

T.R.
GEBZE TECHNICAL UNIVERSITY
GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

**INVESTIGATION OF INDUSTRIAL
STORAGE RACK SYSTEMS
UNDER EARTHQUAKE LOADS**

MOHAMMAD JARADAT
A THESIS SUBMITTED FOR THE DEGREE OF
MASTER OF SCIENCE
DEPARTMENT OF CIVIL ENGINEERING
EARTHQUAKE AND STRUCTURAL ENGINEERING PROGRAM

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THESIS SUPERVISOR
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2020

T.C.
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GTÜ Fen Bilimleri Enstitüsü Yönetim Kurulu'nun 10/07/2020 tarih ve 2020/33 sayılı kararıyla oluşturulan jüri tarafından 20/07/2020 tarihinde tez savunma sınavı yapılan MOHAMMAD SABER SAEED JARADAT'ın tez çalışması İnşaat Mühendisliği Anabilim Dalında YÜKSEK LİSANS tezi olarak kabul edilmiştir.

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SUMMARY

Multi-story Industrial Storage Racking Systems (ISRS) constructed from thin-walled steel profiles. They are characterized with delicate fabrication and light lateral stiffness which leads to risky consequences when these systems are exposed to earthquakes. By reviewing the national regulations for the design of structures, they include no reference to the design of the ISRS for seismic loading. The current thesis work involves a numerical study of the ISRS joints and their influence on the performance of the entire system under different earthquake records. Two numerical models are constructed using ABAQUS and analyzed. Both models have identical dimensions but different cases of beam-column joints (hooked and semi-rigid). The study applied Incremental Dynamic Analysis (IDA) on 22 actual earthquakes. For each case, two different parameters were calculated 1) the maximum inter-story drift and 2) the maximum beam-to-column connection rotation. To handle the large number of iterations in this parametric study, a Python code was implemented. While producing fragility, three limit states were considered in this research: immediate occupancy (IO), life safety (LS), and collapse prevention (CP) with threshold values chosen as 1%, 2%, and 4% drift ratio, respectively. It was proved that the semi-rigid model is stronger and stiffer than the hooked model since the base shear was greater even at the same PGA. Moreover, in the fragility analysis, the semi-rigid models experienced less probability of exceeding the specified capacity limit states, leading to the inference that the semi-rigid connections provide higher strength and stiffness to the whole structure.

Key Words: Thin-walled steel profiles, Industrial Storage Racking Systems (ISRS), Dynamic behavior, IDA, Fragility.

ÖZET

İnce duvarlı çelik profillerden yapılmış Çok Katlı Endüstriyel Depolama Raf Sistemleri (EDRS), Bu sistemler depreme maruz kaldığında riskli sonuçlara yol açan hassas imalat ve hafif yanal sertlik ile karakterize edilirler. Yapıların tasarımına ilişkin ulusal yönetmelikleri inceleyerek, sismik yükleme için ISRS tasarımına herhangi bir atıfta bulunmazlar. Mevcut tez çalışması, ISRS eklemlerinin sayısal bir çalışmasını ve bunların farklı deprem kayıtları altında tüm sistemin performansı üzerindeki etkilerini içermektedir. ABAQUS kullanılarak iki sayısal model oluşturulmuş ve analiz edilmiştir. Her iki model de aynı boyutlara sahiptir ancak farklı kiriş-kolon birleşim durumları (kancalı ve yarı sert) vardır. Çalışma, 22 gerçek depremde Artımlı Dinamik Analiz (IDA) uyguladı. Her durum için, iki farklı parametre hesaplandı 1) maksimum katlar arası kayma ve 2) maksimum kiriş-kolon bağlantı dönüşü. Bu parametrik çalışmada çok sayıda yinelemeyi işlemek için bir Python kodu uygulandı. Kırılabilirlik üretilirken, bu araştırmada üç sınır durum göz önünde bulundurulmuştur: sırasıyla 1%, 2%, ve 4% sürüklenme oranı olarak seçilen eşik değerleri ile anında doluluk (AD), can güvenliği (CG) ve çökme önleme (ÇÖ). Yarı rijit modelin kancalı modelden daha güçlü ve daha sert olduğu kanıtlanmıştır çünkü taban kesme aynı PGA'da bile daha büyüktür. Dahası, kırılabilirlik analizinde, yarı rijit modeller, belirtilen kapasite sınır durumlarını aşma olasılığının daha az olduğunu tecrübe etti ve bu da yarı rijit bağlantıların tüm yapıya daha yüksek mukavemet ve sertlik sağladığı çıkarımına yol açtı.

Anahtar Kelimeler: İnce duvarlı çelik profiller, Endüstriyel Depo Raf Sistemleri (EDRS), dinamik etkiler, IDA, kırılabilirlik

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LIST OF ABBREVIATIONS AND ACRONYMS

<u>Abbreviations and Acronyms</u>	<u>Explanations</u>
Δ	: Displacement
θ	: Rotational Angle
ν	: Poisson's ratio
4P	: 4-Pin
P- Δ	: Force - displacement
A_{max}	: Max Acceleration
E	: Young's modulus
g	: Gravity
H	: Clear height of uprights
kN	: Kilo Newton
kN/m	: Kilo Newton per meter
L	: Clear span of beams
m	: Meter
M_s	: Surface wave magnitude
Mpa	: Mega paskal
N_{bay}	: Number of bays
N_c	: Number of beam-upright connections
N_L	: Number of levels
NP	: No Pin
sec	: Second
V_{max}	: Max Velocity
W_{pi}	: Pallet weight
CBF	: Concentrically Braced Frame
CP	: Collapse Prevention
DDBD	: Direct Displacement-Based Design
EDPs	: Engineering Demands Parameters
FBD	: Force-Based Design
FEMA	: Federal Emergency Management Agency

IDA	:	Incremental Dynamic Analysis
IM	:	Intensity Measure
IO	:	immediate occupancy
ISRS	:	Industrial Storage Racking Systems
LS	:	Life Safety
MDOF	:	Multi Degree Of Freedom
NSPA	:	Nonlinear Static Pushover Analysis
NTHA	:	Nonlinear Time History Analysis
PBEE	:	Performance-Based earthquake engineering
PEER	:	Pacific Earthquake Engineering Research Center
PGA	:	Peak Ground Acceleration
RMI	:	Rack Manufacturers Institute
SDOF	:	Single Degree Of Freedom

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1. INTRODUCTION

Multi-storey and open Industrial Storage Racking Systems (ISRS) constructed from thin-walled steel profiles constitute an important part of the supply chain systems. Their importance stems from their location and ease of access in warehouses where they are used for short-term and long-term preservation of manufactured products. In regions reported with high earthquake hazard, the ISRS are expected to carry much more load than their specific weights. Their quite delicate fabrication and light lateral stiffness of the crosses would lead to undesired consequences, especially when loads originally carried by the ISRS are not uniformly distributed [1].

1.1. The Rational Of The Thesis

By reviewing the national regulations for the design of structures, they include no reference to the design of the ISRS. However, on the international scale, some design methods have been published recently [2] which address the design of the ISRS under static conditions only. They do not take into account the dynamic effects of external loads. The necessity of considering the dynamic effects is proven by the earthquake records in Italy since 2012 which show that earthquake loads should also be considered in the design of the ISRS. Therefore, this thesis is concerned with establishing a numerical study to obtain a good insight into the necessary design regulations for the ISRS for earthquakes. The study majorly focuses on the response of two different types of joints commonly used in the ISRS when they are subjected to several earthquake records. Their effect on the overall performance of the ISRS structure is investigated by resorting to the incremental data analysis (IDA) and the fragility analysis.

1.2. Scope

The current thesis work involves a numerical study of the ISRS when subjected to different recorded actual earthquakes. Only unidirectional analysis is covered by considering the down-aisle direction alone. Two numerical models are constructed in the ABAQUS environment and analyzed using the finite element method. Both models have identical dimensions but different cases of beam-column joints (one with hooked joints and the other with hooked joints attached to pins on two sides).

The study applied data recorded for 22 actual earthquakes taken from Pacific Earthquake Engineering Research Center (PEER) ground motion database. The peak ground acceleration (PGA) of each earthquake is applied with 40 scales ranging from (0.1 - 4.0) times the original PGA. After applying these earthquakes to the involved ISRS frames, fragility curves are produced, with the damage limit state taken as the global drift.

Because of the large number of the analyses conducted in this study, the analysis process is automated using Python scripts. In these scripts the analysis is controlled to iterate over the parameter ranges indicated above. Python scripting is also used in the postprocessing stage to evaluate results statistically.

1.3. Thesis Outline

The report at hand starts with introducing the rationale of the thesis work, and the incentives that led to explore the intended topic. The first chapter also states the scope covered and the details probed throughout this work.

The second chapter provides an overview about the storage rack systems design procedure, starting with explaining the seismicity in Turkey. The characteristics of the storage rack systems are also discussed, followed by introducing two approaches to the design of these systems, one is based on the gravity loads and the other based on the seismic load.

In the third chapter, the concept of performance-based design of storage rack systems is explained. It addresses two analysis methodologies on which design is based – the nonlinear static pushover analysis and the response time history analysis.

The framework of this research is presented in the fourth chapter, including the important features of the numerical modeling in ABAQUS program and the controlling of this process through Python scripting. This chapter also presents a numerical example to explain the steps followed in the modelling process and the numerical results that ensued.

The results as well as the findings of the Incremental Dynamic Analysis (IDA) and the Fragility Analysis which is presented in the fifth chapter are subjected to a thorough discussion at the end of the chapter.

This report ends with chapter 6 which summarizes the important concluding remarks extracted out of this research work. In the same chapter is listed a number of recommendations to extend this work in the future.

2. STORAGE RACK SYSTEM DESIGN PROCEDURE

By looking to the structure of storage rack systems, features of simplicity may dominate at the first glance, but this view is immediately overturned as soon as this type of structures is scrutinized closely. This chapter presents a description of the storage rack systems in terms of their structural fabrication, modeling, design and the seismic demand these structures are expected to meet, particularly in Turkey.

2.1. Seismicity In Turkey

The majority of Turkey is characterized by regions known to be active with earthquakes. Among these regions, the northern part of Turkey is affected by a right-lateral fault called the North Anatolian fault extends from Karliova to Istanbul. Along this fault have occurred a number of earthquakes of high magnitudes ($M_s > 7$) that led crust to rupture and to experience coseismic surface faulting. Most of these events took place during the 18th and 20th centuries at the region of Marmara. Some regions in Turkey faced only mild earthquakes including the northwestern Sea of Marmara and the east of Bursa. Historically, most of the earthquakes that have stricken Istanbul were offshore and ranged in magnitude between 6.8-7.2, which are less severe than those that have been reported in the west of Marmara [3].

2.2. What Are Storage Rack Systems?

Storage rack systems are light structures used for storing goods in the industrial facilities. They save space by accommodating densely stored items. The structural concept behind this system is that it transfers the weights of the items from shelters on which they are placed, to side beams and finally to several uprights. The uprights and the beams are made of cold-formed steel. Beam ends are attached to pre-welded fastening tabs. These tabs are inserted into perforations that are distributed along the uprights. To avoid pulling out of the uprights when uplift loading occurs, the end tabs are locked into the uprights' perforations using pins. One of the challenging aspects of

the rack storage systems is that they are intended to carry live loads that exceed their self-weights [4].

The thin layout of the upright columns and the intermediate perforations make them vulnerable to different types of buckling. Global buckling may be observed in the entire column, local buckling in the flanges or distortional buckling. The beam-column and base plate connections usually experience nonlinear response at loading. The axial loads transferred from the columns determine the capacity of the base plate connections [5].

As for the lateral stability of the storage rack systems, it is enhanced by installing lateral braces in a direction transverse to the aisle, called the cross-aisle direction. Braces are absent in the longitudinal direction (called the down-aisle direction) to avoid obstruction to the accessibility to the stored products. The lateral stability in this case is maintained in reliance on the performance of the beam-column joints as well as the base plate joints [5].

The beam-column joints existing in the storage rack systems are semi-rigid in nature. The rigidity of a joint connecting two member ends is determined by the degree one end rotates relative to the other at some moment value. If the same value of moment is applied to two different joints, the joint with the least relative rotations has the highest rotational stiffness. A connection is called rigid when the difference between its end members rotations is zero, and therefore the rotational stiffness is infinity. The opposite extreme case is the pinned joint which has zero rotational stiffness, and therefore it conveys no moment across the connected members. Beam-column connections in the rack storage systems take an intermediate nature called semi-rigid. To determine the response of these connections under seismic loading, they are subjected to numerical or experimental tests. The results aid in the design of these connections to provide the expected performance within the entire frame [6]. The partial flexibility in the rotations of the joints existing in the storage rack systems leads to significant distortions in the connected columns. These distortions play an important role in reducing the axial stiffness of braces to a notable degree [7].

Beam end connectors behavior is dictated by the type of the tab and bracket from which connectors are made. It also depends on the beam size to which the tab is attached and the manner the beam is positioned on the bracket. The larger the number of tabs, the higher the strength and the stiffness of connectors and the entire frame. To increase

resistance to uplifting actions, upwards or pins are added to the tabs. This helps connectors to stay linked with the columns [8].

2.3. Design of The Rack Systems

The complex layout of the columns in the storage rack systems makes difficult the prediction of their behavior theoretically. Therefore, the available design standards have been established by considering the findings of experimental tests performed on individual members and joints. In the cases where design is based on structural analysis, the frame should be modelled as a sway frame, with semi-rigid joints. For the significant slenderness of the modelled parts, analysis should consider the second order effects as they are no longer negligible in this case [5].

2.3.1. Gravity Design

Unbraced rack systems were analyzed by [9] under loads in the gravity and horizontal directions. The concept of equivalent free-sway column was implemented in this study. Solutions were obtained by finding solutions to the flexure governing equation while allowing the P- Δ effect. The solutions accounted for imperfections in the frame. To verify the results, comparisons were made with nonlinear finite element analysis solutions.

2.3.2. Seismic Design

According to the Federal Emergency Management Agency (FEMA) report number 460, analytical equations are proposed to predict the base shear, top lateral displacement and the fundamental period of the storage rack system due to seismic loading. The equations are developed based on the displacement-based seismic design approach. The outcomes of these equations are found by taking the characteristics of the beam-column joints as inputs, including the rotational stiffness as well as the rotational capacity of the joints [6].

FEMA 460 recommends determining joints characteristics by conducting experimental tests. It is certainly inaccurate to quantify them based on the analytical

formulas usually used for joints in normal steel structures. This is due to the non-uniformity in the engaged member sections and the numerous perforations in their flanges. However, in reality, it is difficult to capture the behavior of the joints through experimental tests due to their semi-rigid behavior. Another reason is the wide variety of joints types being used [6].

As for the design standards of the storage rack systems, the currently available design standards are the American specifications such as the (Rack Manufactures Institute Specifications for the Design, Testing and Utilization of Industrial Steel Storage Rack, RMI 2008), and the (Steel Static Storage Systems Adjustable Pallet Racking System-Principles for Structural Design EN15512-2009) in Europe. Different countries in the world adopt various cross-section specifications of members. This makes other countries lack the development of compatible design standards for their own use, as in the case of Turkey. Accordingly, less care is expected to be taken in the design of storage rack systems which would lead to disastrous economic effects in addition to losses in life. Because of this high risk, further research should be pursued to help in developing design provisions of the beam-column connections in the first place as their performance affects the entire system [6].

Design philosophy is based on the three essential pillars – safety, regulations and economy. Due to the insufficient familiarity with the behavior of the storage rack systems, the first two pillars are ignored and the most consideration is given to the economy factor. In other words, designers put more emphasis on producing the cheapest designs at the expense of safety and regulations, which leads to risky structural designs [6].

A lot of research has been dedicated to understand the behavior of the steel storage rack structures under seismic loading. The objective is to find significant solutions for the problems mentioned above. Most of the research focuses on finding solutions specially for the safety parameter in the design of the rack structures. This thesis aims to propose different solutions for three main parameters safety, economy and regulations in the design of the storage rack systems.

A design approach that has long been applied to the seismic design of structure was revisited by [10] to expand its applicability to rack systems. This approach tries to equip the structure with the necessary strength, stiffness and energy dissipating capabilities during earthquakes. Based on this approach the expected plastic hinges are identified which represent the zones where received energy from earthquakes is

dissipated. This study tried to investigate the development of the plastic hinges in the rack systems during seismic action. The basic premise indicated that collapse in frames are referred to the interaction between instability and plasticity in beam-column connections. It is observed columns are unexpected to reach their ultimate capacity and beams rarely experience plastic hinges at their midspans. For this reason, it is more probable that plastic hinges develop at the joints connecting beams to columns, and that their hysteric behavior determines ability to dissipate energy. As the behavior of the beam-column joint influences the behavior of the entire frame, this study implemented an experimental program to have more in-depth understanding of the joints. The first stage of the work consisted of testing the joints under quasi-static cyclic reversal loading. In the second stage, full scale tests were conducted on a shaking table for complete rack structures.

