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Faculty of Graduate Studies

Effects of Structural Links Between Building and Ground Cut on its Seismic Performance

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This thesis was defended successfully on 14/10/2020 and approved by:

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Dedication

To our first and only perfect teacher who had laid out the groundwork for the spiritual education of mankind, our prophet Mohammed Peace be upon him.

To the soul of my grandmother AMINA

To my father

To my mother

To my brothers

To my sisters

To my precious ones

To all friends and colleagues

To my teachers

To everyone working in this field

To all of them

I present this thesis.

Acknowledgment

First of all, praise is to Allah for helping me making this research possible.

I will never forget to express my great appreciation to my teachers for their generous support which they offered to me along through the whole period of my study.

My special gratitude appreciation is directed towards my supervisors:

Dr. Abdul Razzaq A. Touqan.

Dr. Monther Diab.

To all the teaching staff, teachers and supervisors.

To the great center of science; An-Najah National University.

Besides, everyone who contributed in completing this research.

الإقرار

أنا الموقع أدناه، مقدم الرسالة التي تحمل عنوان:

Effects of Structural Links Between Building and Ground Cut on its Seismic Performance

أقر بأن ما اشتملت عليه هذه الرسالة إنما هو نتاج جهدي الخاص، باستثناء ما تمت الاشارة اليه حيثما ورد، وأن هذه الرسالة ككل، أو أي جزء منها لم يقدم من قبل لنيل أي درجة علمية أو بحث علمي أو بحثي لأي مؤسسة علمية أو بحثية أخرى.

Declaration

The work provided in this thesis, unless otherwise referenced, is the researcher's own work, and has not been submitted elsewhere for any other degree or qualification.

Student's Name: عبد الحميد نواف يونس قدادحة **اسم الطالب:**

Signature: **التوقيع:**

Date: 14/10/2020 **التاريخ:**

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List of Abbreviations

***ACI*:** American Concrete Institute

ASCE: American Society of Civil Engineers

SEI: Structural Engineering Institute

FEM: Finite element method

IBC: International Building Code

LFRS: Lateral force-resisting system

MRF: Moment resisting frame

MRS: Modal response spectrum

PGA: Peak ground acceleration

RC: Reinforced concrete

RH: Response history

SDC: Seismic Design Category

SMRF: Special moment resisting frame

***SRSS*:** Square root of the sum of squares

***SW*:** Shear wall

***UBC*:** Uniform Building Code

SDOF: Single Degree of Freedom

2DOF : Two degrees of Freedom

3DOF : Three Degrees of Freedom

MDOF: Multi Degrees of Freedom

PBEE: Performance – Based Earthquake Engineering

FRF: Frequency Response Function

CSM: Capacity Spectrum Method

MPA: Modal Pushover Analysis.

SEAOC: Structural Engineers Association of California.

SFSI: Structure–Foundation–Soil Interaction.

LIST OF SYMBOLS

: Modulus of elasticity of concrete

: Vertical seismic load effect

: Moment of inertia of gross section of beam about neutral axis

I : Moment of inertia of cracked section transformed to concrete

: Earthquake importance factor

: Modal mass of the nth-mode

: Seismic effect of orthogonal loading

: Response modification factor

S_1 : 5% damped, dimensionless coefficient of one second period horizontal spectral acceleration for rock

S_{D1} : 5% damped, design spectral response acceleration coefficient at long period for deterministic site

S : 5% damped, design spectral response acceleration coefficient at short period for deterministic site

S_{M1} : 5% damped, spectral response acceleration coefficient at long period for deterministic site

S : 5% damped, spectral response acceleration coefficient at short period for deterministic site

: 5% damped, dimensionless coefficient of short time period horizontal spectral acceleration for rock

(g) : Maximum spectral response acceleration

: Natural period of vibration

: Seismic zone factor

$f_{c'}$: Compressive strength of concrete

: Yield strength of steel

: Standard acceleration due to gravity (9.81/ms^2)

: Damping ratio

: Modal participation factor of an nth-mode

[Φ] : Modal matrix

[m] : Mass matrix

[K] : Stiffness matrix

[ι] : Influence vector

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Abstract

The key figures in ensuring safe seismic design are the seismologist and the structural engineer. However, in common practice, the architect initiates the building design and determines a number of issues relating to its configuration that have a major influence on the building’s seismic performance.

Configuration is defined as the building’s size and three-dimensional shape, the form and location of the structural elements, the connection between building and the adjacent structures or ground, and the nature and location of structural and nonstructural components that usually affect seismic performance.

This research sheds light on buildings that have such configurations that prevent them perform freely due to lateral loads.

Buildings under concern have a link to adjacent ground from one side, which restricts building’s movement in link’s side while it’s still free to drift in the opposite side.

For the complexity and uniqueness of each building model, many factors affect its seismic performance, and thus should be studied individually.

To develop a clear vision of such buildings performance under seismic loads, a study of simple frames analyzed under different type of loadings and compared to the unrestricted case using SAP2000 software were done as a main source in this study, after verifying the results by comparing them to theoretical equations of harmonic excitation, and hand calculations of period using Rayleigh's method.

For the purpose of this study, two measurements approaches applied to all possible configurations of link location for single, two and three degrees of freedoms models:

- Response spectrum approach: a multi periodic harmonic sine wave containing periods from 0.1sec to 2sec with steps of 0.1 sec with same amplitudes applied to each model of the same configuration with different natural periods, where 0.1sec< T_n <1sec.

The displacement and relative displacement results has been shown in graphs with respect to excitation period.

It has been shown that for certain natural periods the linked structure may have a larger response for a certain ground motion.

- Frequency-response approach: excitations periods from 0.1sec to 1.6sec applied individually to models with determined unrestricted natural period.

The displacement and relative displacement graphs illustrate the change in natural period and the maximum response of restricted and unrestricted models could be compared.

The percentage of exceedance in relative displacement between restricted and unrestricted models illustrated in normalized graphs with ratio of T/T_n.

The model is validated using available published test data, then the models used to conduct a parametric study on the key factors that affects nonlinear behavior of linked structures.

Results are used to develop simple conceptual graphs to predict the effect of such links on the building's seismic performance.

After conducting this study, it has been found that the links with ground cut have a major effect on the fundamental period and on the lateral stiffness of frame structures. Results from this study suggest that in some cases, designing frames using story shear strength patterns based on unrestricted vibration may not be the conservative to mitigate the occurrence and/or the extent of damage in frames that experience considerable levels of inelastic deformation as a result of vibration restrictions.

Chapter One
Introduction

1.1 General

Inspection and analysis of earthquake-damaged buildings play important roles in understanding the effectiveness of seismic design and construction. Although earthquake damage often appears random (one building may survive while its immediate neighbor will collapse), there are, in fact, patterns of damage that relate to the characteristics of the site and to the building's characteristics.

To develop an effective seismic design, the architect and engineer must work together from the inception of the project so that seismic issues and architectural requirements can be considered and matched at every stage of the design process. For this process to be successful, the architect and engineer must have mutual understanding of the basic principles of their disciplines. Hence, the architect should have a basic understanding of the principles of seismic design so that they will influence the initial design concepts, enabling the engineer and architect to work together in a meaningful way, using a language that both understand. In turn, the engineer must understand and respect the functional and aesthetic context within which the architect works. (FEMA454, 2006)

Pounding is a phenomenon, in which two buildings or building and its surrounding ground strike due to their lateral movements induced by lateral forces, earthquake is one of the major causes for lateral forces on the buildings. An efficient and durable structural design is always required to prevent pounding effect. The simplest method to avoid pounding damage is to provide enough separation gaps. On the other hand, pounding can be reduced by decreasing lateral motion by means of lateral load resisting structural systems, such as special moment resisting frame, shear wall, dual system.

For designing a new structure, connection details and support conditions shall be made as close to the computational models as possible. For an existing structure evaluation, structures shall be modeled as close to the actual as-built structural conditions as possible.

1.2 Problem statement

Due to the mountain terrain in some areas of Palestine, some building's sites have very steep slope which leads to a high cut in ground to establish a building on it. one of the most common issues that a land parcel located between two streets at different levels, so the building has an entrance at the higher-level street which somehow create a link between building and the ground at one or more levels from one side of building, or connection needed to minimize the structural section of nearby retaining wall as shown in fig (1.1).

These links might affect the seismic design of the structure because it makes the building free to drift in one side, and partly restricted in the opposite side at the levels connected to ground cut.

