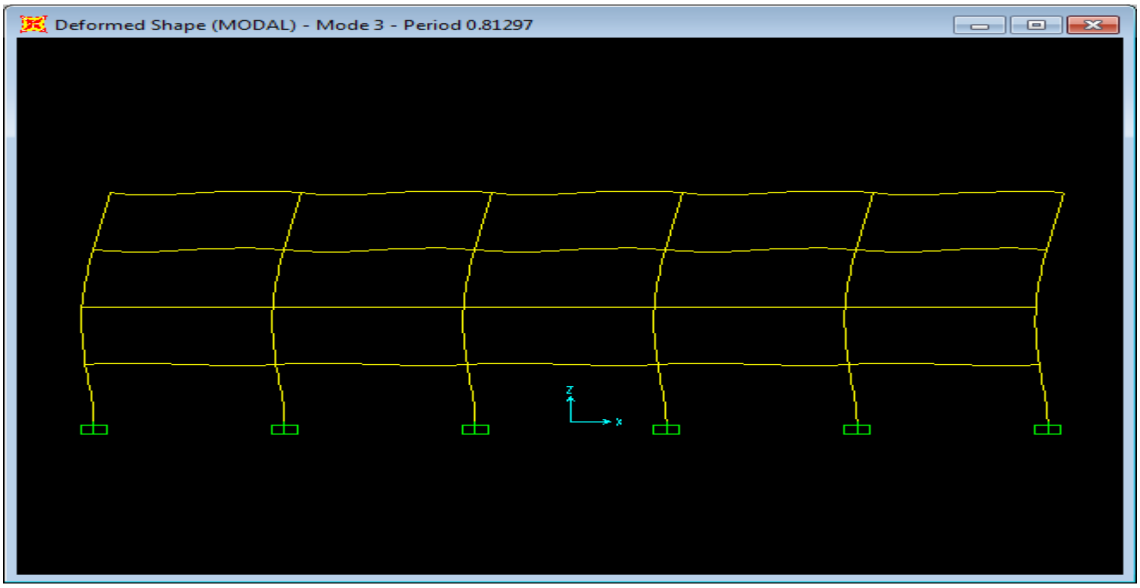




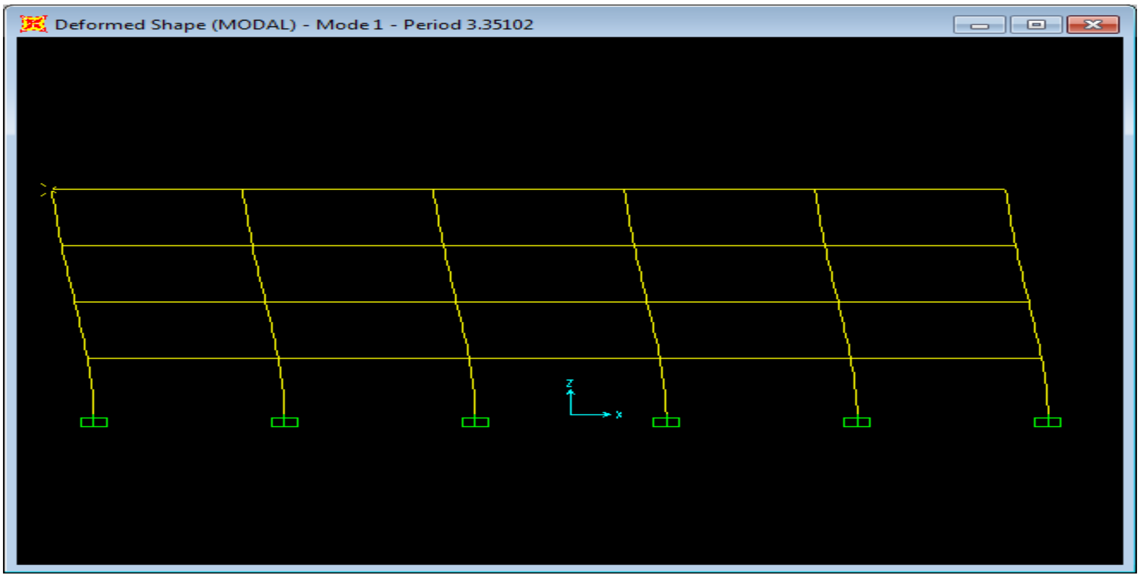
FRAME 2

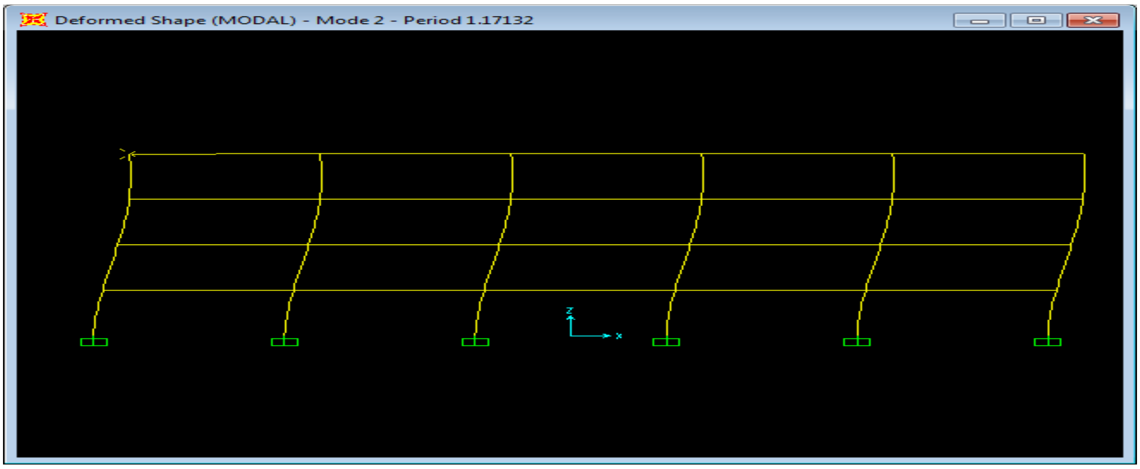

Deformed Shape (MODAL) - Mode 1 - Period 2.79649
z
x