An analytical study was conducted by [11] on a rack system loaded with goods by constructing a numerical model using ABAQUS software. The system was subjected to the Elcentro earthquake by inserting the spectrum data of the earthquake as an input to the software. The primary goal of this research is to observe the dynamic behavior of the system and recommend structural improvements in light of the results. Eccentric braced frames are commonly used in the design of rack systems. The advantages of using eccentric braces to stabilize the rack frames were investigated by [12]. They analyzed the braced frames based on the response spectrum approach. Authors concluded that significant amount of steel material can be saved when eccentric braces are used. Compared with the centric bracing systems, the eccentric braces proved their ability to deform significantly within the inelastic range which mitigates the impact of earthquakes. Authors recommended to subject this study to verification based on experimental studies as none have been conducted yet.

2.4. Modelling of Storage Rack Systems

One of the common challenges in the modelling of storage rack systems is the semi-rigidity of the beam-column joints. This property is referred to the distorted part of the column or the beam at the zones connected to the joint. In addition, as a result of the large loads and the loads of cyclic nature, perforations may be torn. This adds another reason to the semi-rigid nature of the joints. This makes it challenging to

simulate the behavior of these joints especially that a wide range of connection types is available. The solution to this problem is to conduct experimental tests to measure the moment force developed at the joint and the associated rotation. This makes possible the calculation of connections stiffness and their behavior accordingly [13].

Several authors attempted to construct mathematical models that provide a good idea to the behavior of the joints. A polynomial function was proposed by [14] that can be used for evaluating the moment-rotation of several styles of bolted connections. A number of constants has to be provided in addition to a standardization factor to fit the curve. A power model was established by [15] that requires 3 parameters – the initial stiffness, ultimate moment capacity and the shape parameter. A mathematical model was developed by [16] to predict the nonlinear behavior of the beam-column joints in the storage rack systems based on several experimental studies. Other authors such as [17] conducted experimental and numerical studies to determine the rotational capacity of the joints fastened with bolts. The considered joints link members with C-shaped cross-sections together. The main aim was to deploy the advantage of the semi-rigidity of the joints in the design of latticed beams. The work of [18] addressed the non-linear analysis with high strains beyond the elasticity range to reach more accurate understanding of the real behavior of joints.

The behavior of connections of high flexibility was tested by [19]. They performed an experimental study in which they applied cyclic loading and observed how joint response affects the overall performance of the frame. To have more general idea about this performance, different types of fasteners were used in this study in the beam-column connections.

A sway analysis of spliced structures was conducted by [20]. The non-linear P- Δ effects were incorporated in the analysis through constructing modified stability functions. The semi-rigid nature of all joints was considered, and the results of this study were verified by comparing to the finite element results.

Nonlinear analysis of a rack structure exposed to various seismic conditions was performed by [21]. He proved that the resulting displacements are as twice as those obtained from the linear analysis.

For any structure, the fundamental time period as well as the other mode shapes play an important role in identifying the behavior of structures under seismic loading. For this purpose, several studies have been done on hot-rolled steel structures and reinforced concrete structures. These studies discussed a wide spectrum of the analysis

complexities of the typical steel and concrete structures and suggested several solutions to the expected analysis and design issues. The storage rack systems can be treated in a similar manner as to the traditional structures, but wiser engineers should take the major engineering differences into consideration. Specifically saying, the inherent characteristics of the rack systems that make their behavior under seismic loading hard to predict using simple theories or traditional analysis tools. The semi-rigidity of the beam-column connections and the existence of perforations along the uprights are all plain indications of the complex structure of the rack systems. Using the computational power of computers to perform sophisticated numerical analyses is the best way to handle the structural complexities [22].

Pallet rack connections were investigated by [8]. They carried out 18 experiments with the main parameters were the column thickness, beam depth and end connector depth. The connectors were exposed to moment, shear and axial forces to account for all possible force conditions in reality. The main objective was to create a simple analytical model that can be used to evaluate the rotational behavior of the connectors.

In this thesis, the model setup was designed in ABAQUS to closely match the reality of the experimental study of [6]. Therefore, two different models were created, the first model was the boltless hooked (no pins) connection, which maintains its connectivity through the hooks at the beam end connector bracket that connect the beam to the column. The second model was to simulate the 4-pin connection, in which the same assembly of the first model was used.

3. PERFORMANCE BASED DESIGN OF STORAGE RACK SYSTEMS (PBEE)

3.1. What Is PBEE

Performance-based earthquake engineering (PBEE) is the process where structures are designed and evaluated to meet specific performance criteria. The concept behind this process is based on predicting the performance of structures with measurable reliability under ordinary and extreme loading. This performance is discussed with the client in order to reach appropriate trade-offs based on life-cycle considerations rather than construction costs alone [23].

As a design concept, PBEE is limited by legal and professional obstacles, but whether it can fulfill its promises or not is still a matter of question. If it is targeted to predict structural performance in quantifiable formats then it will be a losing cause. In fact none of the experts suspect that seismic demands and capacities can not be estimated with high reliability, even in a probabilistic format. However, several reasons justify the preference to PBEE as an active and imperative area for research and application [23].

Grand interests have been put so far in the enhancement of earthquake engineering knowledge for different reasons. For one of them is the growing knowledge about seismology and the characteristics of ground motion. The second reason is that many governments realized that the resulting financial loss is by far much significant compared to their anticipations. One may argue that the strongest reason is that current code design procedures do not provide logical answers that satisfy (1) the designer's search for persuasive explanations of the design rules, (2) the owner's willing to reach wise decisions on the costs and usefulness of protection against seismic events, and (3) the society's needs for well-based decisions regarding the highly uncertain seismic demands and seismic capacities of the existing and new constructions. At present, there is a common consent that earthquake design should consider a variety of performance objectives. Design provisions that prevent partial collapse that hazard human lives is currently receiving minimal interest. Society's demands have been extended to include the continuing operation of critical facilities, ensuring safe evacuation of dangerous materials, and the prevention of large-scale

damages that may significantly impact societies at narrower and wider levels. Adding to that, more insightful owners demand maximizing their return on investments from their facilities. All that induces structural engineers to reach predictions of damage levels and their probability to achieve judicious decisions. This necessitates shifting from empirical and experience-based judgments to assessment procedures more scientific oriented with high capabilities of predictions [23].

The PBEE concept is intended to evaluate the seismic risk of structures based on certain performance objectives. A performance objective indicates the probability of a structure to surpass a defined damage state located somewhere between the elastic condition to the complete collapse when this structure is subjected to a specific hazard level. Over the recent 15 years, the Pacific Earthquake Engineering Research (PEER) Center has developed a consistent methodology for PBEE. It starts with specifying a ground motion intensity measure that bears the characteristics seismic hazard that influences the dynamic response of a structure. A nonlinear model of the structure is subjected to a group of ground motions. This model is analyzed and then sought to provide us the Engineering Demands Parameters (EDPs) of the structure such as story drifts, shear forces, overturning moments and inelastic deformation demands. The EDPs are then associated with damage measures that give a description to the damage state of the structure or part of it. Damage measures provide assessments of the impacts on human lives and financial losses. These two parameters are the props of the decision-making process [24].

There are two main factors that represent the major challenges to the PBEE. The first is the availability of highly reliable deterioration models for different structural components and the ability to employ them in the modern analytical tools. The second factor is about handling the uncertainties in identifying the structural components properties and the consequences of uncertainties on the seismic performance of structures [24].

3.2. Nonlinear Response

Nonlinear analysis represents an essential base for the PBEE. With the availability of escalating computational power, nonlinear analysis is capable to provide accurate previews of structural response which serve the purpose of the PBEE. Various

types analyses consider the nonlinear behavior of structures and they are implemented to achieve a variety of goals. The pushover analysis, for instance, reveals the full nonlinear behavior of structures and their components up to failure due to static loading. A major drawback of this method is that it scales the structure down to a single degree of freedom (SDOF) and it fails to account for the dynamic and duration effects of the structure. A more preferable method is the time domain analysis. This method is able to predict the nonlinear dynamic performance of structures under the effect multiple ground motions [25].

Several researchers applied the nonlinear analysis to study the rack structures. [26, 27, 28] studied the seismic behavior and collapse of the pallet rack structures under the effect of ground vibrations. [29,30] presented a novel methodology that combines the nonlinear time history method with the low cyclic fatigue damage approach to estimate the residual fatigue life, levels and locations of damage and the new load-bearing capacity of structures after experiencing a seismic event. They developed seismic design methods for the rack structures with higher reliability.

3.3. Analysis Methodologies

3.3.1. Nonlinear Static Pushover Analysis

Nonlinear static pushover analysis (NSPA) is an approach performed to estimate the capacity of a structure in terms of strength and deformation. These capacities are put into comparison with the demands to meet specific performance levels. NSPA is achieved basically by subjecting the structure to lateral loads in the principle directions of the structure and letting these loads increase in a monotonic manner. The sought output is the history of the structural behavior during load increase until collapse. This type of analysis has succeeded in recent years to seize the interests of engineers and to participate into the guidelines of seismic design. NSPA is distinguished over the elastic static and the dynamic analyses in that it provides different useful response information. A number of its features are mentioned below [31]:

- It is needed to gain an insight to the strength and deformation capacities of the structures for the fundamental mode of vibration.

- It enables analysts and designers to locate the weak points in the structure. The weak points are those characterized by high inelastic deformations.
- It helps to identify the influence of a deteriorated component on the performance of the whole structure.
- The order in which failures occur within the structure and the trend taken by the capacity curve during loading progression.

NSPA has been the focus of several researchers. A simplified approach to the seismic analysis called N2-method was proposed by [32]. This approach is a combination of the pushover static analysis results of a multi degree of freedom (MDOF) model with the response spectrum analysis results of the single degree of freedom (SDOF) version of the same model. This method relates the parameters that control the seismic behavior of the structure. It provides the ability to interpret the seismic response visually. The NSPA is the most preferred over other pushover analysis techniques for its simplicity and accuracy for the fundamental mode of vibration despite its low consumption of computational efforts.

3.3.2. Response Time History Analysis

Nonlinear time history analysis (NTHA) allows knowing the response of a structure over a period of time during which a varying load is applied. It provides explicit results for the behavior of a structure under the action of a recorded earthquake. Taking this point in mind, the NTHA method excels over other techniques such as the direct displacement-based design (DDBD) and the force-based design (FBD) which consider not to exceed a target maximum displacement, regardless of the time history response of the structure [33]. Interested researchers such as [34] suggested to impose improvements to the NTHA techniques to provide better predictions of the displacement response of concentrically braced frame (CBF) systems. They developed an improved numerical model for CBF systems using the NTHA. The model was validated by comparing its predictions with the results obtained from real-time experiments in which the model is constructed and installed on a shaking table. The table was provided with real earthquake records scaled to reach significant ductility demand on the structure.

3.4. Incremental Dynamic Analysis (IDA)

3.4.1. History Of IDA

Today witnesses huge computational potentials thanks to the emergence of high processing capabilities of computers. This has encouraged a significant shift in the level of the analysis methods from traditional less practical methods to more complex and realistic ones. One of these methods that has been receiving much attention within the field of seismic analysis and design is called the incremental dynamic analysis (IDA). This method appeared for the first time in literature in 1977 [35]. Afterwards, several researchers have addressed it in their research works. In the recent years it has been adopted in the FEMA guidelines [35].

The IDA is a parametric study in which a structure is subjected to one or more ground motion time histories, each scaled to incremental multiples of intensity. Nonlinear dynamic analysis is then performed to obtain the response of structure versus the seismic intensity level. The IDA is currently essential for the performance-based earthquake engineering (PBEE). It provides a powerful tool to estimate the performance of structures under seismic loading [35].

3.4.2. Methodology Of IDA

The IDA requires constructing a nonlinear structural model and the availability of earthquake records as a first step. Each record is scaled to multiple levels that are pre-selected, and all the records with all their scaled levels are used to perform dynamic analysis of the structure. The IDA curves are then produced. Each curve provides the structural response given a ground motion intensity level. The structural response is expressed by a defined damage measure such as the peak interstorey drift. The ground motion is expressed by an intensity measure such as the peak ground acceleration (PGA). Interpolation of these values is performed and a summary is prepared to show how structural demand would correlate with the intensity measure. Finally, the limit states are selected for each curve to help in finding the probability that a limit state is exceeded when a specific intensity level is reached [35].

3.5. Fragility Analysis

3.5.1. Definition and History of Fragility Analysis

Fragility analysis is a common and efficient method for assessing the probability that a structure (or part of it) meets the seismic demand in a specific geographical location. The fragility of a structure is expressed through a probability function. Each value of the function represents the probability of seismic demand to exceed a prescribed limit state capacity, at a specific seismic intensity. Different measures are used to represent seismic intensity, but most commonly the peak ground acceleration (PGA) is used for this purpose. Various sources of data can be used to generate the fragility curves. Data can be taken from experimental works, numerical analyses or based on a specialist expertise [36].

3.5.2. Methodology of Fragility Analysis

The starting point in the fragility analysis is to decide a specific capacity limit state. A capacity limit state is the stage beyond which a structure or a component of the structure is no longer capable to satisfy its design requirement. Fragility curves can be based on a variety of capacity limit states and these curves provide a clear idea of the uncertainty that a specific limit state would meet the expected seismic demand, which is a very important feature of this kind of analysis. Inter-story drift or the peak floor acceleration are examples of capacity limit states that indicate a global failure in the structure. However, it is more advantageous to go to a lower scale by considering limit states of structural components, especially when estimating repair costs is of great matter [36].

3.5.3. Methods Used in Creating Damage Curves

Damage probability curves can be achieved in several different ways:

- Empirical fragility curves.
- Judgmental fragility curves.

- Analytical fragility curves.
- Hybrid fragility curves.

In this study, damage probability curves were obtained by analytical method and damage the parameters required to determine the distribution, calculation in the time domain method, and nonlinear dynamic analysis was performed. parameters required for determining the distribution of damage, were found by calculation in time domain method, and nonlinear dynamic analysis.

In order to collect data for the estimation of a fragility function there are various methods for conducting nonlinear dynamic structural analysis. One of the common approaches is the Incremental Dynamic Analysis (IDA); where the Intensity Measure (IM) caused by each ground movement to collapse in order to find a pair of ground movements are repeatedly scaled [12],[35].

Often a normal cumulative distribution function used for defining a fragility function:

$$P(C / IM = x) = \Phi\left(\frac{\ln(x/\theta)}{\beta}\right) \quad (3.1)$$

Here: $P(C / IM = x)$ leads to the probability of the destruction of the structure by a ground motion with $IM = x$; $\Phi()$, the standard normal Cumulative Distribution Function (CDF); θ , fragility median of its function (IM level with a 50% probability of collapse); and β for $\ln(IM)$ is the standard deviation (sometimes referred to as the IM distribution). Equation 3.1 shows that the IM values that cause destruction in the structure are distributed normally.

The fragility function parameters used in equation 3.1 “ θ ” and “ β ” requires estimation from the results.

3.5.4. Estimation of Fragility Function Parameters

There are some methods to estimate the fragility function parameters which are:

- i) Incremental Dynamic Analysis
- ii) Discrete Incremental Dynamic Analysis
- iii) Multiple Line Analysis

Incremental dynamic analysis allows the ground motion scaling to cause collapse. [35] This process (shown in figure 3.1 (a) as an example) associated with the onset of the crash for each ground motion generates a series of Intensity Measure (IM) values. Then, at a given IM level, x , crashes the probability of collapse occurring at a level lower than x which can be calculated as part of the records.

A graph of these possibilities is shown in Figure 3.1(b) referred to as the empirical cumulative distribution function. Fragility function parameters can be estimated by taking the logarithms of the IM values for the onset of collapse of each ground motion and calculating their average and standard deviations.

$$\hat{\theta} = \frac{1}{n} \sum_{i=0}^n \ln IM_i \quad (3.2)$$

$$\hat{\beta} = \sqrt{\frac{1}{n-1} \sum_{i=0}^n \left(\ln \left(IM_i / \hat{\theta} \right) \right)^2} \quad (3.3)$$

n , the number of ground movements considered and IM_i , the result of ground motion IM value associated with the start of the crash. It's a moment estimate a method; where $\ln \theta$ and β are the normal distribution of $\ln(IM)$ values, respectively, mean and standard deviation. If IM is lognormally distributed It should be noted that the average of $\ln(IM)$ will be equal to the median of IM; That's why using the same sample mean in this way reveals an estimate of θ . The mean and standard deviation of the distribution, or moments, are estimated using sample moments from a data set. A fragility function adapted using this approach is shown in Figure 3.1.