Therefore, in this study, connection between buildings and ground for different cases were investigated and analyzed

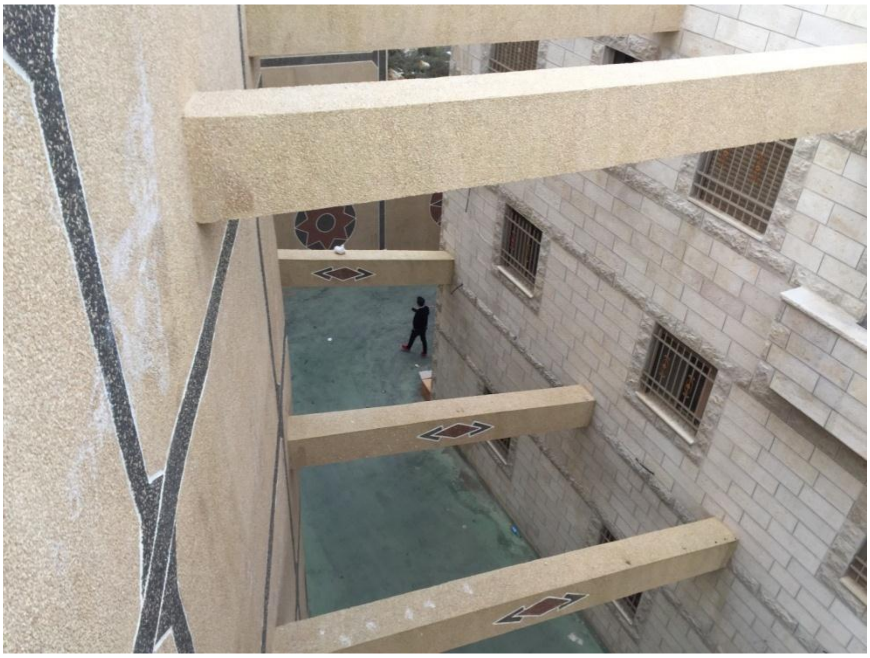


Figure (1. 1): Building connected to ground (Birzeit/Palestine).

Modern seismic codes propose a large enough separation between building and surroundings above the level of the lower portion of structure completely underground, which appears to be ineffective in many cases. Because of the insufficient separations, structural interaction can occur between building and its surrounding during strong ground motions. (Shakya, 2006).

Some of the building codes such as (IBC, 2003) have provided a clause for sufficient separation between adjacent buildings in order to avoid seismic pounding (figures 1.2 and 1.3). However, the provision has been removed from (IBC, 2009) due to constraints in availability of land and to fulfill functional requirements.

1620.4.5 Building separations. All structures shall be separated from adjoining structures. Separations shall allow for the displacement δ_M. Adjacent buildings on the same property shall be separated by at least δ_{MT} where

$$\delta_{MT} = \sqrt{(\delta_{M1})^2 + (\delta_{M2})^2} \qquad \textbf{(Equation 16-64)}$$

and δ_{M1} and δ_{M2} are the displacements of the adjacent buildings.

When a structure adjoins a property line not common to a public way, that structure shall also be set back from the property line by at least the displacement, δ_M, of that structure.

Exception: Smaller separations or property line setbacks shall be permitted when justified by rational analyses based on maximum expected ground motions.

Figure (1. 2): IBC 2003 statement for building separation.

ISOLATION JOINT. A separation between adjoining parts of a concrete structure, usually a vertical plane, at a designed location such as to interfere least with performance of the structure, yet to allow relative movement in three directions and avoid formation of cracks elsewhere in the concrete and through which all or part of the bonded reinforcement is interrupted.

Figure (1.3): IBC 2003 statement for isolation joint

In most of the seismic pounding analysis the effects of underlying and surrounding soil are ignored. The consideration of soil adds extra degrees of freedom at the multi levels and also allows energy dissipation. Hence, it is necessary to include effects of soil on the seismic pounding analysis of buildings. (Shakya, 2006)

At local stage, different ways used to connect the building to the ground will be discussed and analyzed here:

1. **The cut side is soft rock or backfill**: and there are two main systems used to deal with this case:

A. The earth retaining wall is part of the building, which means it carries load from slab system and contribute in lateral stiffness.

Figure (1.4): Earth retaining wall is part of the building.

B. The earth retaining wall is partly separated from building by a void, but it connected to the building at slabs levels, so links create a lateral support to the wall as beam columns elements carrying their self-weight and axial load from earth pressure. In this case connections can resist compression and tension, and the uppermost link mainly is a slab connecting the building to the street.

The instant axial stiffness for these links varies depending on compression or tension phase and the modified cracked section properties.

For simplifying analysis, in this study the active soil pressure neglected, and the tension stiffness assumed to be zero.

Figure (1.5): Earth retaining wall is partly separated from the building.

2. **Links to hard rock cut**: the connection needed at one level only (higher street level), there is no need to earth retaining structure because the cut is stable free standing.

Here, the connection can move with the building free side and restricted from moving cut side, so it simply modeled by (one-way link, compression only)

Figure (1.6): Cut is stable free standing.

1.3 Assumptions

For the three cases discussed in the previous section, and from the inspection of similar cases in practice, some of these practices are not acceptable from dynamics point of view, and they should be avoided by engineering sense, others could be modeled to be studied in this research:

1. If the side ground is stable due ground shaking, where its rock or stable rock boulders, then an imaginary gap link presents the compression forces acting on the building when moving towards ground cut, and its stiffness equal the axial stiffness of the beams or slabs connecting the building to the side cladding system or wall as shown in fig (1.7).

Figure (1. 7): buildings connected to stable ground cut.

Its considerable here to note that the cladding system should be modeled as it, and it contribute in the lateral stiffness of the structure.

The following figures will illustrate the model of the structure with link.

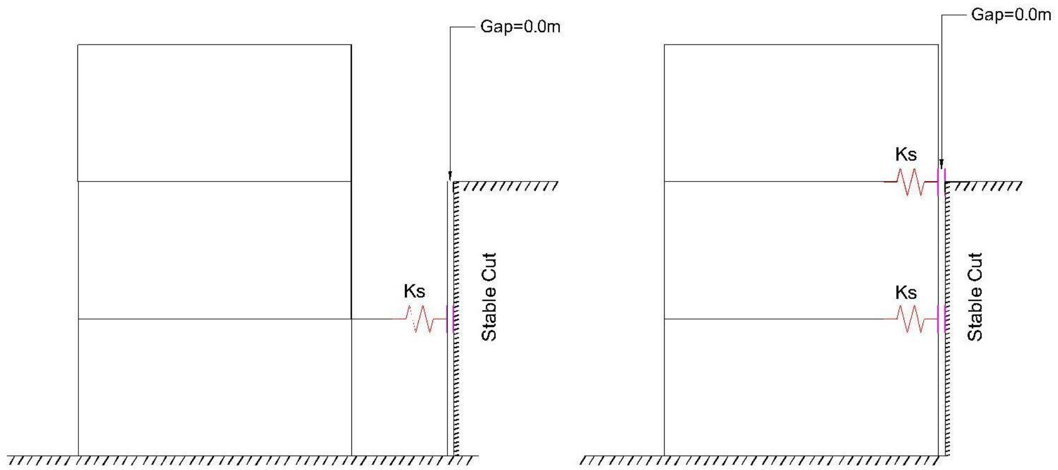


Figure (1. 8): model of Structure with link.

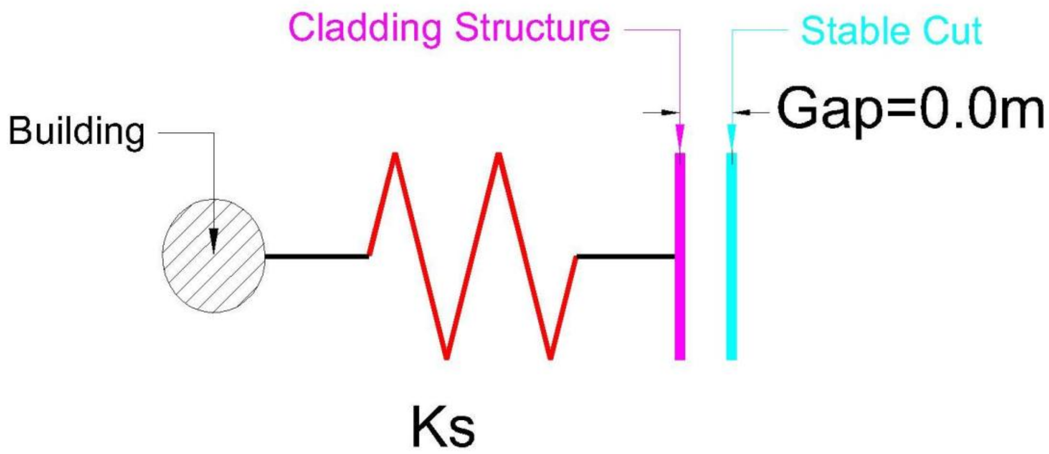


Figure (1. 9): model of Gap link.