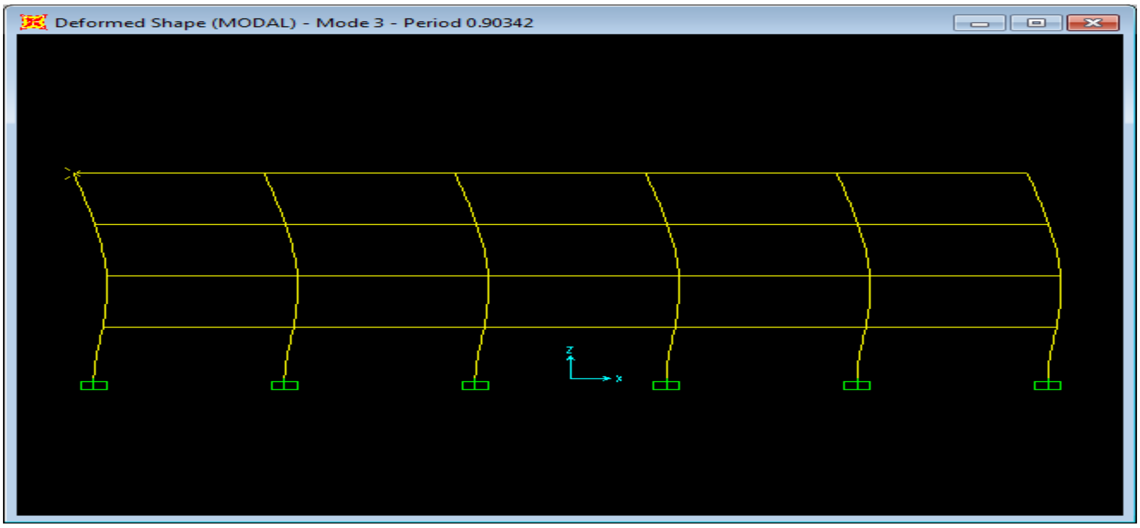

Deformed Shape (MODAL) - Mode 2 - Period 1.14169
z
x


Deformed Shape (MODAL) - Mode 3 - Period 0.81297
z
x

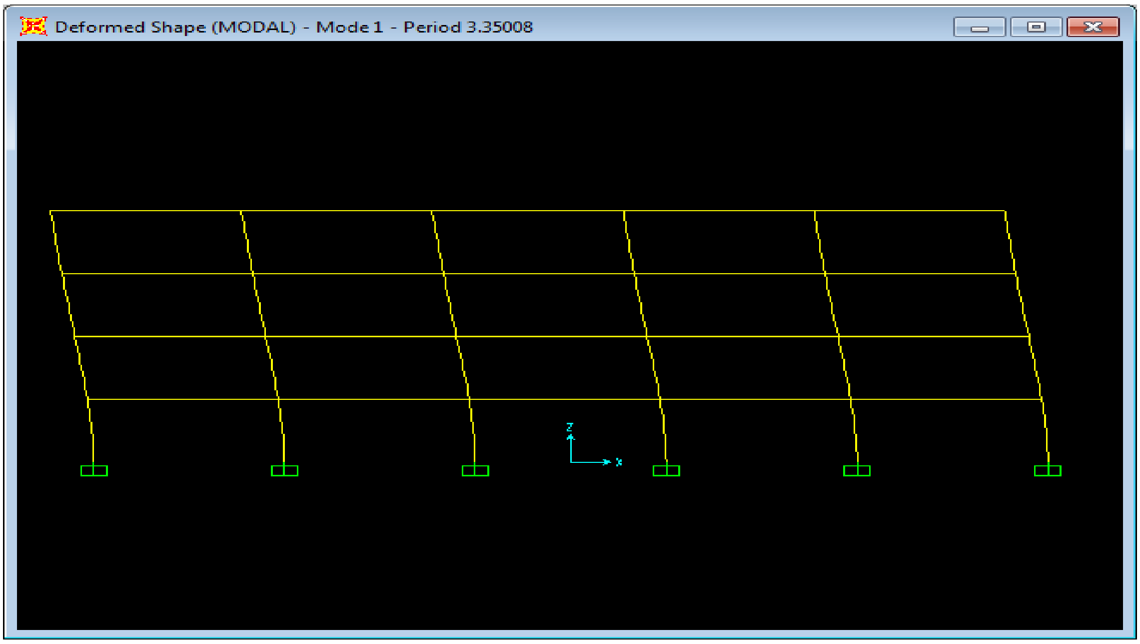
FRAME 3