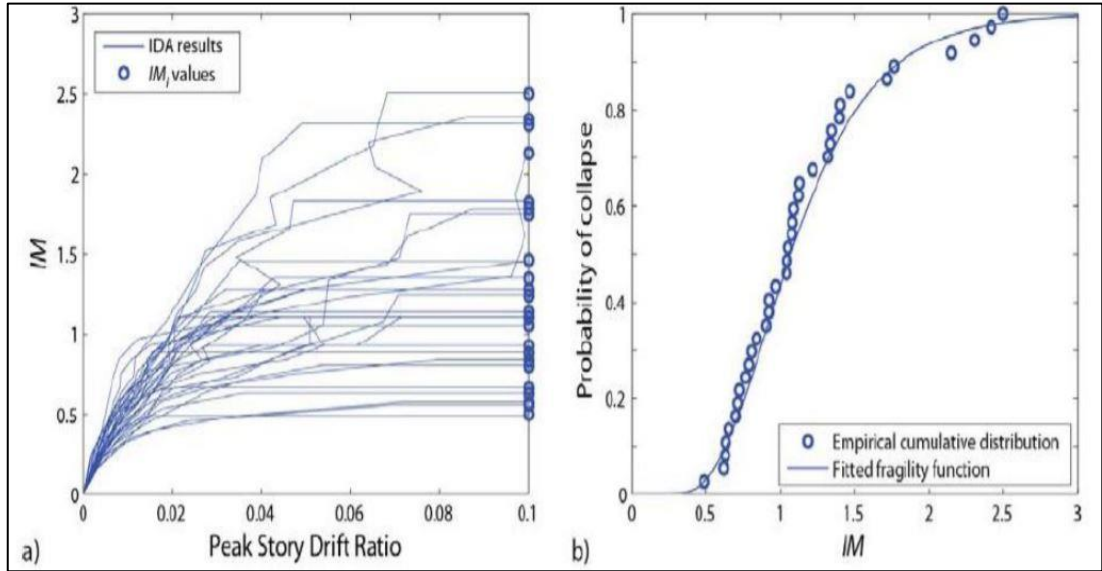


Figure 3.1: Incremental Dynamic Analysis Example Results.

3.6. Thesis Contribution

Previous research works yielded a plenty of different findings related to the design of the rack systems. However, it is still less clear the capacity values of the engineering demand parameters (EDPs) for each limit state. Therefore, in this research these demand parameters were obtained according to the results of the moment-rotation relationships and the drift values. Moreover, this research combined the IDA in the seismic safety assessment of racking system. This was performed using Python scripts to iterate analysis over a large number of numerical models. The outcomes of the IDA were then employed to obtain the fragility curves to enable prediction of the damage parameter probability at each certain PGA value according to the seismic area of construction.

4. MODELLING AND VERIFICATION

Numerical modeling is the main approach followed to study the behavior of beam-to-column connections in rack systems under earthquakes. The numerical models are verified by taking the experimental tests of [6] as a reference. A portion of these tests is numerically modelled herein. The setup of these tests and the list of models studied in this thesis are discussed in the following sections.

4.1. Case Study (Geometry and General Description)

The setup of the tests conducted by [6] is illustrated in Figure 4.1. Three different beam depths (100 mm, 120 mm and 140 mm box sections) were examined in the tests. The same column cross-section was used in all tests. In addition, constant column length of 500 mm, and beam length of 750 mm were used. Column and beam cross-sectional dimensions that are adopted in this thesis are given in Figures 4.2 and 4.3, respectively. As the latter figure shows, a single beam depth is involved in this thesis which equals 120 mm. The applied loads were measured with two 50-kN-precision load cells installed between the actuator rod and the beam-end clamp fixture. A constant 50 kN axial compression load was applied to the rack column by a hydraulic cylinder during the test. A 5 kN force was applied at each side of the beam to simulate pallet loads.

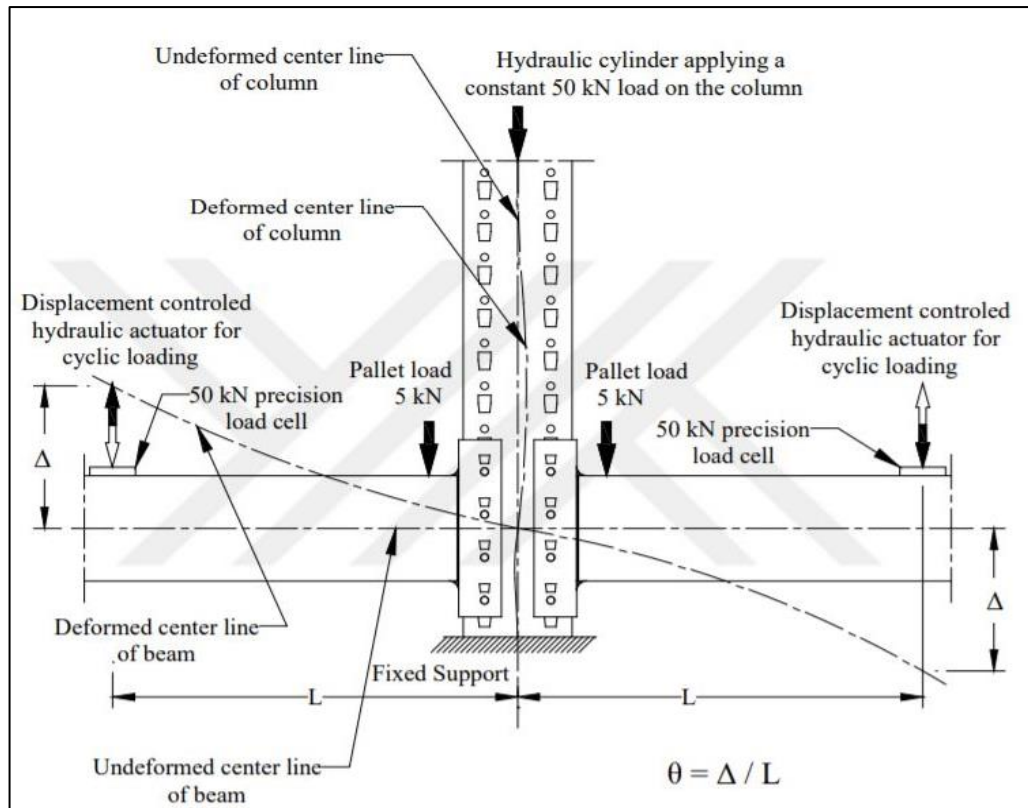


Figure 4.2: Schematic diagram of the test setup that was installed according to RMI 2012 specification.

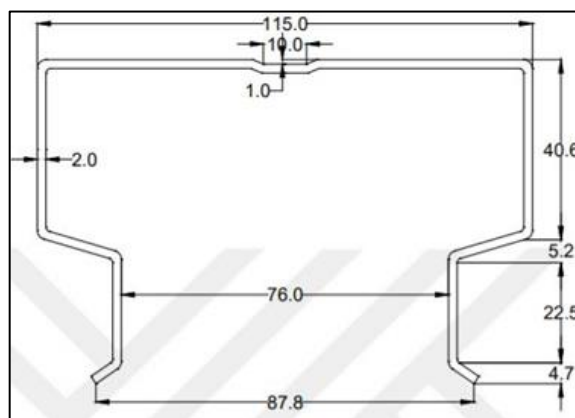


Figure 4.3: Column cross-sectional dimensions adopted in this thesis.

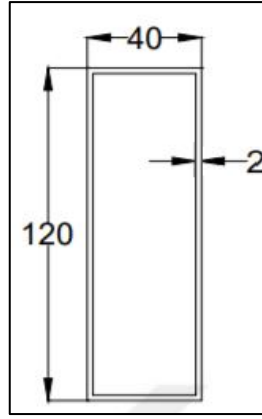


Figure 4.4: Beam cross-sectional dimensions adopted in this thesis.

For the connection types, three different connecting methods were adopted in the experimental tests (hooked, 2 pins added to both sides and 4 pins added to both sides). In this thesis, only the hooked and the “4 pins added to both sides” connections were studied, representing two behaviors, the hinge and the semi-rigid, respectively. Figure 4.4 shows the Schematic description of both types. In addition, Figure 4.5 displays the dimensions of the perforations in the column and bracket. The tests were carried out in 2015 at the Industrial Storage Rack Systems Design and Test Center at Buyukcekmece Campus, which is currently affiliated with Istanbul University-Cerrahpaşa.

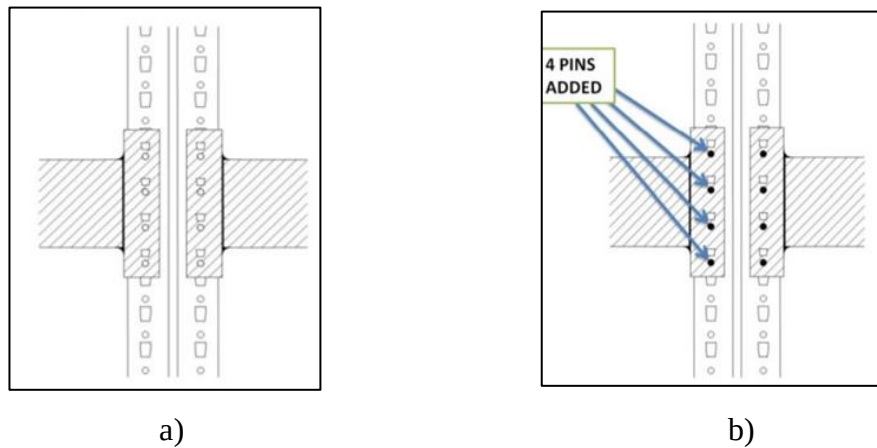


Figure 4.5: Beam-column connecting methods used in this thesis a) Hooked b) 4 pins added to both sides.

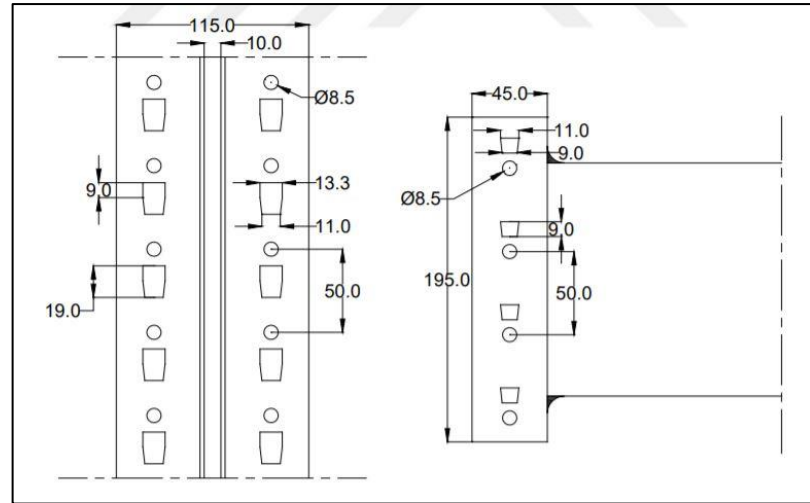


Figure 4.6: Dimensions of the column's and the bracket's perforations in mm.

Laboratory pictures were taken for the experimental model of the beam-column joint and the beam-column connecting methods used in this thesis, respectively displayed in Figures 4.6 and 4.7.

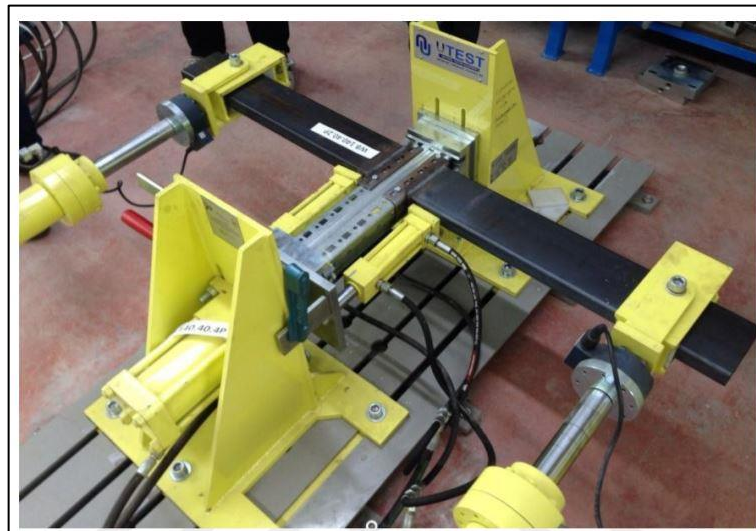


Figure 4.7: Setup of the experimental model of rack beam-column connection.

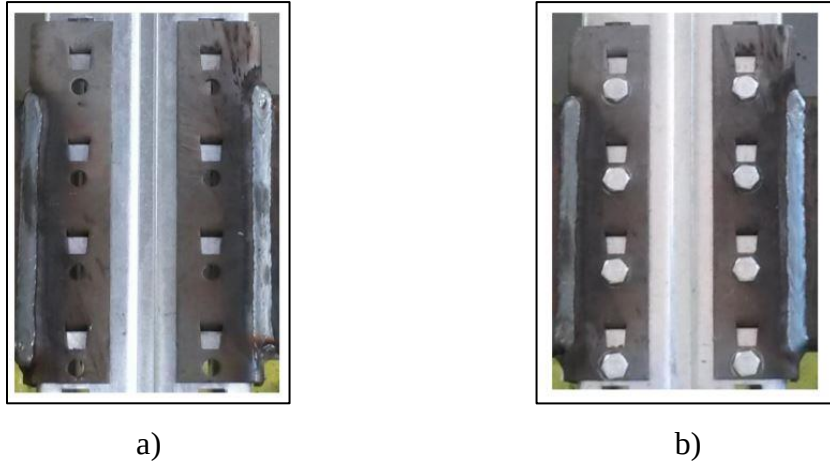


Figure 4.8: Actual pictures of the beam-column connecting methods used in this thesis a) Hooked (unbolted) b) double 4-pin (4 bolts).

The moment-curvature relationships of the hooked and the 4-pin connections were taken from the experimental tests of [6] and were employed to verify the numerical models prepared for the current study. These relationships were picked for the same dimensions indicated above and applied as inputs to a case study created to represent a rack system which is described in the next sections. After verifying this case study, it is used to carry out all subsequent analyses.

4.2. Numerical Modelling of The System

The product used to build the finite element models of the current study was Abaqus CAE version 6.13, using Abaqus/Standard analyzer.

The model setup was designed to closely match the reality of the experimental study. Therefore, two different models were created, the first model was the boltless hooked (no pins) connection, which maintains its connectivity through the hooks at the beam end connector bracket that connect the beam to the column. Therefore, in this model, the moment-rotation relationship which had been obtained from the experimental test of [6] was defined as the main property of the linking element between the column section and the beam. The second model was to simulate the 4-pin connection, in which the same assembly of the first model was used, but the only difference here is that the 4-Pin moment-rotation relationship was defined for the beam end connector. A large number of trials was done, excessive time was spent to get the results and promising results were finally obtained.

4.2.1. Steel Material Modelling

Isotropic A36 grade steel material was used for both column and beam sections with typical stress-strain material model. Figure 4.8 displays the stress-strain curvature of the steel material used in ABAQUS.

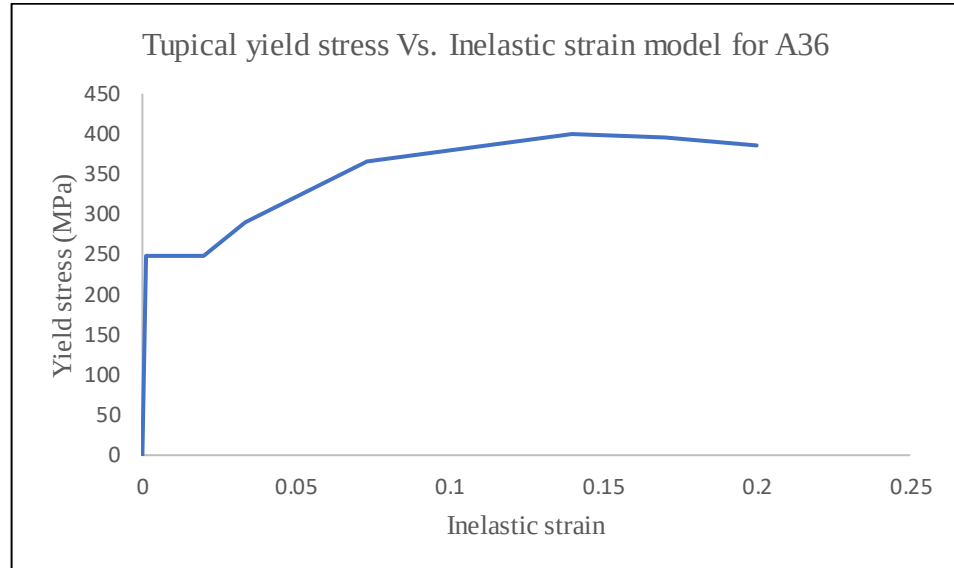


Figure 4.9: Typical stress-strain material model of steel used in ABAQUS.

4.2.2. Sections and Link Element Definitions (Moment-Rotation Relationships)

The column section dimensions were taken from the experimental study and then inserted to ABAQUS by specifying coordinates information of a group of points along a path passing along the center of the section as shown in Figures 4.9 and 4.10.