Stiffness of link acting in parallel with building's stiffness.

For initializing premier understanding of this issue, the basic assumption here is the connection between ground cut and structure is active in compression only and doesn't affect the structure in tension phase. So, the side ground assumed to have the same base ground movement to ensure the validity of the simplified model.

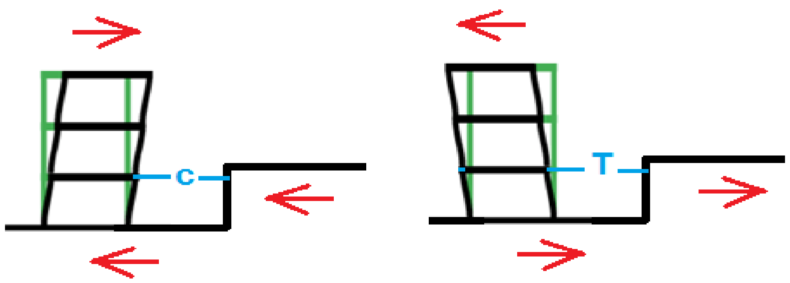


Figure (1. 10): Building relative displacement due ground's movement.

As shown in the fig (1.10) above that when the side ground has same movement with base ground in the opposite of building side, the building will move away with no effect on structure, and when it moves in the building side with same magnitude, the building moves towards side ground with relative displacement related to structure and link stiffness's only.

The structure assumed to interact with ground gently, and the influence of magnification of forces due to classical physics impact analysis is not considered.

2. If the backfill behind the retaining structure is unstable, and it is probable to move towards the building due to its drift, then the active earth pressure should be taken into account and the building may don't return to its original vertical alignment before ground shaking, so the building could suffer additional forces the rest of its life due to its self-weight and active pressure.

This case of clear vulnerability should be avoided in future buildings design, and the assessment of existing buildings having these configurations could be studied later after introducing the simple case.

1.4 Research Objectives

1.4.1 Research Overall Objective

Due to local regulations and functional requirements of certain buildings sites, a link made between building and natural ground which clearly affect the structural performance of structure and not considered in the structural design.

The main concern of this study is to investigate the effect of connection between building and ground cut at levels which constraint the building in one direction and let it free to move in the other direction.

This study deals with the seismic analysis of this case considering link, and compares it to the analysis of the same structure performs freely neglecting lateral motion restrictions, as per common practice.

By knowing the unlikely effects of these configurations, looking to find a design procedure of such buildings, or make provisions to avoid unlikely structural damage.

1.4.2 Research Sub-objectives

Many basic related concepts will be checked and compared with the conventional method used to deal with this type of structures:

1. Using the mode shapes of the building which computed assuming free vibration, in nonlinear analysis of the structure considering changing in stiffness and period due to combining with link.

2. Showing how the natural period of structure changes with respect to link stiffness.

3. Comparing the collision forces and duration of collision using classical physics and dynamics equations.

4. Finding the effect of link in the displacement and relative displacement of the structure in the two directions comparing with free vibration.

The main two assumptions here are:

- The building still performs in its natural mode shapes.

- The model of structure's interaction will be used in the analysis is a link with stiffness "K" and strain "u" depends on combined stiffness.

5. Identifying vulnerabilities of vertical structural elements and how the lateral stiffness of diaphragm system contributes in this issue, and also what are the restrictions about light slab systems used lately in Palestinian market, or the alignment of ribs slabs.

6. Using time history analysis to show the effect of these links to structure comparing with unrestricted same structure with respect to time over natural period ratio.

1.5 Conceptual Identifying of Vulnerabilities

Structure is subjected to one-sided impact, and as a rule, experience response amplifications that can be quite substantial.

At first view, pounding may sometimes reduce the overall structural response and thus be considered beneficial in such cases, more often it will amplify the response significantly. Especially for the above free floors and the free floors below the link.

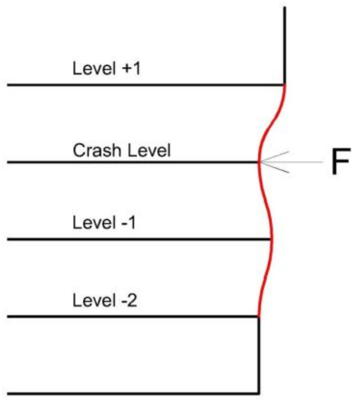


Figure (1. 11): Expected additional relative drift due collision.

Also takes into account the local damage for slab that is almost always caused as a result of pounding.

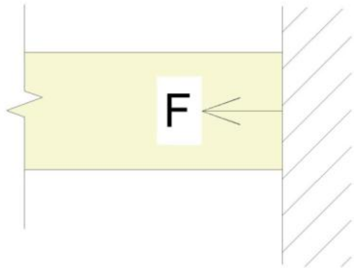


Figure (1. 12): Axial force in diaphragm due collision.

Pounding causes similar effects on elastic and on inelastic structures. The consequences, however, for inelastic structures will normally be more serious.

1.6 Research Scope and Limitations

To understand the effect of the connection to frame buildings, the performance of a frame which is free to vibrate under lateral load will be compared with the same frame with a link restricts its movement in one direction.

For this purpose, starting with a verified model of a frame with computed mass and stiffness, then trace its displacement to determine the maximum drift in positive and negative X-Direction.

To develop the understanding gradually, needing to start with a well-known load's type behavior applied to a simple frame could be checked manually.

Three types of loads will be applied to three levels of frames complication:

1. Single degree of freedom (SDOF) frame:

- Periodic harmonic sine excitation.
- Multi periodic harmonic sine excitation.
- Real earthquake time history analysis.

2. Two degrees of freedom (2DOF) frame:

- Periodic harmonic sine excitation.
- Multi periodic harmonic sine excitation.

3. Three degrees of freedom (3DOF) frame:

- Periodic harmonic sine excitation.
- Multi periodic harmonic sine excitation.

Different cases for each level will be studied and compared to the same frame without restrictions.

For such configuration, many factors affect the overall performance of the structure: link's stiffness, exact model of link's connection to ground, nonlinearity of materials as a result of hysteresis load cycles, and the side ground interaction with structure.

Active earth pressure and the effect of final position of earth retaining structure are not considered in this study.

The impact of earthquakes to the structures in concern is not limited to the factors which will be studied here, other effects such as vertical component of excitation and stability of side ground, are not taken in consideration throughout this study. These are advanced topics, hopefully we can introduce to them for later researches.

1.7 Structure of the Thesis

This research thesis consists of ten chapters; the followings are the summary of the contents of the chapters:

Chapter One (Introduction).

This chapter sets the problem statement, assumptions, research questions, research objectives as well as research scope and limitations.

Chapter Two (Literature review).

This chapter gives a brief introduction to the basics of seismic analysis, vibration, types of excitations, and structural special behaviors. Methods of analysis are illustrated and the performance levels are explained. After that, structural analysis procedures and their major limitations are outlined

This chapter also contains a literature review on similar phenomena of bounding effect, and finally, the overcome of these studies.

Chapter Three (Physical insight)

This chapter presents the physical side of the problem and analyzes the effect of link physically to make initial understanding and prediction of model results.

Chapter Four (Structural modeling)

This chapter summarizes the guidelines and principles for structural analysis and modeling used for this study.

Chapter Five (Model verification)

Analysis results obtained from the computer aided analysis software (SAP2000) are verified thorough a series of hand calculation procedures.

In this chapter, the behavior of link will be verified by verifying the results of the single degree of freedom model, and then a check to fundamental modes will be done to the multi degrees of freedom models.

Chapter Six (Structural analysis).

For the purpose of this study, two measurements approaches (Response spectrum approach and Frequency-response approach) will be applied to all possible configurations for link location for single, two and three degrees of freedoms models. In this chapter, a series of frames models would be analyzed in the two approaches.

Chapter Seven (Normalization of results).

In this chapter a process of normalization will be done to make variables comparable to each other. In this study the reference of results is the unrestricted frame analysis, so the plots of relative displacements of each case in the previous chapter that computed based on frequency response method will be normalized as a ratio to the results of unrestricted case of the same model.

Chapter Eight (Sensitivity study).

In this chapter, two main design parameters will be checked to ensure the overall results: the minimum number of modes taken into consideration due the analysis and the sensitivity of structure response analysis to the size of excitation period intervals.

Chapter Nine (Parametric study).

In this chapter, the effect of link-structure stiffness ratio will be checked and the results will be compared for different values to show how the response of each model varies with excitation frequency.

Chapter Ten (Conclusions, recommendations and future work).

This chapter provides conclusions drawn from the research with a focus on what has been observed from results presented in previous chapters. Recommendations and suggestions for future works are also presented.