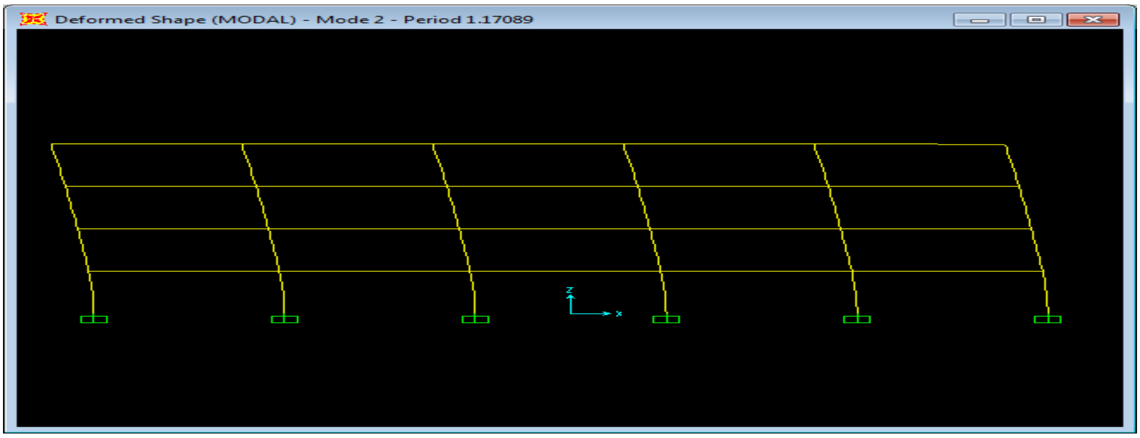

Deformed Shape (MODAL) - Mode 1 - Period 3.35102
Z
X

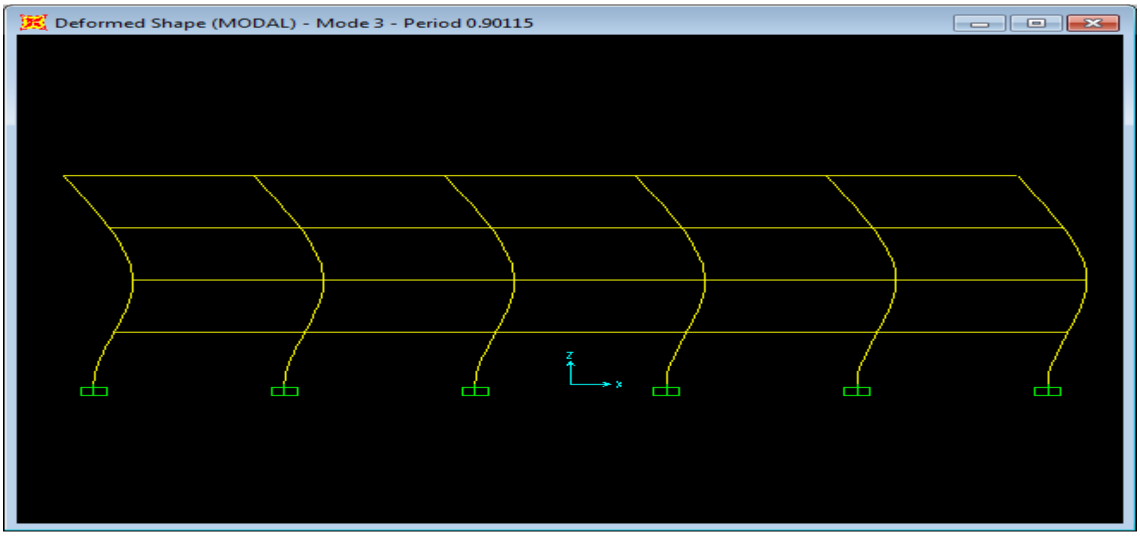

Deformed Shape (MODAL) - Mode 2 - Period 1.17132
Z
X