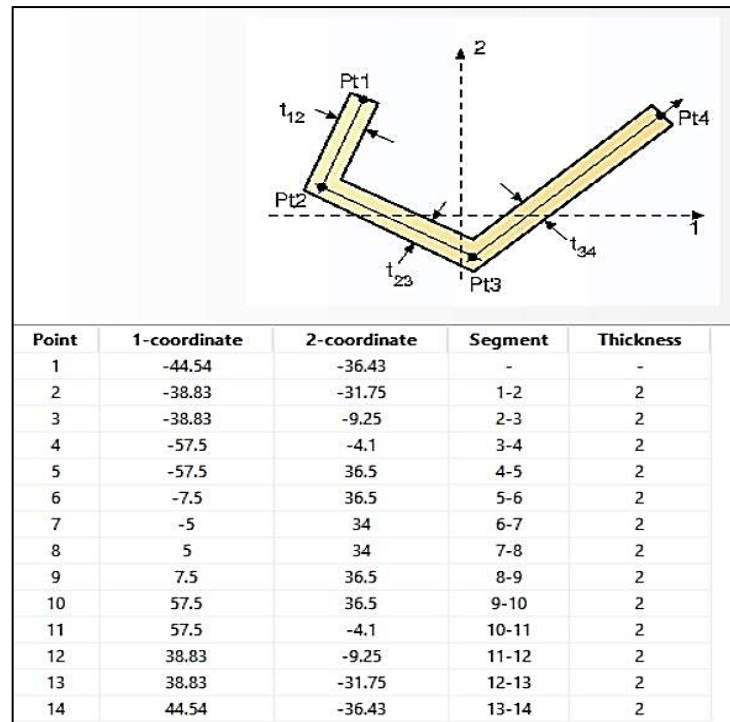


Figure 4.10: Defining column cross section in ABAQUS using coordinates information (units in mm).

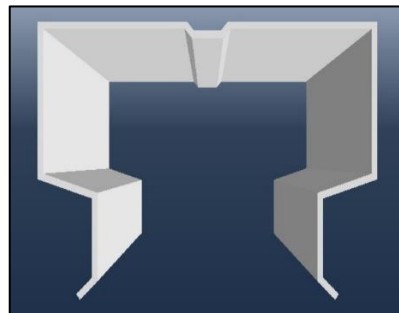


Figure 4.11: 3D view of the column section in ABAQUS.

The beam cross section was also modelled according to the dimensions used in the experimental test. It is a simple box with a depth of 120 mm, width of 40 mm and thickness of 2 mm as shown in Figures 4.11 and 4.12.

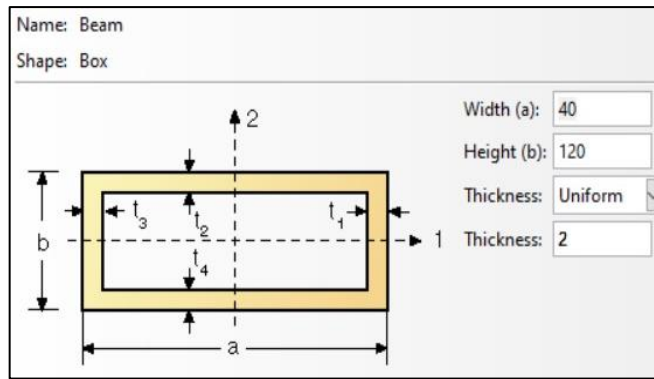


Figure 4.12: Defining beam cross section in ABAQUS (units in mm).

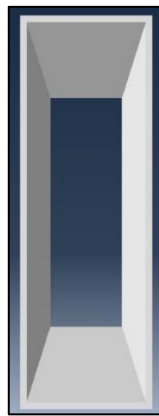


Figure 4.13: 3D view of the beam section in ABAQUS.

For the linking (connector) element, as is the case in SAP2000, ABAQUS allows the user to specify all needed properties of the link element. Before that, the user must specify the type of that connector which depends on the number and the type of the available degrees of freedom of that connector. In our case, “U Joint” was selected as the connector type. This type enables the user to insert the rotational properties keeping the translational degree of freedom pinned. This exactly simulates the connection type considered in this thesis. Figure 4.13 shows the connector type (U Joint) diagram.

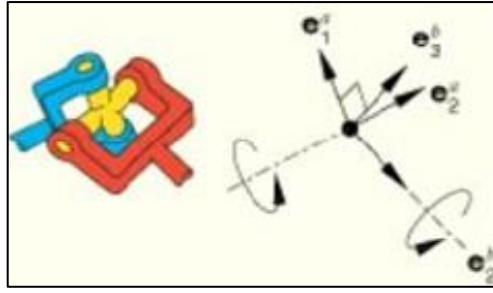


Figure 4.14: Diagram of a U joint to represent beam-column connector.

After selecting the connector type and its sections, then comes the role of the behavior options to be selected in ABAQUS which represent in our case the moment-rotation relationship properties which were taken from the experimental test. Because we have two types of connection (hooked and 4-pin), two models were thereby created. the moment-rotation relationship of the two types of connections as taken from the experimental test is depicted in Figure 4.14. The legend of this figure include the nomenclatures (NP = No Pin (Hooked), 4P = 4-Pin).

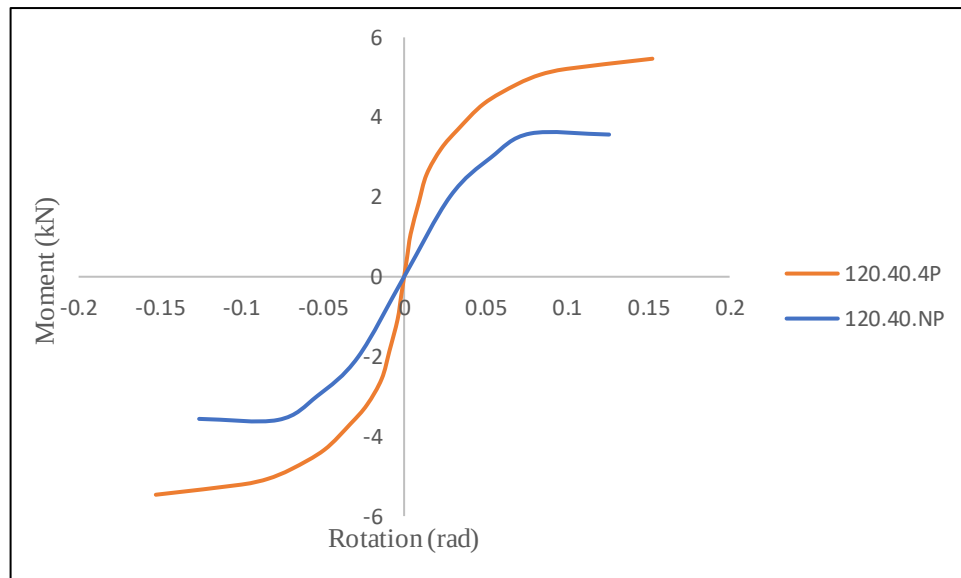


Figure 4.15: Experimental Moment-Rotation relationship of two connector types.

While assigning the connector section in ABAQUS, both elasticity and plasticity properties were defined according to the moment-rotation relationships stated earlier. To define the elastic behavior, the slope at the linear stage was inserted. On the other hand, the plastic behavior was defined as non-linear with kinematic hardening defined

as half-cycle. In that definition, the yielding moment and the inelastic motion (rotation) were deducted and inserted.

After the sections and the connecting elements between them had been defined, the whole model was created which is described in section 1 of this chapter. The model consists of three-story rack system of 4 bays with the dimensions mentioned earlier. Figure 4.15 shows the model as created in ABAQUS.

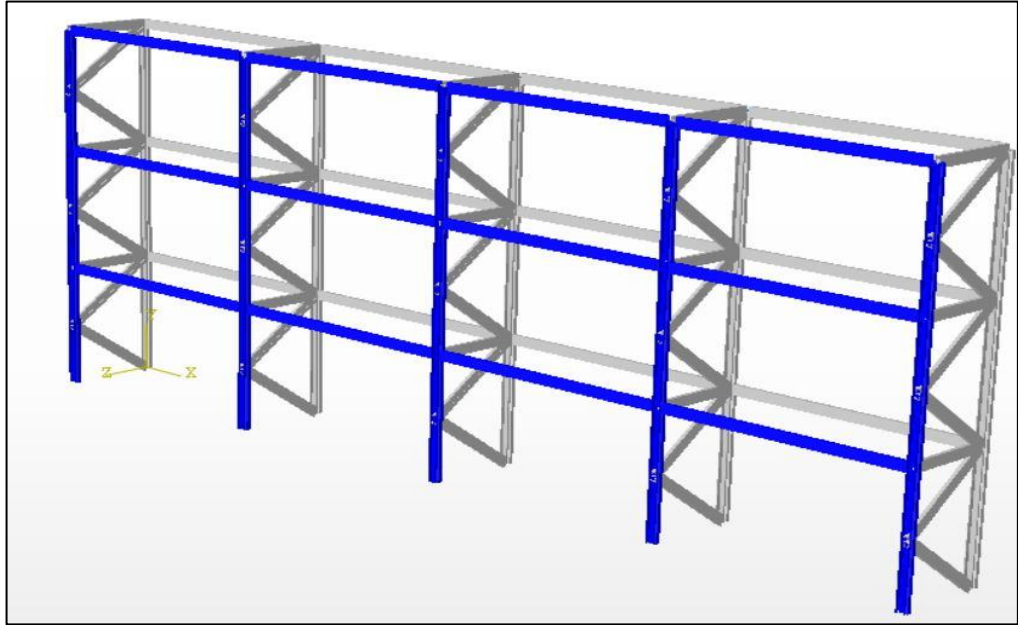


Figure 4.16: The selected frame model as created in ABAQUS.

In Figure 4.15 it can be noticed how the connector-link element (U Joint) is assigned between the end of each beam and the neighboring column. Figure 4.16 illustrates the definition of the link more clearly.

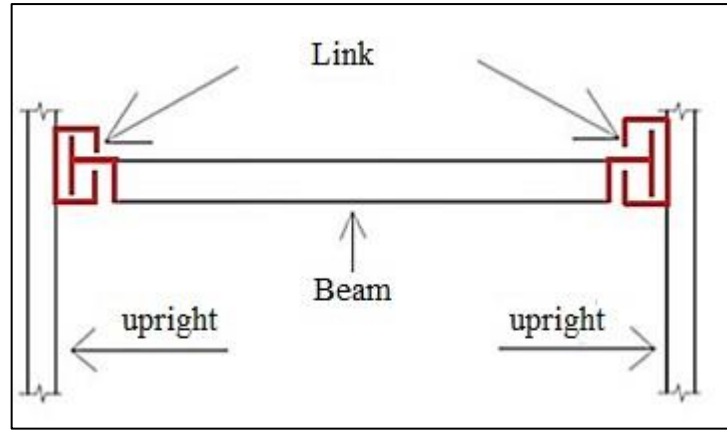


Figure 4.17: Illustration of the U Joint assignment in ABAQUS.

4.2.3. Loading Type and Boundary Conditions

In this research, two types of load were defined for the numerical model, gravity loads and seismic loads. Gravity loads consist of the total self-weight of all model components (columns and beams weights) which is calculated according to the section dimensions and material densities. The pallet load which is to be carried by the system is also assigned as a gravity load. In this research the pallet loads are considered uniformly distributed along the beam length with a value of 2 kN/m. Since the span length is 2.70 m, each span thus carries 5.40 kN, and since we have four spans and 3 levels, then the total pallet gravity load amounts to 64.8 kN. This load was assigned to the ABAQUS model. As for the seismic loads, multi earthquake records were used in this research (22 records) to construct the IDA curve based on the nonlinear dynamic response history analysis. The description of these earthquake records is simplified in the parametric study section. In addition, the time history graphs for the selected records can be found in Appendix A.1.

Regarding the boundary conditions, several conditions were assigned to the model to make sure that the model runs correctly in a way that simulates the reality. Therefore, the first assumption was to neglect outside motion by preventing joint motion in the Z-direction. Moreover, supports were considered pinned except in the direction of the earthquake, along which the earthquake load was defined as a vibrating boundary condition.

Figure 4.17 shows the loading system and the boundary conditions of the model in ABAQUS.

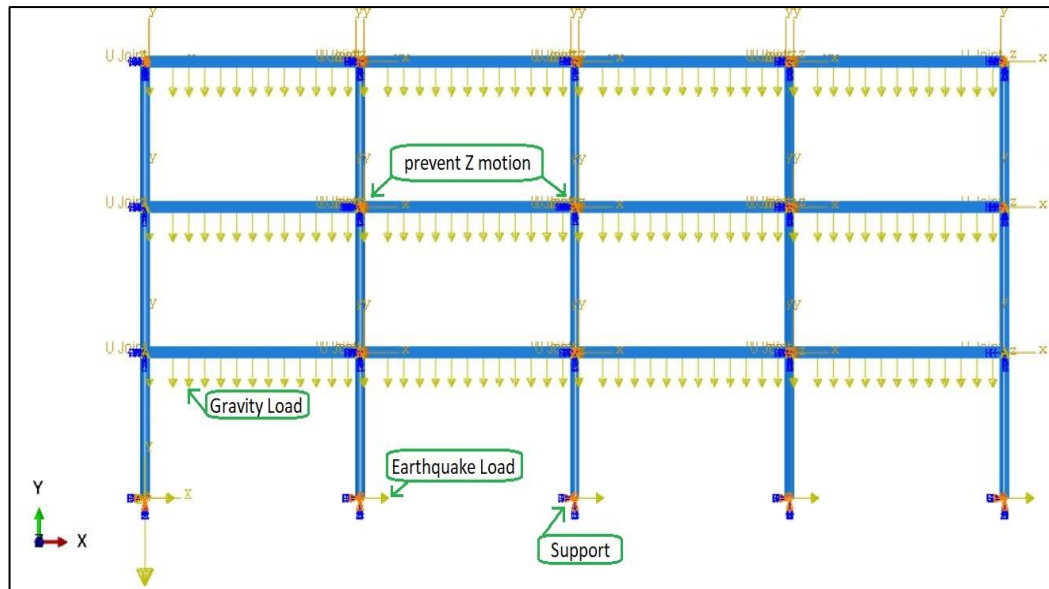


Figure 4.18: The loading system and the boundary conditions of the model in ABAQUS.

4.2.4. Mesh Type

B31 Beam element was used for both column and beam parts, which indicates a 2-node linear beam in space. For the mesh size it was selected as 50 mm to get appropriate and accurate results according to a sensitivity study that had been carried.

4.3. Verification

After the models had been completely prepared in ABAQUS, modal analysis was first run to verify its results by comparing them to those obtained from SAP2000 for the same models (NP and 4P). The results were almost the same. Figure 4.18 shows the first modal shape from the modal analysis of the hooked connection in SAP2000. The consistency between the two programs (SAP2000 and ABAQUS) can be noticed. For example, the frequency of the first mode was 3.43 cycle/time (0.29 s time period) and 3.54 cycle/time (0.28 s time period) in SAP2000 and ABAQUS, respectively, which implies a very good compatibility in the results of the two models. The first modal shape and the associated frequency of the hooked connection model created in ABAQUS are shown in Figure 4.19.

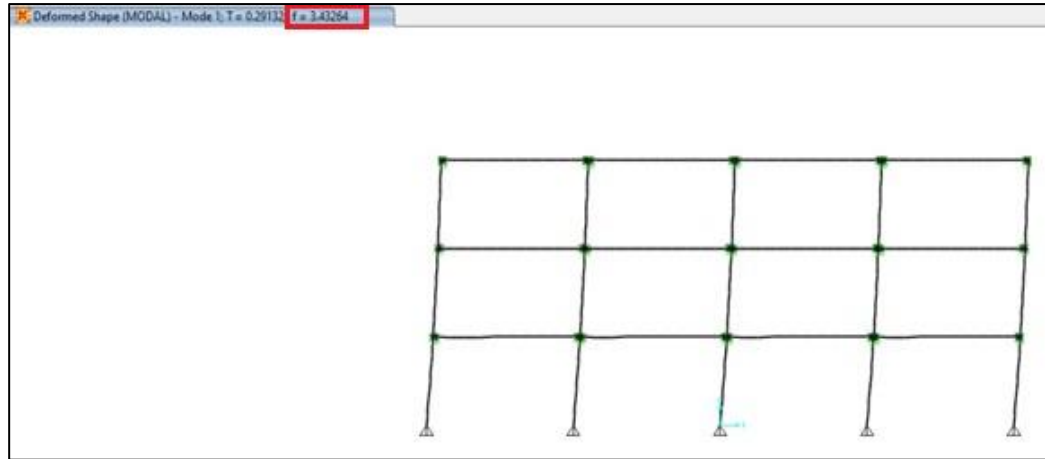


Figure 4.19: The first modal shape and frequency for the hooked connection model created in SAP2000.