Chapter Two
Literature Review

2.1 General

“In recent earthquakes, buildings have acted as weapons of mass destruction. It is time to formulate plans for a new United Nations mission - teams of inspectors to ensure that people do not construct buildings designed to kill their occupants” (Bilham, 2010).

This chapter gives a brief introduction to the basics of seismic analysis, vibration, types of excitations, and structural special behaviors.

Methods of analysis are illustrated and the performance levels are explained. After that, structural analysis procedures and their major limitations are outlined.

However, the choice of thesis topic is carefully selected and argued throughout this text, brief of the seismicity of the region, and description of diaphragm system and concrete slabs. In the meantime, scholarly materials are also analyzed comprehensively in order to derive a better feedback, and to obtain a real understanding into the sensitive issues.

2.2 Basics of Seismic Analysis

The major objective of seismic analysis is to develop a quantitative measure or a transfer function that can convert the strong ground motions at a structure’s foundation to loading and displacement demands of the structure, which provide essential input for a reliable assessment of structural capacity. (Jia, 2016).

As seen in figure (2.1), a building has the potential to wave back and forth (vibration) during an earthquake (Excitation). This is called the fundamental mode, and is the lowest frequency of building response. Most buildings have higher modes of response, which are uniquely activated during earthquakes. Nevertheless, the first and second modes tend to cause the most damage in most cases.

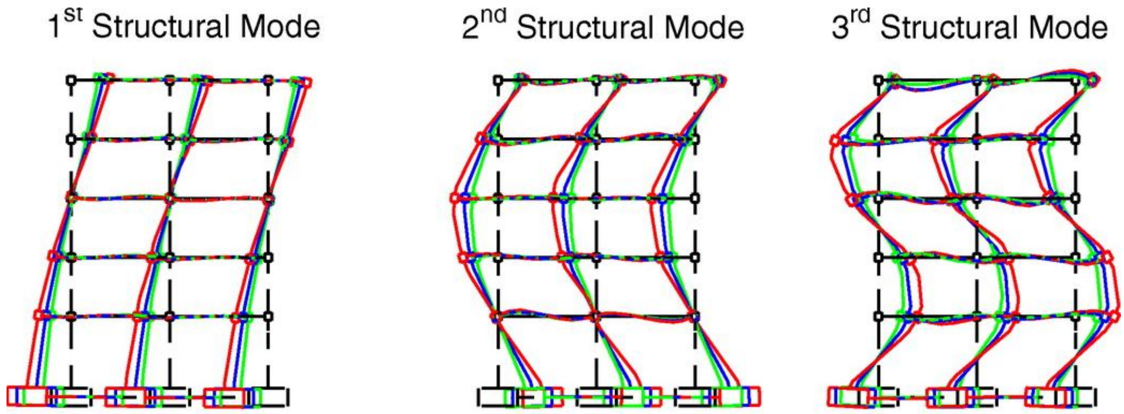


Figure (2. 1): Structural modes due earthquake (Ray W. Clough, Joseph Penzien, 2003)

Perhaps what distinguishes earthquakes from most other dynamic excitations, is that earthquakes apply in a form of support motions rather than by external forces applying on the above-ground portion of buildings (Ray W. Clough, Joseph Penzien, 2003).

If the ground and the base of the building shown in Figure (2.1) go a sudden incipient motion to the left, the ground floor and its contents will oppose to move with the base because of the inertia of their mass that resists the motion (Taranath, 2004).

As a result, the story with its contents will shift in an opposite direction just like if the structure is withdrawn to the right by a fictitious force, i.e. inertia force. Seismic loads are reversible in nature, and equal a portion of the weight of the building in their intensities (Amr S. Elnashai , Luigi Di Sarno , 2008).

2.2.1 Vibration:

is a structural phenomenon whereby oscillations occur about an equilibrium point.

Free vibration: occurs when a structural system is set in motion with an initial input and allowed to vibrate freely. The structural system vibrates at one or more of its natural frequencies and damps down to motionlessness.

Forced vibration: is when a time-varying disturbance (load, displacement or velocity) is applied to a structural system. The disturbance can be a periodic and steady-state input, a transient input, or a random input.

Steady state vibration: In systems theory, a system or a process is in a steady state if the variables (called state variables) which define the behavior of the system or the process are unchanging in time. In continuous time, this means that for those properties p of the system, the partial derivative with respect to time is zero and remains so:

$$\frac{\partial p}{\partial t} = 0, for\ all\ present\ and\ future\ t.$$

In discrete time, it means that the first difference of each property is zero and remains so:

$$p(t) - P(t-1) = 0, for\ all\ present\ and\ future\ t.$$

The steady state response is always harmonic, and has the same frequency as that of the forcing.

The amplitude of vibration is strongly dependent on the frequency of excitation, and on the properties of the spring-mass system.

The steady state response of a forced, damped, spring mass system is independent of the initial conditions.

Transient vibration: is the response of a system to a change from equilibrium or steady state.

Transient response: In electrical engineering and mechanical engineering, a transient response is the response of a system to a change from equilibrium or steady state. The transient response is not necessarily tied to abrupt events but to any event that affects the equilibrium of the system. The impulse response and step response are transient responses to a specific input (an impulse and a step, respectively).

Transient vibration is defined as a temporarily sustained vibration of a structural system. It may consist of forced or free vibrations, or both. Transient loading, also known as impact, or a non-periodic excitation.

In our case, Transient vibration could be as a result of two main issues:

1. Changes in structures natural period due to change in mass or stiffness.

2. Change in excitation's period due to non-periodic load.

In analysis of systems involving transient load, most of times is necessary to idealize the forcing function (displacement, velocity, acceleration or force) of such system, as a step, pulse or non-periodic function.

Unrestricted vibration: Joint range of vibration refers to both the distance a joint can move and the direction in which it can move, the system is unrestricted if it oscillates in a normal range of vibration without exterior limits.

Restricted vibration: A reduction in a normal range of motion in any of the joints is known as restricted range of vibration.

Butterfly effect: is the sensitive dependence on initial conditions in which a small change in one state of a deterministic nonlinear system can result in large differences in a later state.

2.2.2 Excitation:

the application of energy to something.

Periodic excitation: A periodic function is any function that repeats itself in time, called period T.

$$f(t) = f(t + T)$$

The periodic input can be a harmonic or a non-harmonic disturbance.

For linear systems, the frequency of the steady-state vibration response resulting from the application of a periodic, harmonic input is equal to the frequency of the applied force or motion, with the response magnitude being dependent on the actual structural system.

Non periodic excitation: Harmonic and steady-state excitation and response are conveniently described in the frequency domain. For deterministic non-periodic excitation and response, time domain technique is more suitable. We cannot find the repeated pattern that lasts forever (both in the past & future) for the non-periodic excitation.

Arbitrary Excitation: Ideally, arbitrary excitation can be expressed as linear combinations of simpler excitations. The simpler excitations are simple enough that the response is readily available. This concept is exactly used by Fourier. Now, the idea is to regard the arbitrary excitation as a superposition of impulses of varying magnitude and applied at different times. It is used when the excitation can be easily described in time domain.

2.3 Methods of analysis:

Displacement-based analysis: It refers to analysis procedures, such as the nonlinear static analysis procedures, whose basis lies in estimating the realistic, and generally inelastic, lateral displacements or deformations expected due to actual earthquake ground motion. Component forces are then determined based on the deformations.

Force-based design (FBD) methods: Traditional seismic design codes in the world are generally based on elastic analysis methods, where earthquake is presented as static forces. This comes in contrast to reality, where the structures can be exposed to large inelastic deformations in strong earthquake events, and this is not accurately accounted for in current force-based design methods.

Current building codes use static (ELF) procedures for seismic design of regular structures. This procedure is used for buildings with relatively short periods, but for buildings with relatively long periods, (ELF) procedure could be inaccurate, and the structure must be designed using other procedures (Chopra, 2012).

The design lateral forces acting on any structure depend on vibration properties of the structure and the site classification. Based on the estimated fundamental modal behavior of the structure, formulas are specified for calculating base shear, and then lateral forces are distributed over the height of the building accordingly. Static analysis of the building for these forces provides the design forces, including shears and overturning moments for the different stories and structural elements. (Chopra, 2012).

In these methods, the inelastic behavior of the building is incorporated as a reduction factor "R" of the base shear force.

Performance based seismic Design (PBD) methods: The goal of PBD is to develop design methodologies that produce structures of predictable and intended seismic performance under stated levels of seismic hazards. Then the international codes developed guidelines based on PBD to assess and rehabilitate existing buildings. (SEAOC, 1995).

Performance level: A limiting damage state or condition described by the physical damage within the building, the threat to life safety of the building’s occupants due to the damage, and the post-earthquake serviceability of the building. A building performance level is that combination of a structural performance level and a nonstructural performance level.