Deformed Shape (MODAL) - Mode 3 - Period 0.90342
Z
X

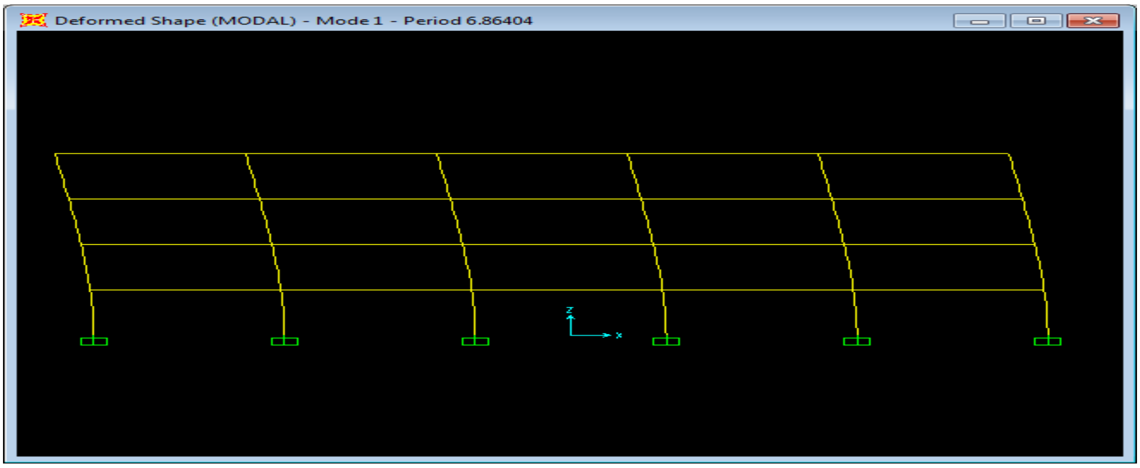
FRAME 4

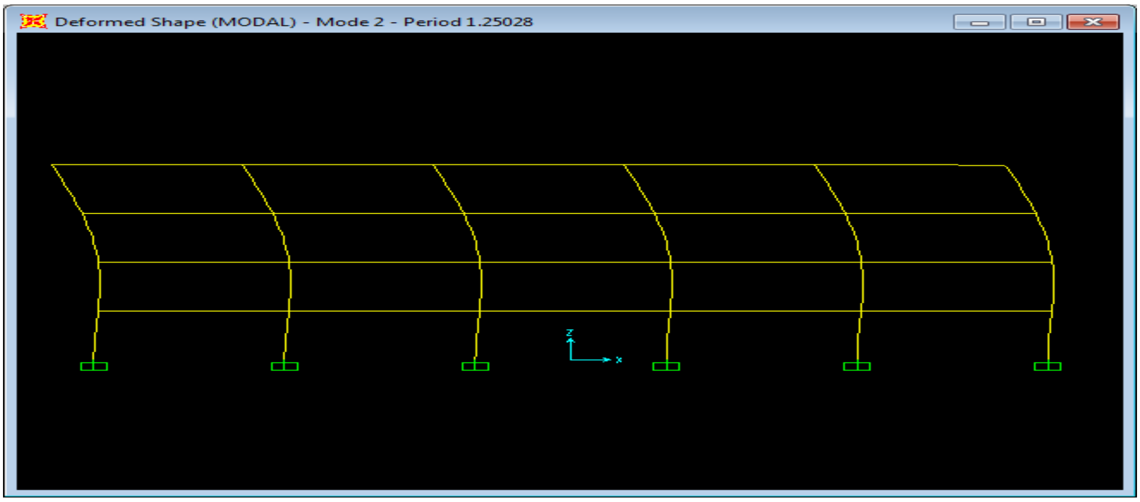

Deformed Shape (MODAL) - Mode 1 - Period 3.35008
z
x