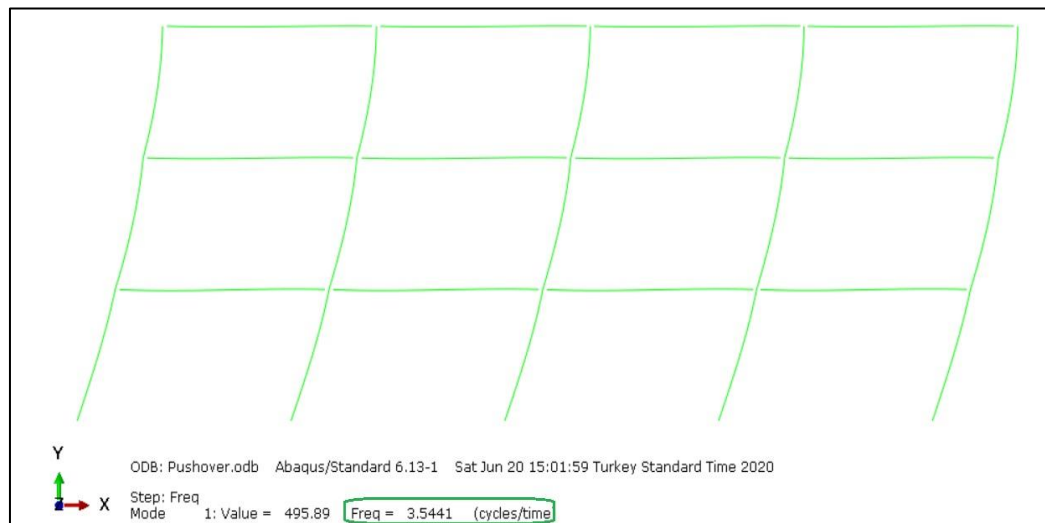


Figure 4.20: The first modal shape and frequency for the hooked connection model created in ABAQUS.

Modal analysis was also performed for the other connection type (4-pin). The frequency of the first mode was 4.18 cycle/time and 4.32 cycle/time in SAP2000 and ABAQUS, respectively, which implies a noticeable agreement in the results of the two models. Figures 4.20 and 4.21 show the first modal shape and its associated frequency for the 4-pin connection model as reported by SAP2000 and ABAQUS, respectively.

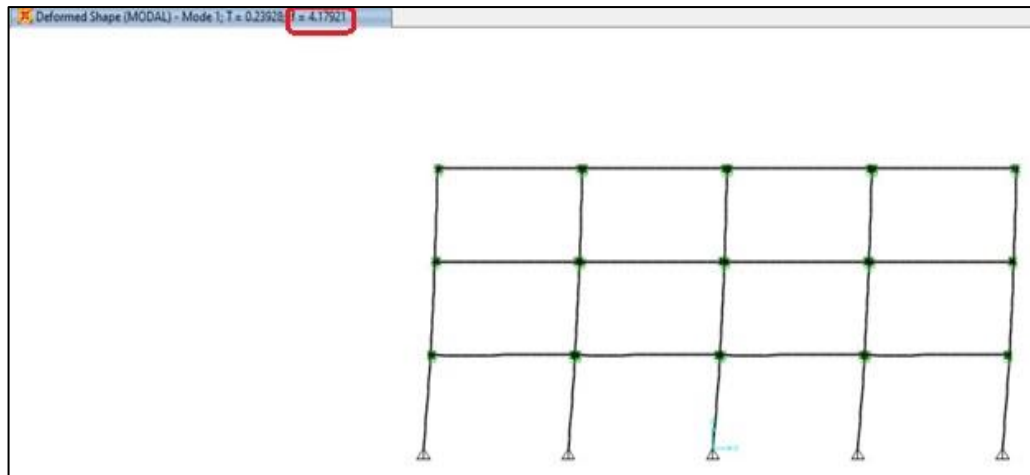


Figure 4.21: The first modal shape and frequency of the 4-pin connection model created in SAP2000.

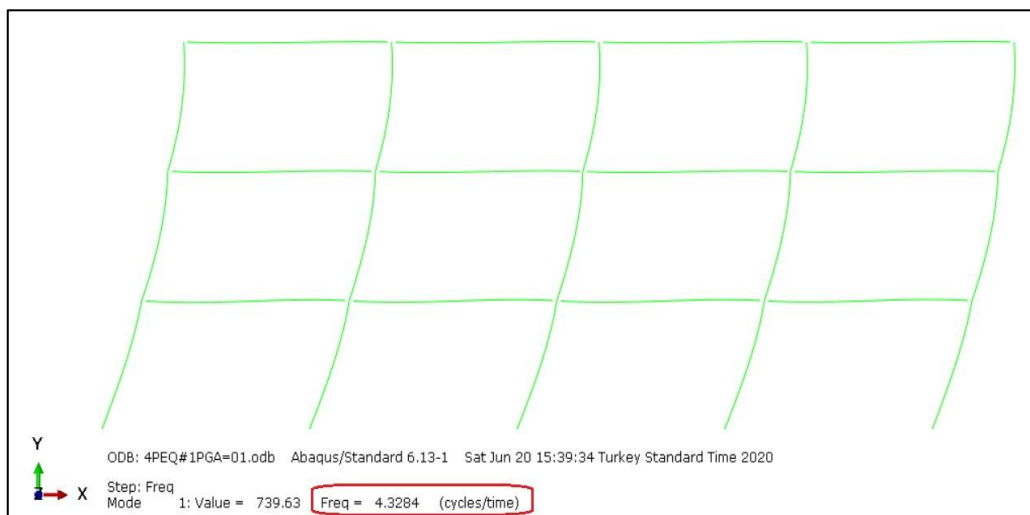


Figure 4.22: The first modal shape and frequency of the 4-pin connection model created in ABAQUS.

The next step involved the application of pushover analysis in both programs (SAP2000 and ABAQUS). Results from both programs were almost the same as can be seen in Figures 4.22 and 4.23, which present the comparison between the results for the cases of the hooked and 4-pin connections, respectively.

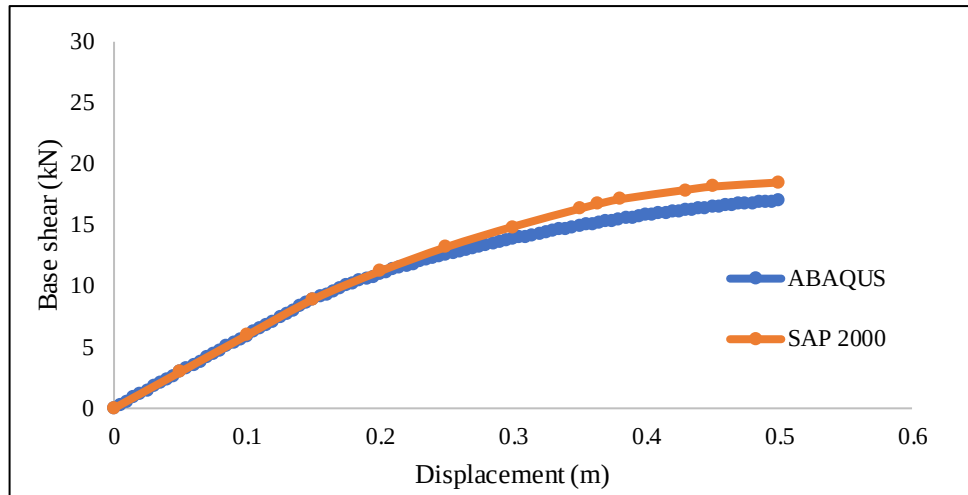


Figure 4.23: Comparison between SAP2000 and ABAQUS results of the pushover analysis for the hooked connection type.

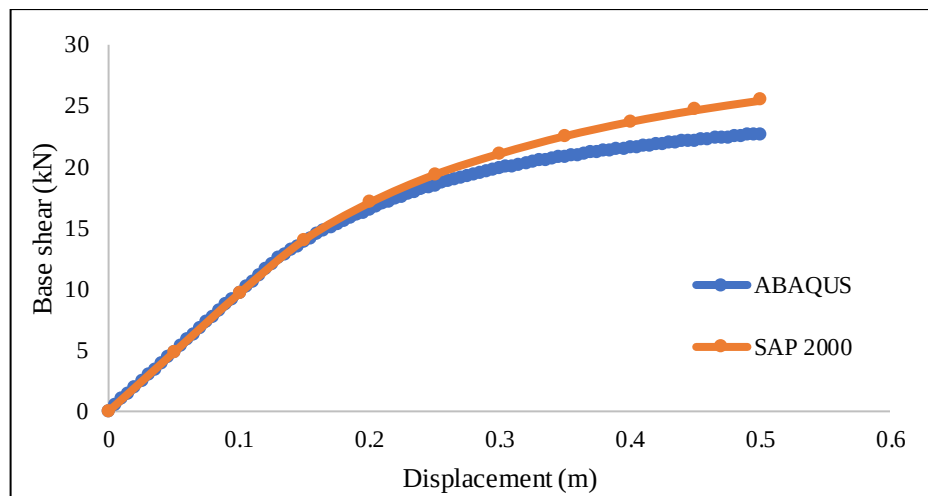


Figure 4.24: Comparison between SAP2000 and ABAQUS results of the pushover analysis for the 4-Pin connection type.

Based on the above results we can start verifying ABAQUS results, but this time with the aid of the experimental test results which were used as an input for the moment-rotation relationship for the two models (hooked and 4-Pin connection). We can conclude from the previous graphs that SAP2000 and ABAQUS models showed almost the same behavior in the linear stage. Once the nonlinear stage started, SAP2000 model yielded higher results than the ABAQUS model. In ABAQUS, the whole behavior had been defined including the inelastic stage. This explains why some portions of the structure in ABAQUS experienced yielding while they did not in SAP2000.

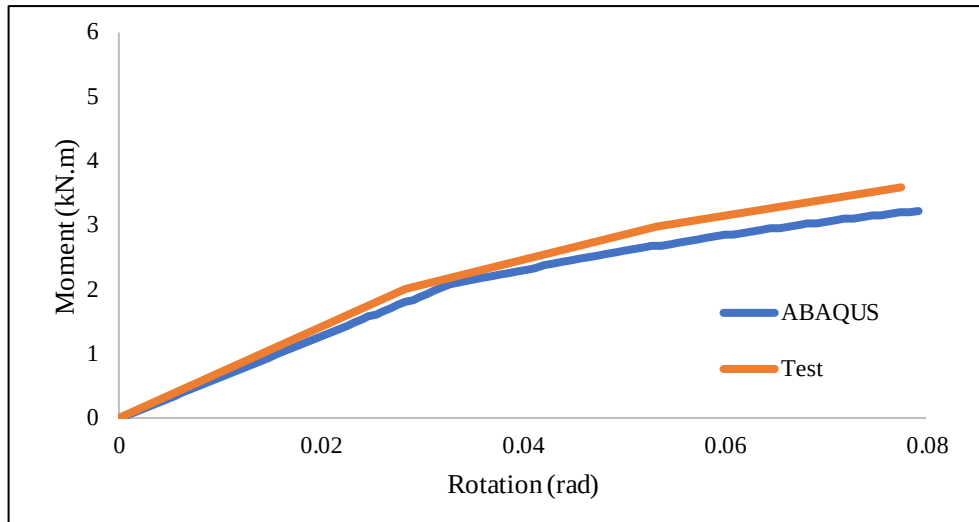


Figure 4.25: Comparing the moment-rotation relationship from the experimental test and ABAQUS analysis for the hooked connection type.

The joint behavior had been also checked and compared with the input data (moment-rotation curve) which had been taken from the experimental test. Figures 4.24 and 4.25 show the moment-rotation relationship for the hooked and 4-pin connections, respectively.

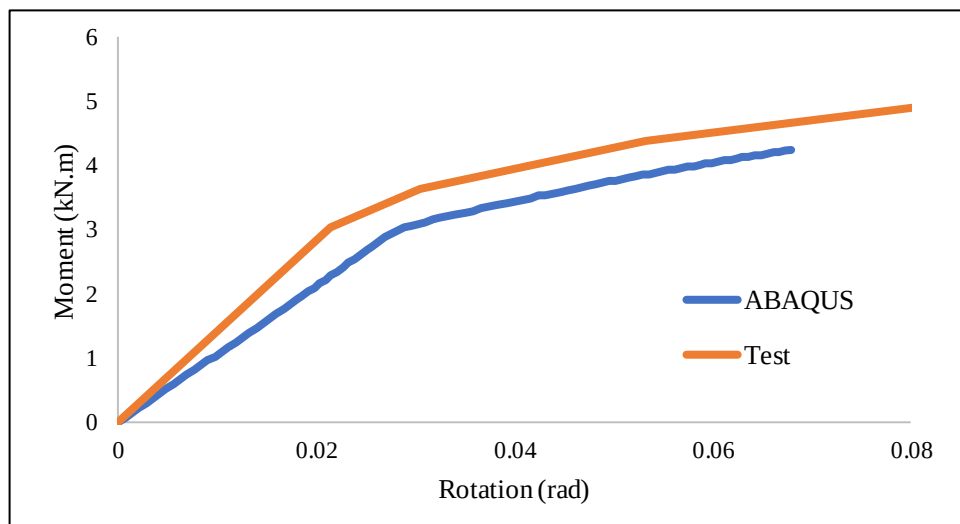


Figure 4.26: the moment-rotation relationship from the experimental test and ABAQUS analysis for the 4-pin connection type.

According to the above results, a very good congruence can be noticed between the results of the ABAQUS models and the experimental tests with a small amount of deviation. This is referred to the fact that a constant axial load of 50 kN was applied to

the columns in the experimental tests. However, axial load was varying in ABAQUS depending on the location of the joint and it even never reached 50 kN. As the axial load in ABAQUS was always lower compared to the experiments, it led to a lower stiffness. With this reasonable justification, ABAQUS model can be considered reliable for conducting next analyses including the incremental data analysis (IDA) and the fragility analysis.

5. SEISMIC ASSESMENT OF RACK SYSTEMS

This chapter presents the parametric study with the model details and types, and the numerous scaled earthquake records used to perform IDA analysis. Subsequent sections introduce the use of Python programming language and its advantage for efficiently handling the huge number of runs that had to be carried out for the two models. Later in this chapter, the results of the IDA and fragility curves are demonstrated in detail.

5.1. Parametric Study (EQ, Rigidity Of Connections)

A four-bay three-story rack frame with constant width and height (as given in Table 1) was taken as the case study. Constant pallet weight value but with two different connection types (hinged and semi-rigid) were analyzed to evaluate collapse prevention in the down-aisle direction under different earthquake records (total of 22 Earthquake records were used). For the pallet weight it was considered uniformly distributed along the beams in all story levels with a value of 2 kN/m.

Two model types were constructed with a difference in the connection type. The first model type implemented the moment-rotation relationship obtained experimentally from the boltless hooked connection model (hinged). For the other model type, the used moment-rotation relationship was the one obtained from the 4-Pin hooked connection model (semi-rigid). For each model the 22 earthquake records with different Peak Ground Acceleration (PGA) were applied one by one. All these ground motions have different properties in terms of the effective period, arias intensity and velocity-to-acceleration ratio (V_{max}/A_{max}). Moreover, for each earthquake record the PGA value was scaled from 0.1 g up to 4.0 g with a step size of 0.1 g (i.e. [0.1, 0.2, 0.3, ..., 4.0]) with a total of 40 NTHA for each earthquake record, which made a total of 880 scaled records that had to be applied to each model. As we have two model types, then the total number of final models that had to be run was 1,760 models. Since each of these models had different earthquake assignments, doing that by hand is very time consuming. From that emerged the need for writing a programmed script using Python language to control the assignment and running processes and to repeat the steps automatically for all models.

In this research a Python code (the code is attached in Appendix 2) was written to do all needed steps, starting from creating the needed model, making all the assumptions, assigning all the needed properties and boundary conditions, and submitting the job for each case. After each run is completed, the Python code extracts the needed results which, in this research, were primarily the drift of each level and the base shear. The code saves these results in specific files for post-processing. After this job is done with all models, the Python code takes the results and performs IDA analysis for both drifts and base shear. The IDA results are used later for performing the fragility analysis. The following sections illustrate the use of these numerous models for performing IDA and fragility analyses.

Table 5.1: Description and input values for the frame used in the case study.

Description	Nomenclature	Value	Unit
Pallet weight	W_{pi}	2	(kN/m)
Clear span of beams	L	2.70	(m)
Clear height of uprights	H	1.50	(m)
Number of bays	N_{bay}	4	null
Number of levels	N_L	3	null
Number of beam-upright connections	N_c	24	null
Material used	A36 steel		
Young's modulus	E	200000	Mpa (N/m ²)
Poisson's ratio	ν	0.3	null
Connection Type	Both hooked and 4-pin connections were modeled		
Boundary conditions	Pin supports at the base		

Table 5.2: Characteristics of earthquake records.