2.4 Structural analysis procedures

(FEMA356, 2000) divided structural analysis procedures into four procedures: linear static procedure, linear dynamic procedure, nonlinear static procedure (pushover analysis), and nonlinear dynamic procedure (time history). These types will be explained briefly below:

Linear procedures: The linear procedures imply the use of elastic analysis to evaluate the members capacities, then the elastic results are converted to inelastic by multiplying them with empirical inelastic factors. Linear procedures used by FEMA 356 are linear static procedure (LSP) and linear dynamic procedure (LDP). When the linear static procedure is used, the seismic design forces are distributed over the floors, corresponding internal forces and displacement will be determined by linear elastic analysis, and the model will be built using linear elastic stiffness materials, and equivalent viscous damping according to FEMA 356.

Nonlinear procedures: The nonlinear procedures used by FEMA 356 are nonlinear static procedure and nonlinear dynamic procedure. The nonlinear static procedure is done using nonlinear material behavior of members. The lateral load pattern is distributed on each floor of the building in accordance with the dominant mode shapes and floor weights. Then, the load is either statically or dynamically increased until certain deformation target is reached or numerical instability occurs.

Nonlinear procedure is better than linear procedures because it covers inelastic response. On the other hand, nonlinear dynamic procedure NDP simulates reality better than NSP. However, NSP is faster, less data needed, and less calculation intensive than NDP. Because of these advantages of NSP, engineers commonly use NSP in cases of assessment that can be seen in the intensive researches in the subject of performance-based design. (Augusto, 2011).

The nonlinear dynamic procedure is done by building a model that considers the local nonlinear behavior for individual elements in the model and components, then expose the model to realistic earthquake ground motion records (time history) and transient analysis is conducted in order to find the deflection of the building and internal forces. (Augusto, 2011)

2.5 Diaphragm

In structural engineering, a diaphragm is a structural element that transmits lateral loads to the vertical resisting elements of a structure (such as shear walls or frames). The diaphragm forces tend to be transferred to the vertical resisting elements primarily through in-plane shear stress. The most common lateral loads to be resisted are those resulting from wind and earthquake actions, but other lateral loads such as lateral earth pressure or hydrostatic pressure can also be resisted by diaphragm action.

Types of Reinforced Concrete Slabs: Civil engineers and contractors have practiced different traditional types of concrete slabs. Slabs could be classified with reference to different criteria such as the shape of plan, and the method of construction, slabs may be assorted to one-way slabs and two-way slabs.

However, the selection of slab type depends on economy, aesthetic features, loading, and lengths of the spans (Hassoun, M. Nadim, Al-Manaseer, Akthem, 2015).

At present, hollow slab systems have been developed by means of modern technologies. The created slab saves up to 35% of the dead weight of solid slab. Despite the almost equalized bending capacity of the two systems, there still a main difference in shear resistance which is highly dropped in the voided slab systems. Shear in modern slab systems needs check in some configurations especially when it resists pure axial load like our case. (Seyyed Ali Mousavi Gavgani, Babak Alinejad, 2015)

2.6 Seismic vulnerability

Seismic vulnerability means that inability of historical and monumental buildings to withstand the effects of seismic forces. The concept of vulnerability pertains to a system of basic concepts involved in risk analysis.

The closed tested vulnerable configuration that describes our case is the pounding effect of structures.

Pounding effect: Building pounding describes the collision of adjacent buildings as a result of some form of excitation.

typically, seismic excitation. This phenomenon has been the subject of much research over the last 30 years.

Unfortunately, almost all of these works have been contradicted by other researchers at some point.

This is mainly due to the high level of complexity inherent in the problem. Characterizing pounding requires a detailed knowledge of the dynamic performance of multiple buildings, as well as knowledge of how the buildings will react to very high magnitude but very small duration impulsive forces.

Pounding is thus very expensive to model physically and very complicated to represent analytically.

This paper presents the current state of the art of building pounding, with particular emphasis on the fundamental concepts of pounding. Pounding building scenarios can be generally categorized as either floor-to-floor, or floor-to-column pounding, or floor to ground pounding.

The buildings will react to very high magnitude but very small duration impulsive forces.

The main reason of the seismic pounding is the provision of insufficient gap or no gap in the building.

The response of adjacent buildings towards external force is mainly due to following conditions:

- When the separation gap between adjacent buildings is inadequate.

- When building have sufficient gap but they are connected by one or more members.

- When adjacent buildings have different dynamic properties like mass, height, orientation, geometry. It is almost impossible to construct two buildings with same dynamic properties. If the dynamic properties of two buildings are same, then there will be no pounding even if the gap is zero.

- When the center of mass of adjacent buildings is not axial.

Two types of pounding damage can occur:

1. Local damage at the point of impact

2. Global damage resulting from the energy and momentum transfer caused by collision.

Pounding is a very complex phenomenon, which makes the analysis of the corresponding problem complicated.

Earthquake lateral loads, the design lateral loads at different floor levels have been calculated corresponding to fundamental time period and are applied to the model.

For conventional method the differential equations governing the response of an MDOF system to earthquake induced ground motion:

$$\mathbf{m}\ddot{\mathbf{u}} + \mathbf{c}\dot{\mathbf{u}} + \mathbf{k}\mathbf{u} = \mathbf{p}_{\text{eff}}(t)$$

$$\mathbf{p}_{\text{eff}}(t) = -\mathbf{m}\iota\ddot{u}_g(t)$$

Modal analysis, or the mode-superposition method, is a linear dynamic-response procedure which evaluates and superimposes free-vibration mode shapes to characterize displacement patterns. Mode shapes describe the configurations into which a structure will naturally displace. Typically, lateral displacement patterns are of primary concern.

A structure with N degrees of freedom will have N corresponding mode shapes. Each mode shape is an independent and normalized displacement pattern which may be amplified and superimposed to create a resultant displacement pattern, as shown in Figure (1.7):

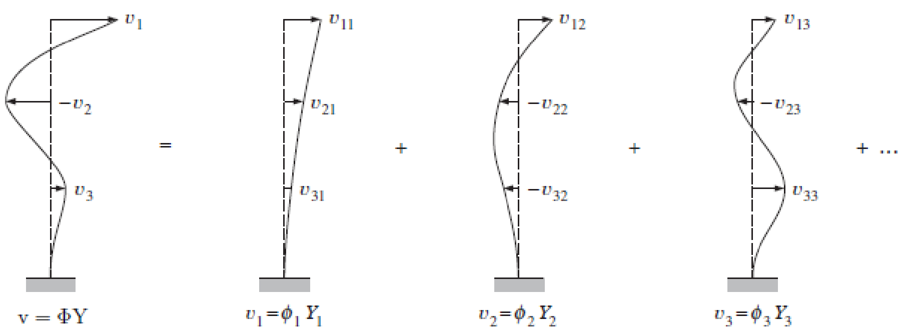


Figure (2. 2): Resultant displacement and modal components.

Numerical evaluation proceeds by reducing the equations of motion (N simultaneous differential equations coupled by full mass and stiffness matrices) to a much smaller set of uncoupled second order differential equations (N independent normal-coordinate equations). The orthogonality of mode-shape relations enables this reduction.

The main issue here is to consider the drift restriction at certain levels in the shape functions of the building, so the equation of motion of these structures can be expressed as (Anagnostopoulos, 1988)

$$M\ddot{u}(t) + C\dot{u}(t) + K^T u(t) + F + R = -M\ddot{u}_g(t)$$

Where upper dots represent derivatives of time, C is the viscous damping matrix, M the matrix of mass, K^T the tangent stiffness matrix and $\ddot{u}_g(t)$ the ground acceleration. Furthermore, F and R are the vectors of impact forces and of restoring forces due to impact, respectively.

For our case, there is no other masses rather those in the original system in motion, the nonlinearity comes from changing in stiffness due coupling with link stiffness.

Rewriting previous equation for the system, we obtain two of differential equations describing the response of the configuration to the ground acceleration $\ddot{u}_g$, this system is uncoupled if the system drifts out of link side, and it respond to the lateral forces with its original stiffness matrix.

Coupling introduced whenever a compression force starts in the link, then the axial stiffness of link added to the diagonal of structure stiffness matrix at the row corresponding to link level.