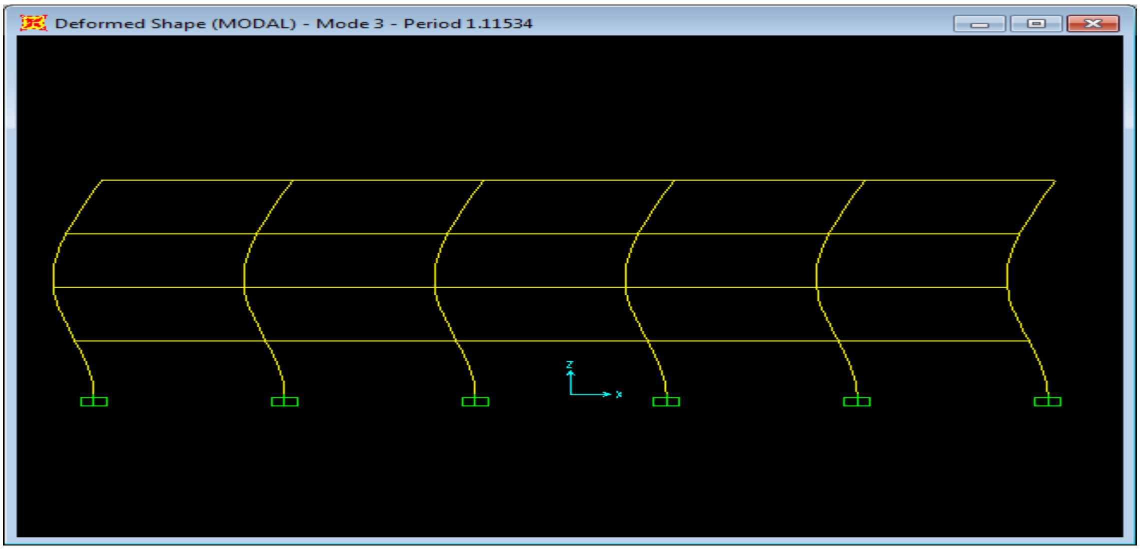

Deformed Shape (MODAL) - Mode 2 - Period 1.17089


Deformed Shape (MODAL) - Mode 3 - Period 0.90115

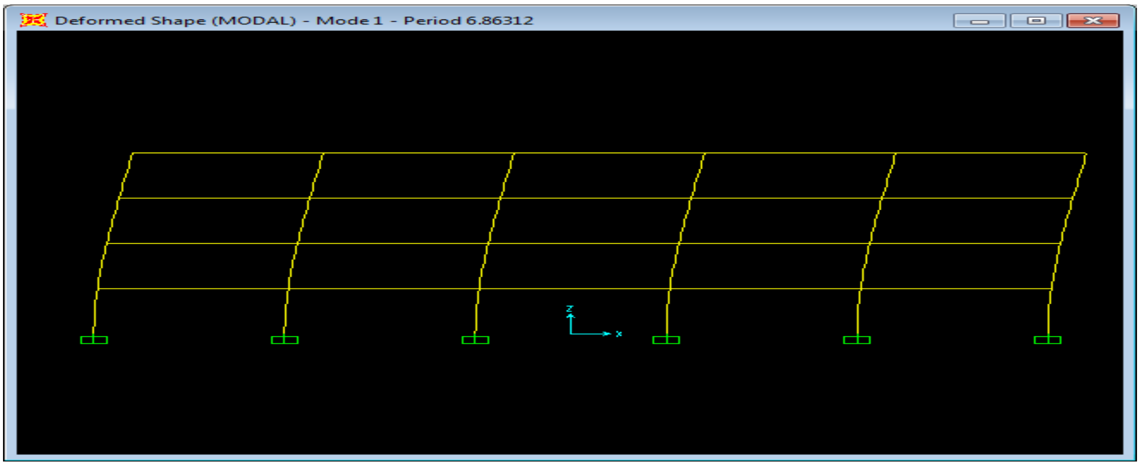
FRAME 5