Record ID No.	Horizontal component	PGA (g)	Effective period (sec)	Arias intensity (m/sec)	$V_{\max}/A_{\max}$ (sec)
1	I-ELC180	0.281	24.19	1.55619	0.11239
2	I-ELC270	0.211	24.12	1.16886	0.15143
3	C05085	0.444	6.69	0.86310	0.01000
4	C05355	0.368	7.79	0.86310	0.06230
5	A-ESO090	0.372	10.405	1.57116	0.06230
6	A-ESO180	0.330	8.26	2.00907	0.09506
7	H-CHI012	0.270	20.56	1.17911	0.09372
8	H-CHI282	0.254	24.75	1.19490	0.11970
9	WSM090	0.377	6.865	1.75864	0.11930
10	WSM180	0.499	6.205	1.90176	0.07324
11	G04270	0.224	13.08	0.71612	0.08683
12	G04360	0.349	12.5	0.77285	0.05060
13	B-WSM090	0.173	22.565	0.82619	0.13879
14	B-WSM180	0.211	19.52	1.19774	0.15623
15	ERZ-EW	0.496	7.43	1.78927	0.16067
16	ERZ-NS	0.387	8.675	1.52189	0.28241
17	SHI000	0.225	11.03	0.82635	0.14193
18	SHI090	0.233	12.55	0.63897	0.09528
19	36531-93	0.238	13.595	0.78429	0.14197
20	36531003	0.290	12.67	0.97510	0.16673
21	PRPCS	0.223	21.325	1.31009	0.25733
22	PRPCW	0.198	21.78	1.03357	0.15117

5.2. Analysis Outputs

In this section, the results of the IDA analysis for the drifts and base shear are discussed along with the fragility curves that were created for the two models.

5.2.1. IDA Drift Results

In this section, the IDA analysis is illustrated for the calculation of drifts values. Two earthquake records (ID No. 6 and 20) were studied separately with scaled PGA value of 2.0 g. Despite the similarity in the PGA of both records, their characteristics including the Arias intensity, $V_{\max}/A_{\max}$ ratio, etc are different, the thing that leads to differences in the structural behavior under each earthquake record. Figures 5.1 and 5.2 reveal these differences by showing the drift time histories at each story as a response to the earthquake records No. 20 and 6, respectively, for the case of 4-pin connection model.

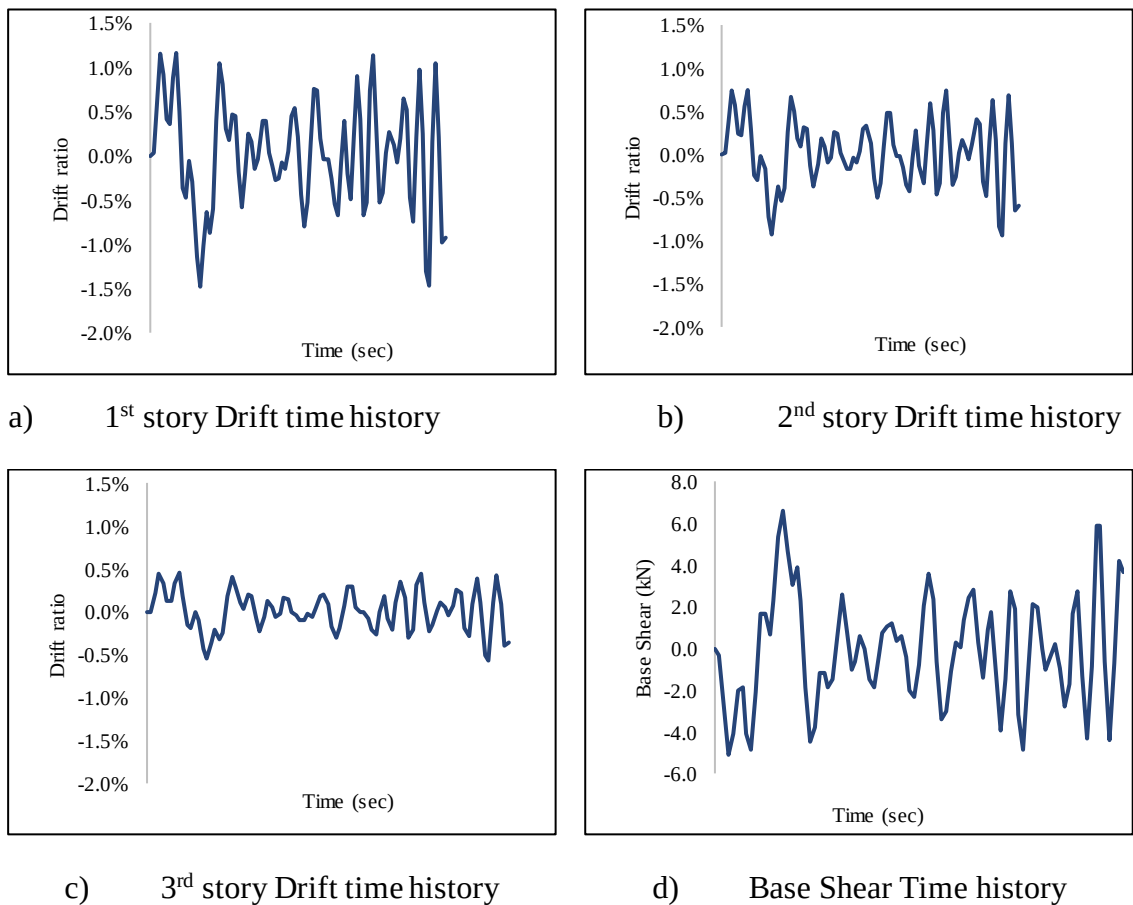


Figure 5.1: Drift and base shear time histories as resulted from earthquake record NO. 20 for the 4-pin-connection type.

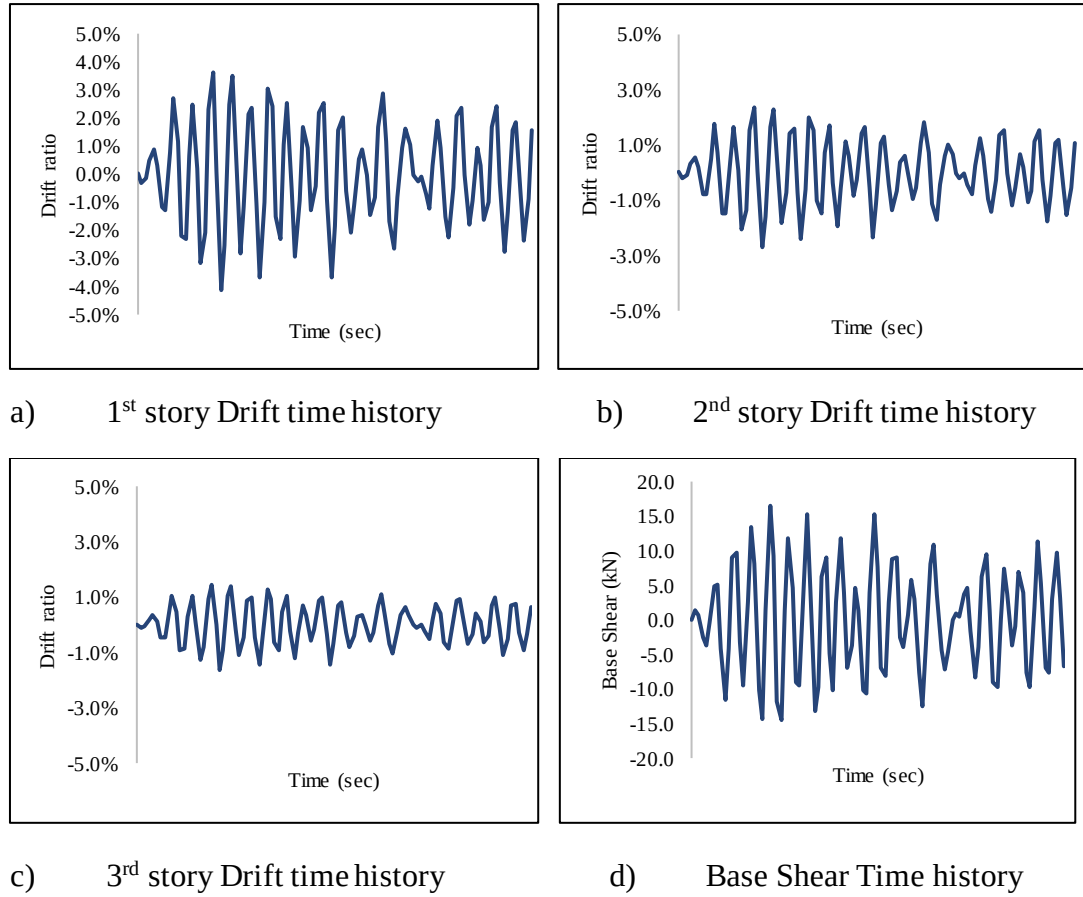


Figure 5.2: Drift and base shear time histories as resulted from earthquake record NO. 6 for the 4-pin-connection type.

As a further support to the point arisen previously, the preceding figures prove that despite the identical PGA scaled value (2.0 g) of both earthquake records No. 20 and 6, the maximum drift response is different. For example, maximum drift ratio at the first story is 4% for earthquake record No. 6, and 1.5% for record No. 20. The same observation holds for the other stories and the base level. That was the reason for choosing to apply several earthquake records. Figure 5.3 shows how the same PGA of two different records cause different responses of the structure. In this figure the IDA curve is created for the first story drift at two different earthquake records. At a constant PGA (2.0 g), a significant difference is clearly observed in the drift of the 1st story, which is dictated by the different characteristics of each earthquake records.

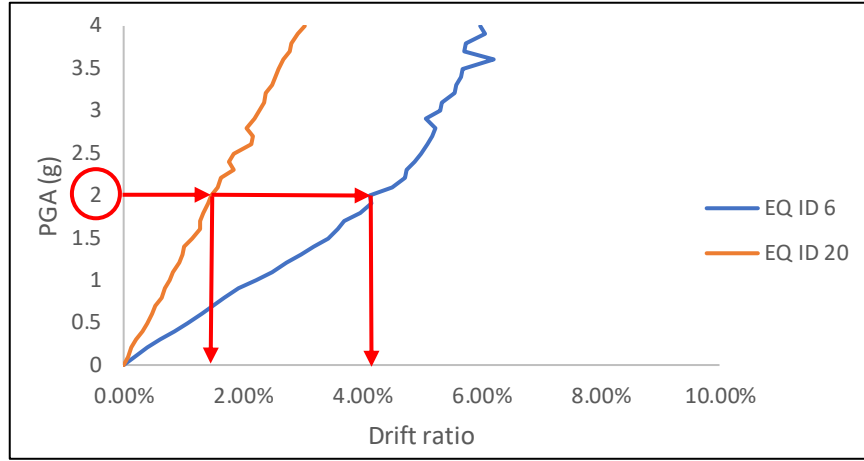


Figure 5.3: IDA curves for the 1st story drift from earthquake records No. 6 and 20 (4-pin-connection model).

Figures 5.4 and 5.5 show the drift time histories at each story as a response to the earthquake records No. 20 and 6, respectively, for the case of hooked-connection model. Figure 5.6 shows the different values of drift ratio of the 1st story at different earthquake records despite having the same PGA scale (2.0 g).

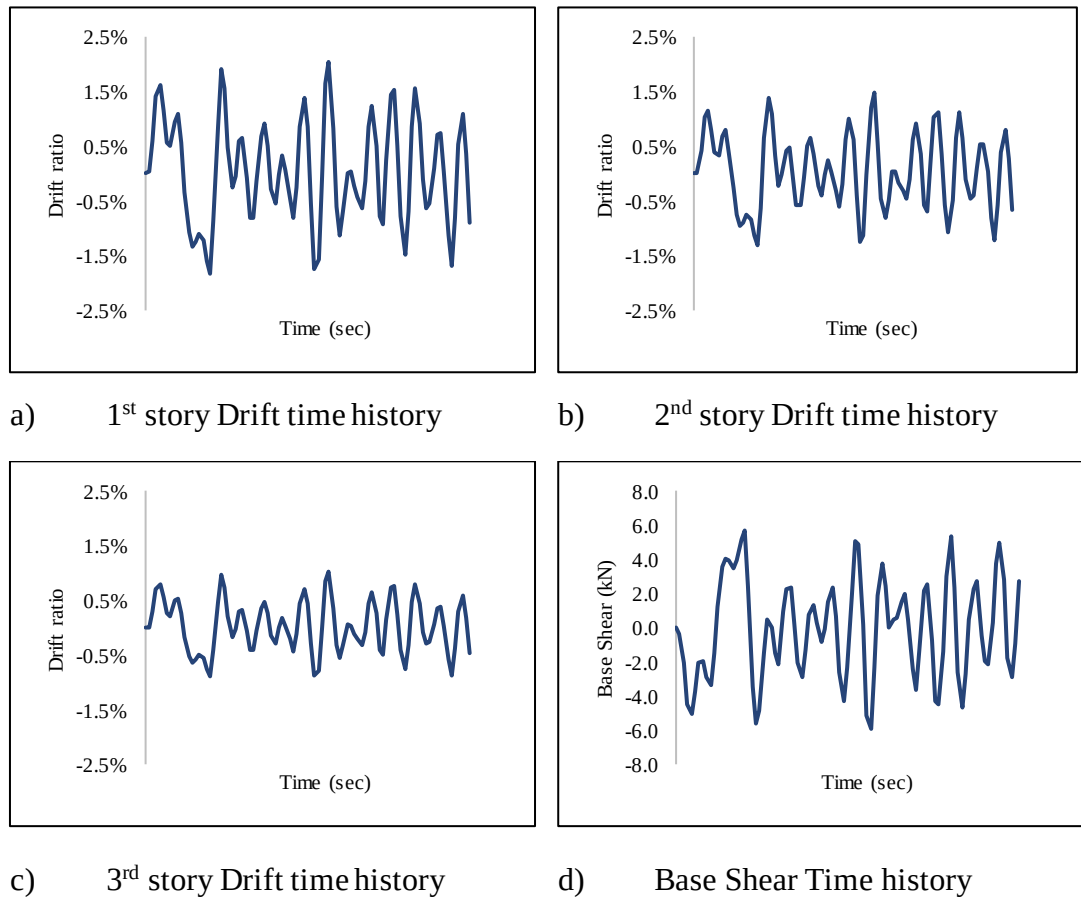


Figure 5.4: Drift and base shear time histories as resulted from earthquake record NO. 20 for the hooked-connection type.

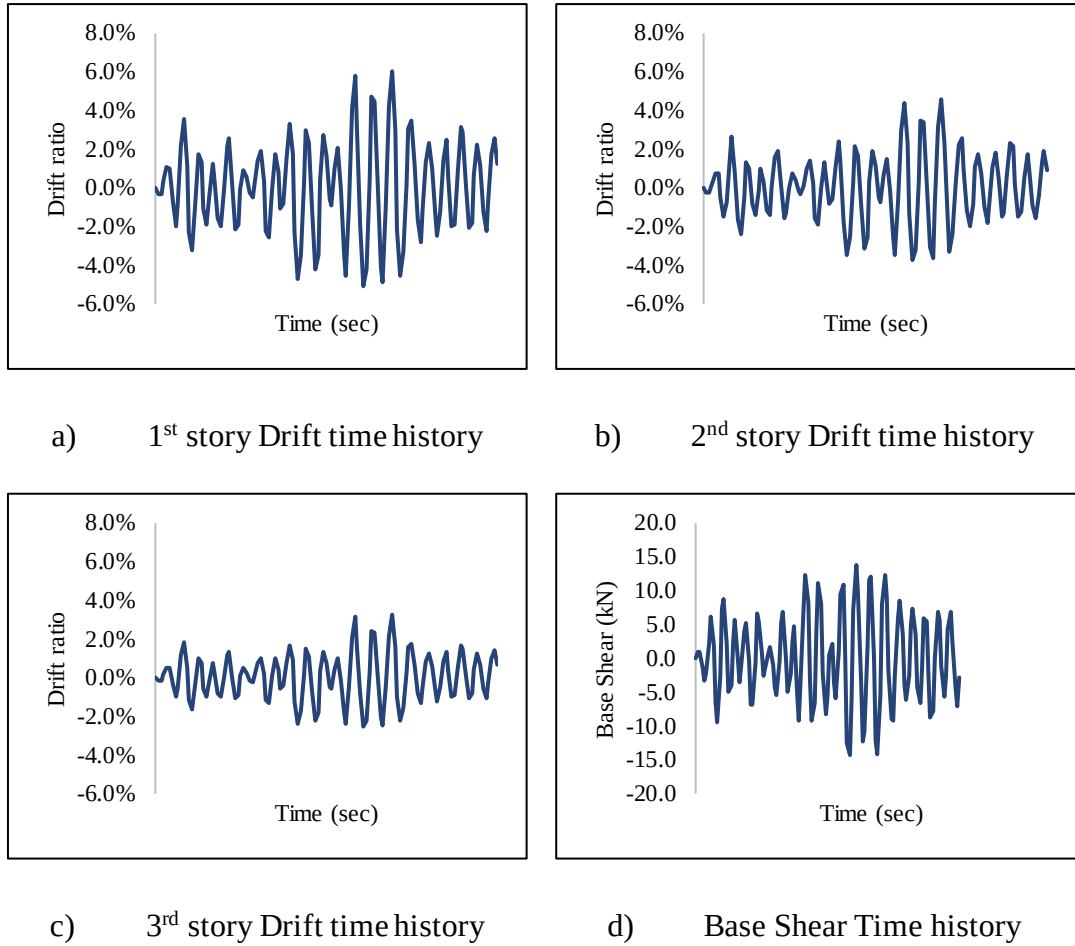


Figure 5.5: Drift and base shear time histories as resulted from earthquake record NO. 6 for the hooked-connection type.