2.7 Seismicity of Palestine

The State of Palestine is historically proven to be prone to earthquakes. These earthquakes were gloom events to Palestinians due to their horrible damage and the large number of deaths, estimated in hundreds and probably in thousands (United Nations, 2014). The geographical location of Palestine puts the country along the Aqaba-Dead Sea Transform Fault (DSTF) which is the most seismically active plate boundary in the Middle East (BEN AVRAHAM Z., LAZAR M., SCHATTNER U., & MARCO S, 2005). Figure (2.2) demonstrates a lot of earthquakes that hit Palestine during the past centuries. Rightly, they struck along the DSTF (AL-DABBEEK, J. & EL-KELANI, R., 2004)

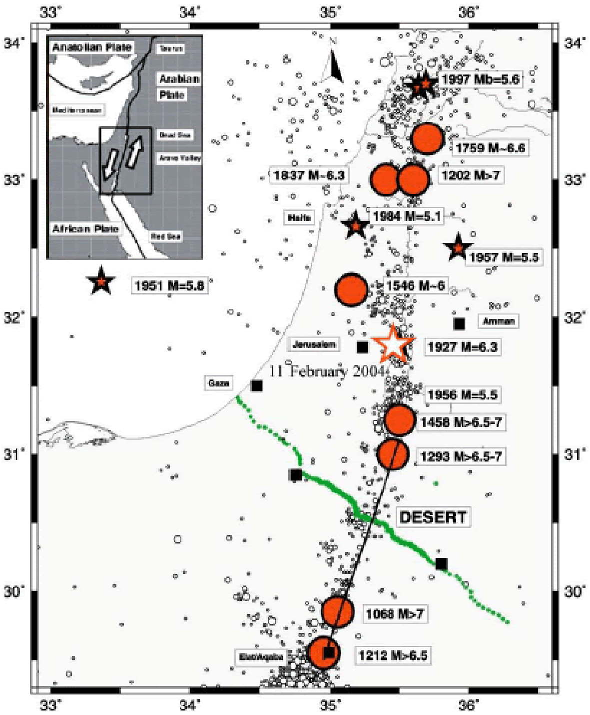


Figure (2. 3): Seismicity map of the Dead Sea Transform region (AL-DABBEEK, J. & EL-KELANI, R., 2004).

2.8 Modeling of uncertain seismic performance buildings

A challenge to characterizing the uncertain future seismic performance of a class of buildings is how to present its variability with rigor and a small sample of individual buildings. Second – generation performance – based earthquake engineering (PBEE-2) , provides insight into seismic performance buildings with rigorous propagation of uncertainty, nonlinear time history structural analysis, performance measured in terms of dollars, deaths, and downtime, and reasonable independence from expert opinion. But its asset definition is deterministic: it works on one building at a time. If one could make the asset definition probabilistic and have the distribution of its attributes represent that of a specified class of buildings, then this enhanced version of PBEE-2 would allow one to treat classes of buildings and model the behavior of buildings at the social level, such as for catastrophic risk modeling. (K.Porter & I.Cho, 2013)

In this work we only try to make a simple model of frames have the same configuration of structures in concern, and know the sample to be representative of the class. We select a sample that spans the readily observable features that matter most to the class.

The procedure allows one to create a class level vulnerability function and reflect all of the most important variability within the class

2.9 Theoretical and experimental studies

Research in pounding has predominantly focused on the analytical modelling of buildings. The general floor-to-floor modelling method consists of either a single node, or multiple nodes slaved together, to create a rigid diaphragm at each floor of each building (Mouzakis and Papadrakakis 2004; Muthukumar and Desroches 2006; ULIEGE 2007).

The most significant effect of the presence of a link between structure and surrounding nonstructural elements is the introduction of certain modes of response that are not present in the free standing case, and some of the degrees of freedom will be coupled. The extent of this interaction is dependent mainly on the characteristics of the connected object: mass ratio of the adjacent object, their natural frequencies, stiffness, and the predominant frequency of the excitation.

Pounding between building and natural ground is a local issue that related to the natural terrain of some countries like some cities of Palestine (Ramallah, Nablus and Hebron) which isn't covered in any researches.

But pounding between neighboring buildings during earthquakes considering soil-structure interaction is an issue that has attracted considerable interest, see, for example:

(Stavros A. Anagnostopoulos , K. V. Spiliopoulos, 1992) studied the earthquake induced pounding between adjacent buildings. They idealized the building as lumped-mass, shear beam type, multi-degree-of-freedom

(MDOF) systems with bilinear force deformation characteristics and with bases supported on translational and rocking spring dashpots. Collisions between adjacent masses can occur at any level and are simulated by means of viscoelastic impact elements. They used five real earthquake motions to study the effects of the following factors: building configuration and relative size, seismic separation distance and impact element properties. It was found that pounding can cause high overstresses, mainly when the colliding buildings have significantly different heights, periods or masses. They suggest a possibility for introducing a set of conditions into the codes, combined with some special measures, as an alternative to the seismic separation requirement.

(Rahman AM, Carr AJ, Moss PJ, 2001) studied the effects of foundation compliance of the conventional structures and the importance of soil flexibility has been highlighted. These authors concluded that the seismic response of the structure increased with consideration of soil flexibility due to the increases in the natural periods of the adjacent buildings, compliance effects must also be taken into account when determining the location of sensitive equipment and appurtenances due to the localized effects of the large amplitude impacts.

(Rabiul Hasan Rabi Hasan, Lei Xu and D.E Grierson, 2002) presented a simple computer based pushover analysis technique for performance based design of building frameworks subject to earthquake loading. The concept is based on conventional displacement method of elastic analysis. To measure the degree of plastification the term plasticity factor was used. The standard elastic and geometric stiffness matrices for frame elements are progressively modified to account for non-linear elastic-plastic behavior under constant gravity loads and incrementally increasing lateral loads.

(Jankowski, 2004) addressed the fundamental questions concerning the application of the nonlinear analysis and its feasibility and limitations in predicting seismic pounding gap between buildings. In his analysis, elastoplastic multi-degree-of freedom lumped mass models are used to simulate the structural behavior and non-linear viscoelastic impact elements are applied to model collisions. The results of the study prove that pounding may have considerable influence on behavior of the structures.

(L. Gong, 2005) investigated the seismic responses of the adjacent buildings subjected to pounding due to spatially varying earthquakes. The attenuation of waves propagating through the soil and the associated time lag caused the buildings to experience different seismic responses. However, the influence of the spatial variation of earthquake ground motions is of secondary importance compared to the SSI, because the adjacent buildings are close to each other.

(Viviane, 2007) summarized basic concepts on which the seismic pounding effect occurs between adjacent buildings. She identified the conditions under which the seismic pounding will occur between buildings and adequate information and, perhaps more importantly, pounding situation analyzed. From her research it was found that an elastic model cannot predict correctly the behaviors of the structure due to seismic pounding. Therefore, non-elastic analysis is to be done to predict the required seismic gap between buildings.

(Shakya K, Wijeyewickrema AC, 2009) analyzed unequal story height buildings considering the underlying soil effects to study the mid-column pounding of the adjacent buildings. They used the SAP2000 software to model the adjacent buildings and the underlying soil. The buildings were connected by a combination of the gap element and the Kelvin–Voigt model. These authors asserted that pounding forces, inter-story displacements and normalized story shears were generally decreased when the underlying soil was considered.

(AbdelRaheem, 2011) developed and implemented a tool for the inelastic analysis of seismic pounding effect between buildings. They carried out a parametric study on buildings pounding response as well as proper seismic hazard mitigation practice for adjacent buildings. Three categories of recorded earthquake excitation were used for input. He studied the effect of impact using linear and nonlinear contact force model for different separation distances and compared with nominal model without pounding consideration.

(Kasim Korkmaz, Ali Sari and Asuman I. Carhoglu, 2011) studied the performance of structures for various load patterns and variety of natural periods by performing pushover and nonlinear dynamic time history analysis and concluded that for taller structures pushover analysis is underestimating seismic demands.

(Mahmoud S, Abd-Elhamed A, Jankowski R, 2013) investigated the coupled effect of the supporting soil flexibility and pounding between neighboring, insufficiently separated equal height buildings under earthquake excitation

(Qin X, Chouw N, 2013) presented a numerical investigation of seismic gap between adjacent structures with structure–foundation–soil interaction (SFSI).

(Alam MI, Kim D, 2014) studied the spatially varying ground motion effects on seismic response of adjacent structures considering soil–structure interaction (SSI) and found that the responses of adjacent structures have changed remarkably due to spatial variation of ground motions.

(Madani B, Behnamfar F, Tajmir Riahi H, 2015) studied the effects of pounding and structure–soil–structure interaction on the nonlinear dynamic behavior of selected adjacent structures.

(Ghandil M, Behnamfar F, Vafaeian M, 2016) studied the dynamic responses of structure–soil–structure systems with an extension of the equivalent linear soil modeling and investigated the problem of cross-interaction of two adjacent buildings through the underlying soil

2.10 Outcomes of Literature Review

From the available literature it was observed that most of the studies are confined on study of 2D frames and simple 3D structures with one story and one bay, which is concentrated in the probability of impact.