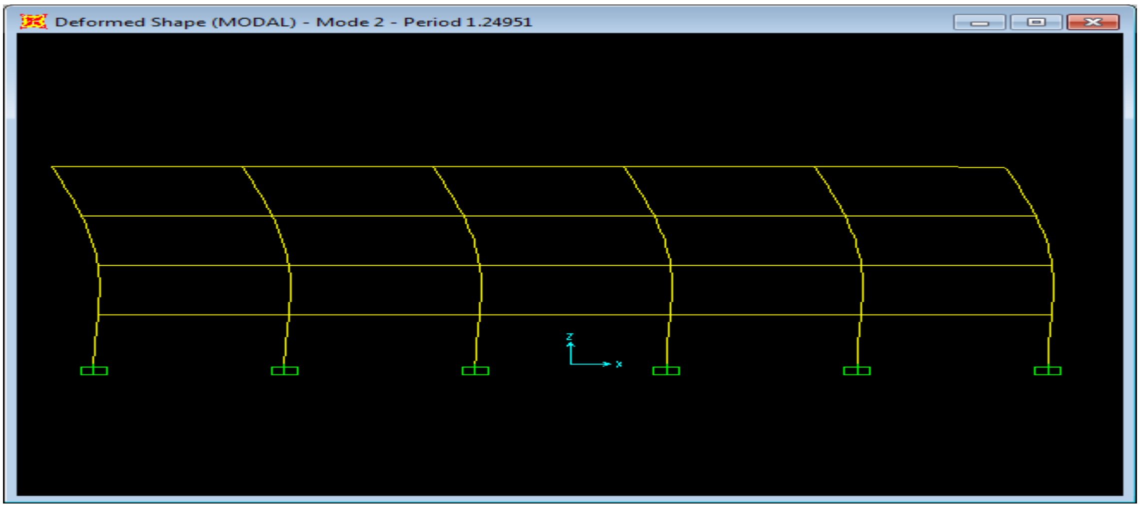

Deformed Shape (MODAL) - Mode 1 - Period 6.86404
Z
X

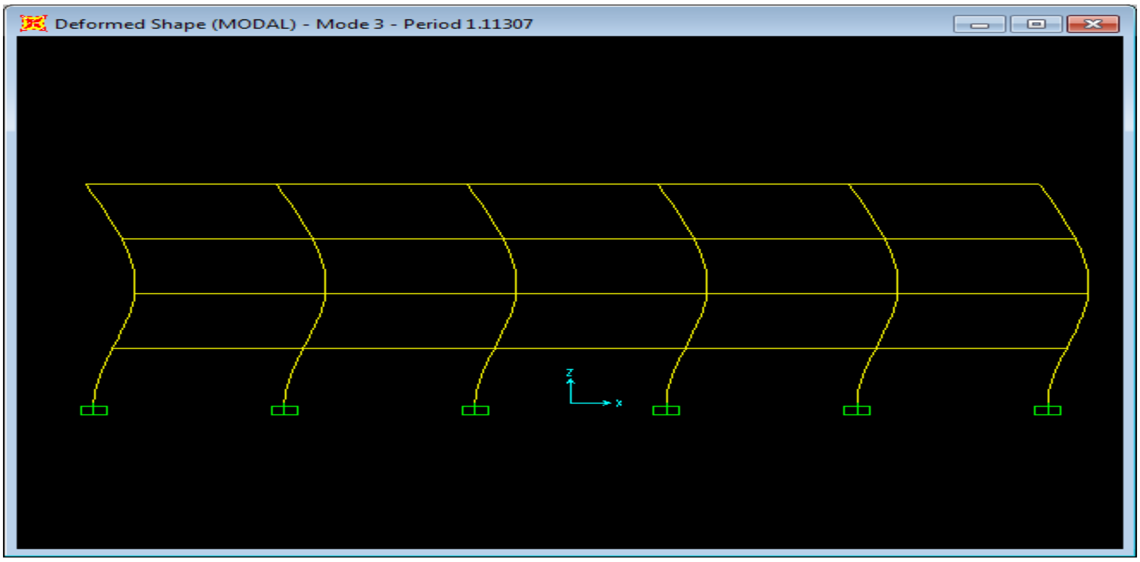

Deformed Shape (MODAL) - Mode 2 - Period 1.25028
Z
X


Deformed Shape (MODAL) - Mode 3 - Period 1.11534
Z
X

FRAME 6


Deformed Shape (MODAL) - Mode 1 - Period 6.86312
Z
X


Deformed Shape (MODAL) - Mode 2 - Period 1.24951
Z
X


Deformed Shape (MODAL) - Mode 3 - Period 1.11307
Z
X