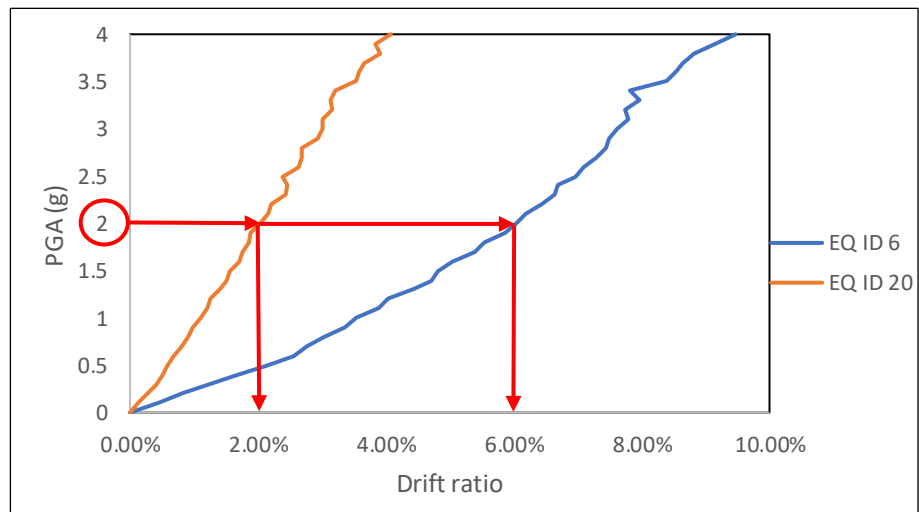


Figure 5.6: IDA curves for the 1st story drift from earthquake records No. 6 and 20 (hooked-connection model).

After the preceding brief demonstration of part of the results, a suite of IDA curves were created for the complete set of earthquake records. The randomness of the IDA curves was obvious under different earthquake records and the discreteness of the response among the IDA curves was imaginable. The fragility curves can be built based on these IDA curves using a statistical approach as illustrated in the coming section.

The IDA curve was established for the maximum drift value which was found at the 1st story level. Figures 5.7 and 5.8 show the IDA curves for the two different types of connections.

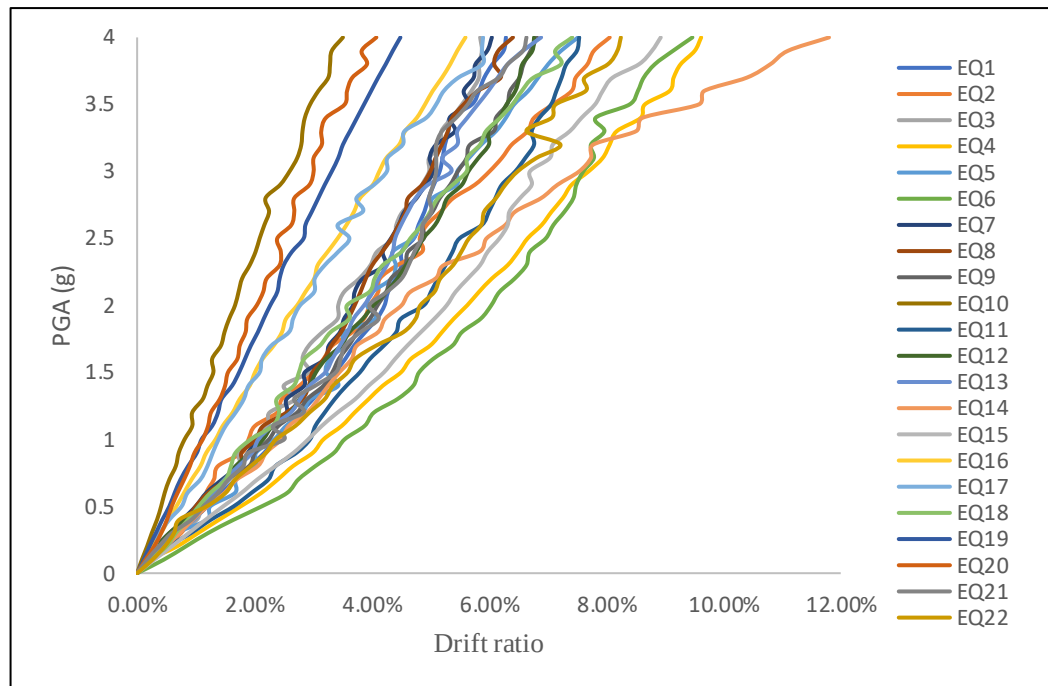


Figure 5.7: IDA curves for the 1st story drift from all earthquake records (hooked-connection model).

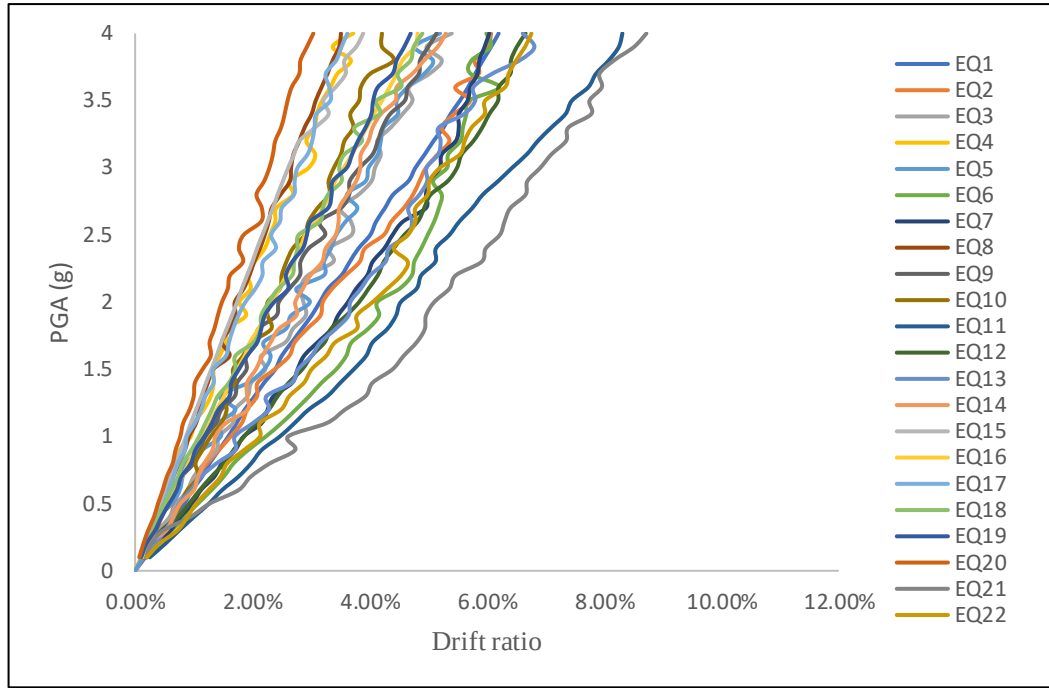


Figure 5.8: IDA curves for the 1st story drift from all earthquake records (4-pin-connection model).

It can be noticed that drift values in the case of 4-pin-connection model is less than in the case of the hooked-connection model. This result is expected because while the connections get stiffer, the overall performance of the structure is affected. This leads to the conclusion that hooked connections behave like hinges while the 4-pin connections behave like rigid connections.

5.2.2. IDA Results for The Base Shear

Similar to the drift IDA curves, the same procedure was followed to establish the IDA curves for the base shear as shown in Figures 5.9 and 5.10.

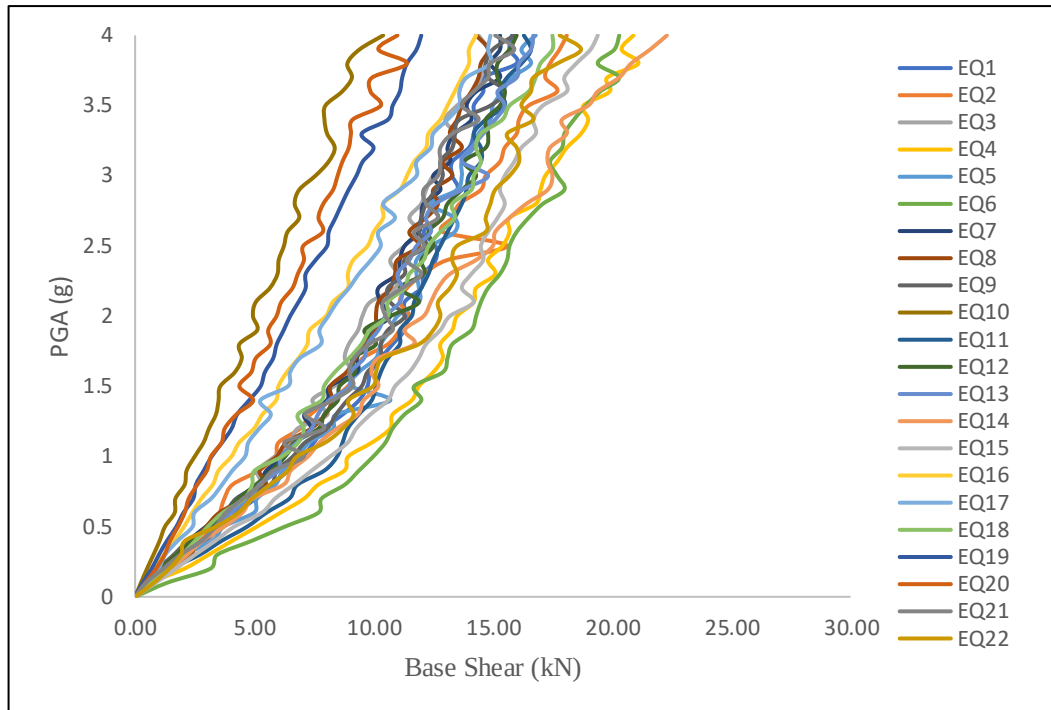


Figure 5.9: IDA curves for the base shear from all earthquake records (hooked-connection model).

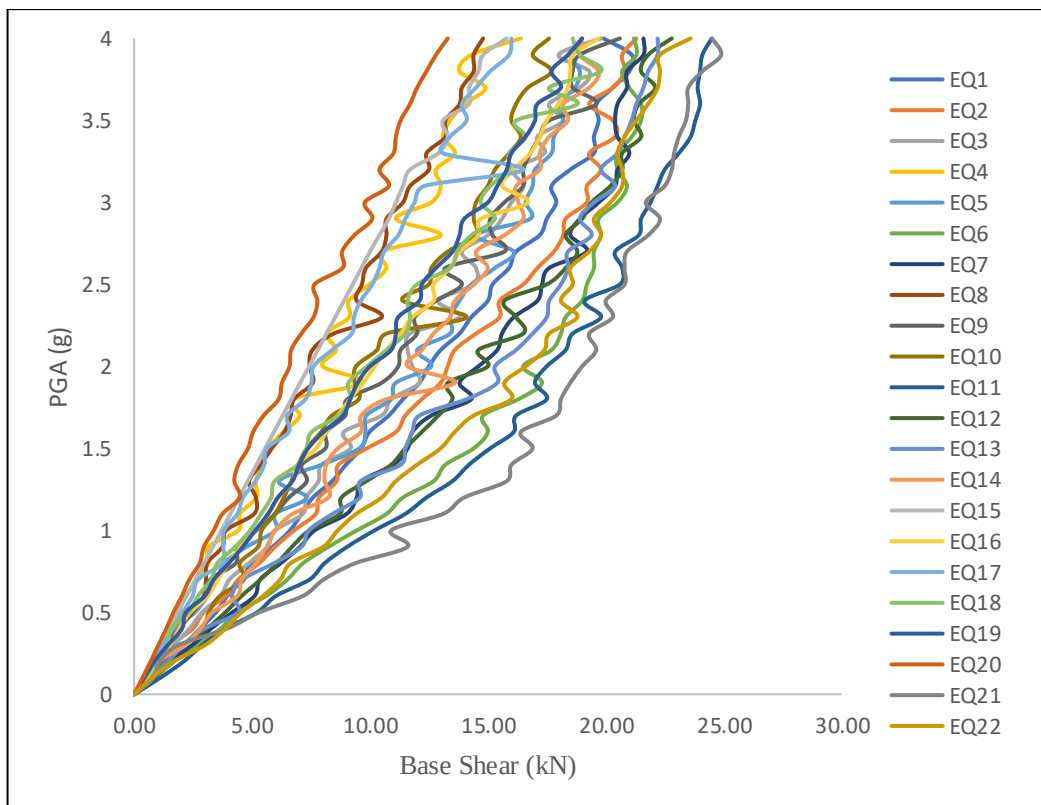


Figure 5.10: IDA curves for the base shear from all earthquake records (4-pin-connection model).

Since the base shear is greater in the 4-pin-connection model, we can say it has more stiffness. This increase in stiffness can be referred to the connection type used as it is the only feature that was changed among the two models. This leads to the same inference that the 4-pin connections can be considered as rigid, while the hooked-connections hinges.

5.3. Fragility Curves

Seismic fragility analysis aims at evaluating the performance of structures under earthquake events and it is an important part of the risk analysis of buildings. For the development of fragility curves, both capacity limits and demand models are needed.

Fragility is defined as a member or system's conditional probability of exceeding a certain limit state under given demand variables and a several researchers have been engaged in developing the fragility models of building systems.

Engineering demand parameters are structural quantities that can be used to predict damage to structural systems. In this study, two different parameters were chosen: 1) maximum inter-story drift and 2) maximum beam-to-column connection rotation. Three limit states were considered: immediate occupancy (IO), life safety (LS) and collapse prevention (CP). Values of these limit states are chosen to be expressed as varying levels of drift ratios. Due to the lack of the capacity guidelines provided by rack systems standards, the capacity values of the engineering demand parameters (EDPs) for each limit state were obtained by performing nonlinear static pushover analysis. According to the results of the moment-rotation relationships and the drift values, the limit state was specified for the immediate occupancy (IO), life safety (LS) and collapse prevention (CP) as 1%, 2% and 4% drift ratio, respectively.

According to the previous limit states, the fragility curves were obtained by the conditional probability of exceeding the prescribed limit states, and the fragility results for both models are shown in Figures 5.11 and 5.12.

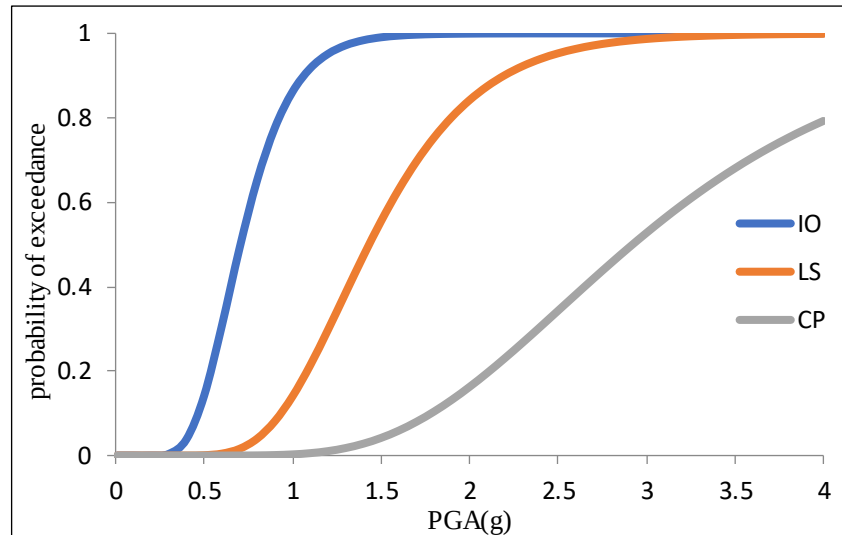


Figure 5.11: Fragility curves for the 4-Pin-connection model.

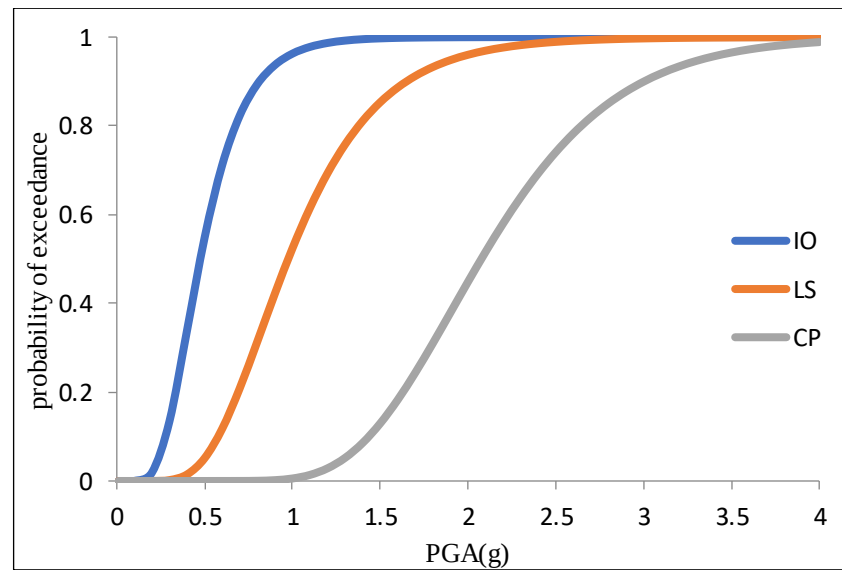


Figure 5. 12: Fragility curves for the hooked-connection model.