Limited number of published works on comparison of use of dynamic and pushover analysis to find out the seismic gap between buildings.

Number of published works trying to find out the local point force and effect on structure.

Thus, after reviewing the existing literature, a comparative study on seismic pounding effect on buildings by dynamic nonlinear analysis is required to find the overall effect of bounding on structure.

Covering materials adopted the gap link in structural modeling for similar cases.

Covering materials do not take part in more than structural stiffness. Thereby, adding another variables relating to link will complicate the equation of motion.

Chapter Three
Physical Insight

3.1 General

Most structures vibrate. In operation, all machines, vehicles and buildings are subjected to dynamic forces which cause vibrations. Very often the vibrations have to be investigated, either because they cause an immediate problem, or because the structure has to be cleared to a standard or test specification. Whatever the reason, we need to quantify the structural response in some way, so that its implication on factors such as performance and fatigue can be evaluated. (Genta, 1999)

By using signal-analysis techniques, we can measure vibration on the operating structure and make a frequency analysis.

The frequency spectrum: description of how the vibration level varies with frequency can then be checked against a specification. This type of testing will give results which are only relevant to the measured conditions.

The result will be a product of the structural response and the spectrum of an unknown excitation force, it will give little or no information about the characteristics of the structure itself.

An alternative approach is the system-analysis technique in which a dual-channel analyzer can be used to measure the ratio of the response to a measured input force.

The frequency response function: (FRF) measurement removes the force spectrum from the data and describes the inherent structural response between the measurement points. From a set of FRF measurements made at defined points on a structure, we can begin to build up a picture of its response. The technique used to do this is modal analysis.

3.2 Modal Behavior

FRF measurement made on any structure will show its response to be a series of peaks. The individual peaks are often sharp, with identifiable center frequencies, indicating that they are resonances, each typical of the response of a single-degree-of-freedom (SDOF) structure. If the broader peaks in the FRF are analyzed with increased frequency resolution, two or more resonances are usually found close together. The implication is that a structure behaves as if it is a set of SDOF substructures. This is the basis of modal analysis, through which the behavior of a structure can be analyzed by identifying and evaluating all the resonances, or modes, in its response.

Let us begin with a review of how structural response can be represented in different domains. Through this we will be able to see how the modal description relates to descriptions in the spatial, time and frequency domains. As our example, we will take the response of a frame, which is a lightly damped structure. When the frame is struck, it produces a visual response containing a limited number of shapes. The associated vibration response has exactly the same pattern, and the frame seems to store the energy from the impact and dissipate it by vibrating at particular discrete frequencies.

The response of the frame represented in different domains:

In the physical domain: the complex geometrical deflection pattern of the frame, can be represented by a set of simpler, independent deflection patterns, or mode shapes.

In the time domain: the vibration response of the frame is shown as a time history, which can be represented by a set of decaying sinusoids.

In the frequency domain: analysis of the time signal gives us a spectrum containing a series of peaks, shown as a set of SDOF response spectra.

In the modal domain: we see the response of the frame as a modal model constructed from a set of SDOF models. Since a mode shape is the pattern of movement for all the points on the structure at a modal frequency, a single modal coordinate q can be used to represent the entire movement contribution of each mode.

Looking back from the modal domain, we see that each SDOF model is associated with a frequency, a clamping and a mode shape. These are the Modal parameters:

• Modal frequency

• Modal damping

• Mode shape

Which together form a complete description of the inherent dynamic characteristics of the frame, and are constant whether the frame is vibrating or not. (Bruel & Kjaer, 1988)

3.3 Modal Analysis

Is the process of determining the modal parameters of a structure for all modes in the frequency range of interest. The ultimate goal is to use these parameters to construct a modal model of the response.

Two observations worth noting here are that:

• Any forced dynamic deflection of a structure can be represented as a weighted sum of its mode shapes.

• Each mode can be represented by SDOF model.

3.4 Single-degree-of-freedom (SDOF) Models

As each peak - or mode - in a structural response can be represented by an SDOF model, we will look at some aspects of SDOF dynamics. In particular, we will examine the way in which SDOF structure can be modeled in the physical, time and frequency domains. These models are not intended to represent physical structures, but will serve as instruments for interpreting dynamic behavior (constrained by a set of assumptions and boundary conditions). They will help us to:

• understand and interpret the behavior of structures.

• describe the dynamic properties of structures, using a small set of parameters.

• extract the parameters from measured data (curve-fitting).

•**An analytical model:** can be constructed in the physical domain. It is an abstract system consisting of a *point mass (m)* supported by a *mass-less linear spring (k)* and connected to a linear *viscous damper (c).* The mass is constrained so that it can move in only one direction (x) - a *Single degree-of-freedom.*

• **A mathematical model**: in the time domain can be derived by applying Newton's Second Law to the analytical model. By equating the internal forces (inertia, damping and elasticity) with the external (excitation) force, we obtain the model

$$m.\ddot{u} + c.\dot{u} + k.u = f(t)$$

which is a second-order differential equation. A model which is more mathematically manageable can be obtained in the frequency domain.

3.5 Multiple-degree-of-freedom (MDOF) Models

Real structures have many points which can move independently - many degrees-of-freedom. To make an FRF measurement on a real structure we have to measure the excitation and response between two points. But any point may have up to six possible ways of moving so we must also specify the measurement direction.

A degree-of-freedom (DOF): is a measurement-point-and direction defined on a structure. An index “i” is used to indicate a response DOF, and “j” an excitation DOF. Additional indices x,y and z may be used to indicate the direction.

Thus $H_{ij}(\omega) = \frac{X_i(\omega)}{F_j(\omega)}$

By writing $H_{ij}(\omega)$ in two different ways, we obtain the two “ij” MDOF models shown as equations in the illustration

• **The MDOF FRF-model:** represents Hij (ω) as the sum of SDOF FRFs, one for each mode within the frequency range of the measurement, where r is the mode number and m is the number of modes in the model.

• **The MDOF modal-parameter model**: defines Hij (ω) in terms of the pole locations and residues of the individual modes. This model indicates two significant properties of the modal parameters:_Modal frequency and damping are global properties. The pole location has only a mode number (r) and is independent of the DOFs used for the measurement.

• **The residue is a local property:** The index “ijr” relates it to a particular combination of DOFs and a particular mode. (Bruel & Kjaer, 1988).

3.6 Force of impact – Classical Physics View

Force of impact is the total force exerted on an object during a collision. To derive the impact force equation, you can consider the law of conservation of energy. At the beginning, a moving object possesses kinetic energy that reduces to zero after the collision (object stops). To fulfill the conservation law, the change of kinetic energy must be compensated by the work done by the impact force. We express it with the below impact force equation.

$$F = \frac{m * v^2}{2 * d}$$

Where

- F is the average impact force,
- m is the mass of an object,
- v is the initial speed of an object,
- d is the distance traveled during collision.

$F = \frac{m * v^2}{2 * d} + \frac{K * d}{2}$, considering axial stiffness of link

Where k is the axial stiffness of the link

It's clear that extending the distance moved during the collision reduces the average impact force. It should be easier to understand if we rewrite the above impact force formula in the alternative version using the time of collision t instead of the distance d:

$$F = \frac{v * m}{2 * t}$$

$F = \frac{v*m}{2*t} + \frac{K*d}{2}$, considering axial stiffness of link

It's considerable to use "t" because of the direct relation to the structure's mode period, where "t" = 0.5 * T_n

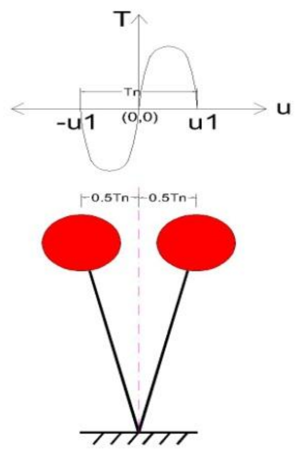


Figure (3. 1): 1mpact time =0.5Tn.

This is a special case of impulse and momentum formula. Now, we can see that extending the time of the collision will decrease the average impact force. (Banas, 2018)

This formula could be applied for free moving objects, or free vibrating structures. Forced vibrating structures have different approach depending on the excitation function as illustrated in the following sections.

3.7 Harmonic periodic load

A sample of harmonic force is p(t) = P0 sin ωt, where P0 is the amplitude or maximum value of the force and its frequency ω is called the exciting frequency or forcing frequency; (Chopra, 2012)

T = 2π/ω is the exciting period or forcing period (Fig. 3.1).