The preceding figures show that for the case of 4-pin-connection model at a PGA value of 0.9 g the probability of exceeding the life safety (LS) limit state is about 9% and for the immediate occupancy (IO) about 78%. On the other hand, for the hooked-connection model, and with the same PGA value, it can be noticed that these probabilities of exceedance increase to become around 42% and 93% for the life safety (LS) limit state and the immediate occupancy (IO) limit state, respectively. This concludes that the 4-pin-connection model is stronger and stiffer than the hooked-connection model.

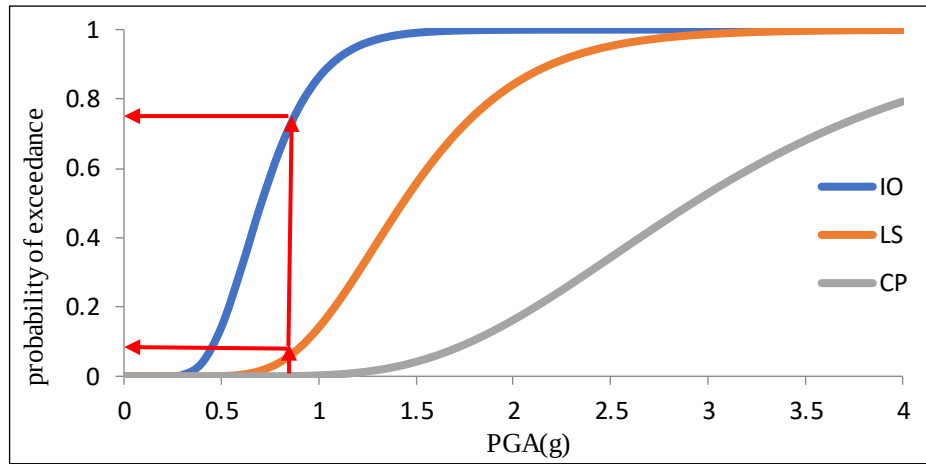


Figure 5.13: Probabilities of exceedance at 0.9 g PGA for the 4-pin-connection model.

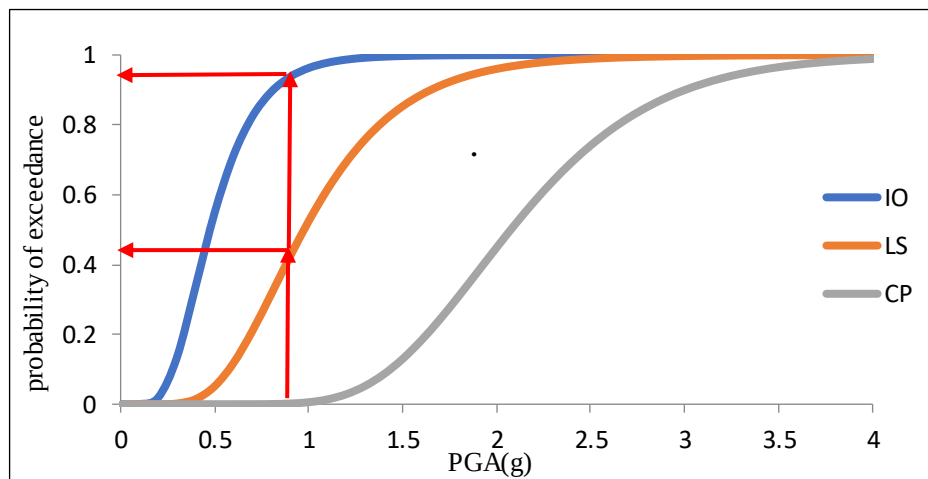


Figure 5.14: Probabilities of exceedance at 0.9 g PGA for the hooked-connection model.

6. CONCLUSIONS AND FUTURE WORK

6.1. Conclusions

Racking systems are indispensable in any warehouse. These systems offer an efficient solution to organize and save spaces. Beam-to-column connections behavior plays an important role in the overall performance of the rack structure under seismic loading. In this research, two models with two different connection types were modelled and verified by using ABAQUS software, hooked connection and 4-pin connection. The seismic behavior of the two types of connection was studied and the following remarks were concluded:

- The 4-pin-connection model is stronger and stiffer than the hooked-connection model. This result was proved through two paths of reasoning. First, the IDA showed that the base shear was greater in the 4-pin-connection model compared to the base shear in the hooked-connection model even at the same PGA, which indicates higher rigidity of the 4-pin connections. Second, from the fragility curves, the 4-pin-connection models experienced less probability of exceeding the specified capacity limit states, leading to the inference that the 4-pin connections provide higher strength and stiffness to the whole structure.
- Even if the earthquake records have the same PGA values, that does not necessarily mean identical response of the structure. There are some other intervening factors such as the Arias intensity, $V_{\max}/A_{\max}$ ratio, etc.

6.2. Future Work

The presented work at hand can be extended for future research in the following tracks:

- Investigating more realistic joint models that are built in ABAQUS or any other simulation environment in a way very close to how it is synthesized in the actual world, and thus leading to more realistic results.
- Extending the ABAQUS 2D model to a 3D model to take into account all possible interactions from all structural components.
- Doing full-scale experimental tests for complete rack units for for actual investigation and verification purposes.

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BIOGRAPHY

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APPENDICES

APPEDIX A: The Time history graphs of the earthquake records

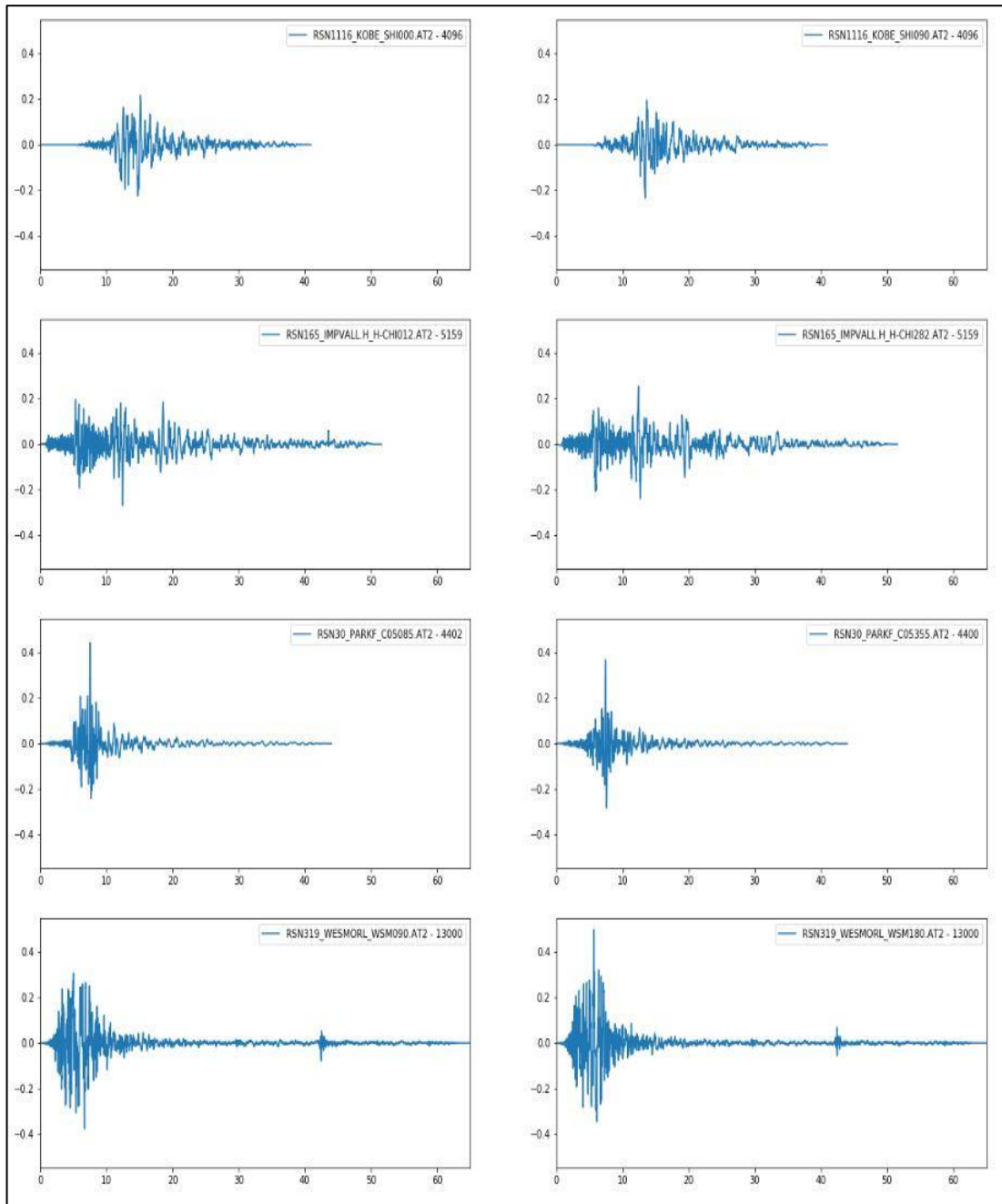


Figure A1.1: The time history graph for first 8 earthquake records.

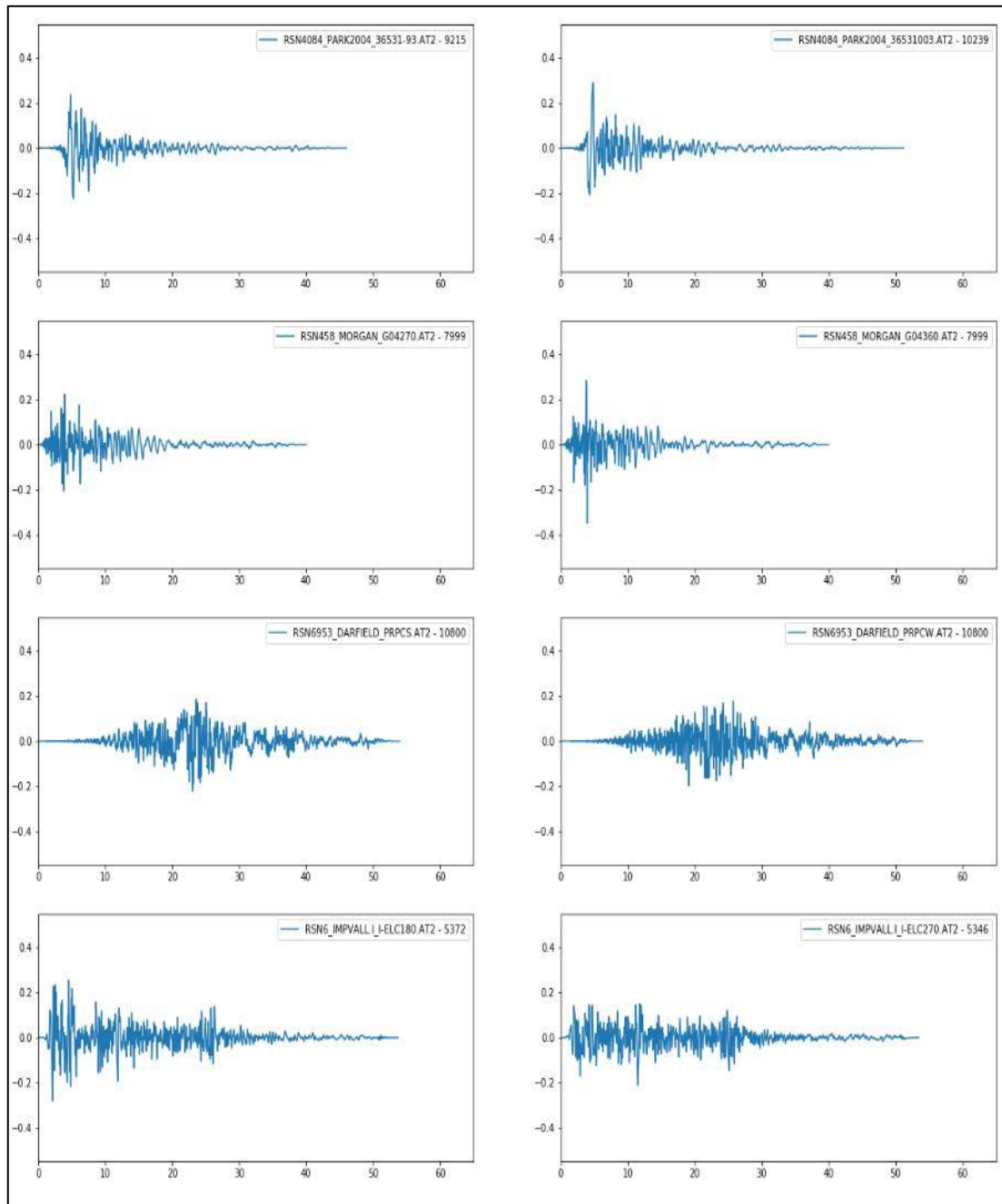


Figure A1.2: The time history graph for second 8 earthquake records.

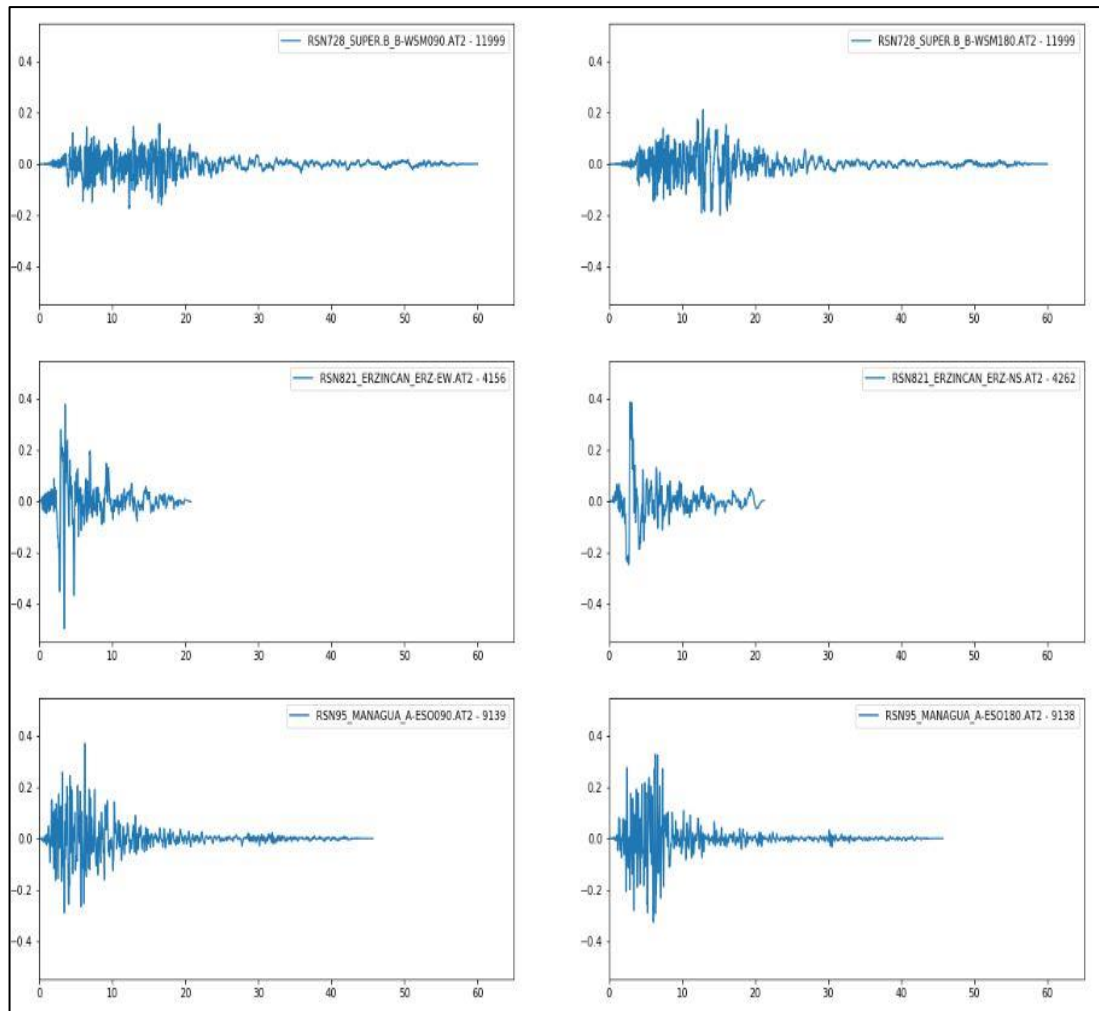


Figure A1.3: The time history graph for last 6 earthquake records.