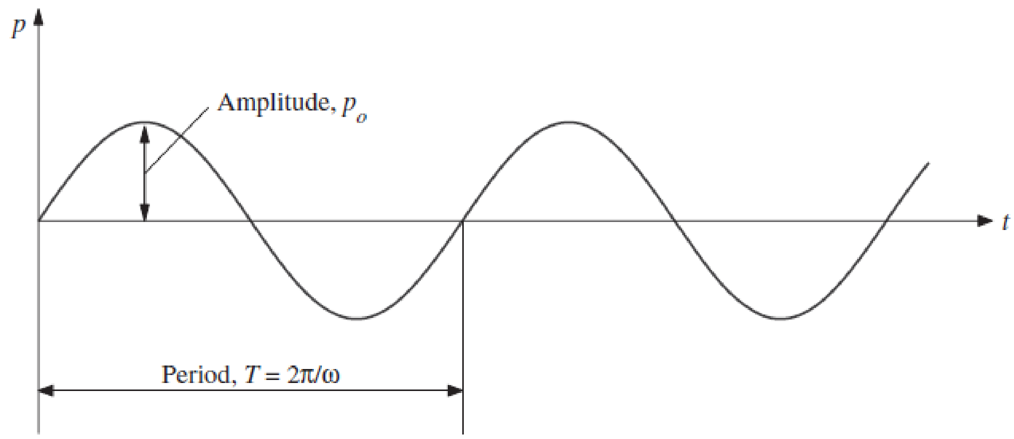


Figure (3. 2): p(t) = P0 sin ωt

The differential equation governing the response of SDF systems to harmonic force is

$$m.\ddot{u} + c.\dot{u} + k.u = P_0.\sin(\omega.t)$$

This equation will be solved for the displacement or deformation u(t) subject to the initial conditions

$$u = u(0), \quad \dot{u} = \dot{u}(0)$$

Where $u(0)$ and $\dot{u}(0)$ are the displacement and velocity at the time instant the force is applied.

The particular solution to this differential equation is:

$u_p(t)$= C.sin(ω.t) + D. cos(ω.t)

Where:

$$C= \frac{P0}{k}\cdot\frac{\left(1-\left(\frac{\omega}{\omega n}\right)^2\right)}{\left(1-\left(\frac{\omega}{\omega n}\right)^2\right)^2+\left(2\cdot\xi\cdot\left(\frac{\omega}{\omega n}\right)\right)^2}$$

$$D= \frac{P0}{k}\cdot\frac{\left(-2\cdot\xi\cdot\left(\frac{\omega}{\omega n}\right)\right)}{\left(1-\left(\frac{\omega}{\omega n}\right)^2\right)^2+\left(2\cdot\xi\cdot\left(\frac{\omega}{\omega n}\right)\right)^2}$$

Then

$$u_p(t)=\frac{P0\left(1-\frac{\omega^2}{\omega n^2}\right)\sin(\omega\, t)}{k\left(\left(1-\frac{\omega^2}{\omega n^2}\right)^2+\frac{4\,\xi^2\,\omega^2}{\omega n^2}\right)}-\frac{2\,P0\,\xi\,\omega\cos(\omega\, t)}{\omega n\left(\left(1-\frac{\omega^2}{\omega n^2}\right)^2+\frac{4\,\xi^2\,\omega^2}{\omega n^2}\right)k}$$

The complementary solution is the free vibration response given by:

$$u_c(t)=e^{-\xi\cdot\omega n\cdot t}\cdot\left(A\cdot\cos(\omega D\cdot t)+B\cdot\sin(\omega D\cdot t)\right)$$

Where

$$\omega D=\omega n\cdot\left(\sqrt{1-\xi^2}\right)$$

$$A = u0 \ , \ B = \frac{v0}{\omega n} - \frac{P0}{k} \cdot \left(\frac{\frac{\omega}{\omega n}}{1 - \left(\frac{\omega}{\omega n}\right)^2} \right)$$

Then:

$$u_c(t) =$$
$$e^{-\xi \omega n\, t} \left(u0 \cos\left(\omega n \sqrt{-\xi^2 + 1}\ t\right) + \left(\frac{v0}{\omega n} - \frac{P0\, \omega}{k \cdot \omega n \left(1 - \frac{\omega^2}{\omega n^2}\right)} \right) \sin\left(\omega n \sqrt{-\xi^2 + 1}\ t\right) \right)$$

3.8 Initial Understanding

To understand the effect of the connection to SDOF Frame, the performance of a frame which is free to vibrate under harmonic load to be compared with the same frame with a link restricts its movement in one direction:

For this purpose, starting with a verified model of a frame with computed mass and stiffness, then trace its displacement to determine the maximum drift in positive and negative X-Direction:

For unrestricted vibration: there is one natural period of the structure depends on its stiffness and mass, displacement contains two distinct vibration components:

The sin (ωt) term, giving an oscillation at the forcing or exciting frequency;

and the sin ($\omega_n t$) and cos ($\omega_n t$) terms, giving an oscillation at the natural frequency of the system.

The first of these is the forced vibration or steady-state vibration, for it is present because of the applied force no matter what the initial conditions. The latter is the free vibration or transient vibration, which depends on the initial displacement and velocity.

For restricted vibration: stiffness changes related to change in the contact condition of link:

First phase, the structure performs in its natural period out of the link and has total displacement equal to free vibration case, because of same parameters and zero initial conditions in the two cases

At the instant the structure returns to its origin and start to drift towards the connection, the structure will go to phase 2 and perform in new period because of the additional stiffness added, starting with initial velocity equals the velocity of the structure at the end of phase 1,

When the compression force in the link released and the structure tend to vibrate out of the link again, it enters phase 3 and vibrates in its original period with initial velocity gained from the previous phase, and it could achieve new higher record in that direction at "lower excitation periods", see fig (3.3).

The alternating change in stiffness causes a transient vibration which prevents natural decaying of vibration.

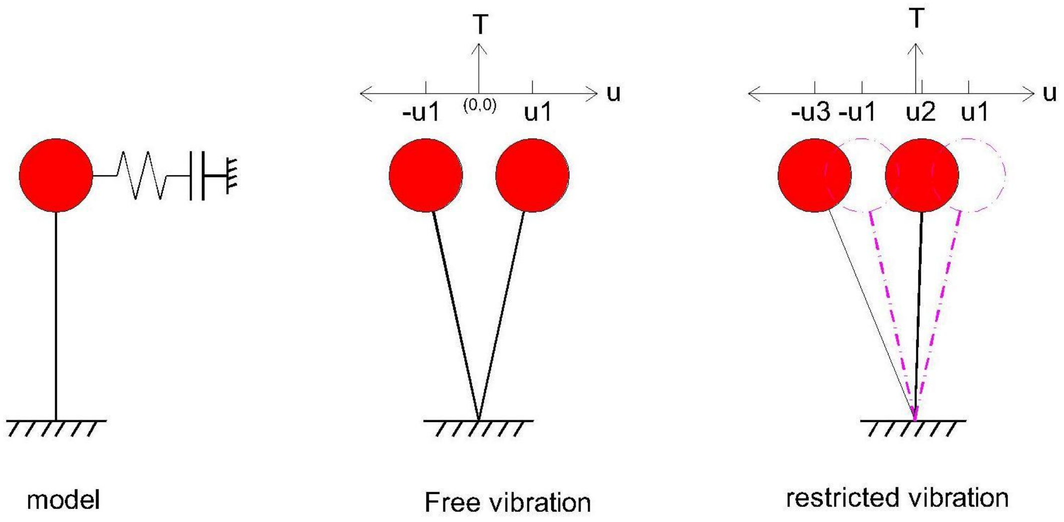


Figure (3. 3): SDOD model 1st mode vibration due to link at level 1.

It's clear that displacement is a sum of periodic functions with different periods, so the summation is not always periodic, it is well known that the sum of two continuous periodic functions on R is periodic if and only if their periods are commensurable, and this is the reason that we cannot use the direct laws of classical physics.

The main concern here is to find how the displacement function changes its direction rather than how it repeats the same vibration.

For unrestricted vibration the plot of the complete solution as the sum of the complementary and particular solutions, provides the performance of the structure for amplitudes and direction directly.

But for restricted vibration, the displacement of each beat should be computed in discrete manner then a long operation of superposition considering amplitudes and durations will be done to describe the whole performance of structure.

So, a nonlinear program will be used for this purpose and then it’s easily checked for any peak.

The nonlinear load-displacement relationship—the stress-strain relationship with a nonlinear function of stress, strain, and/or time; changes in geometry due to large displacements; irreversible structural behavior upon removal of the external loads; change in boundary conditions such as a change in the contact area and the influence of loading sequence on the behavior of the structure requires a nonlinear structural analysis.

The structural nonlinearities can be classified as geometric nonlinearity, material nonlinearity, and contact or boundary nonlinearity.

For effective nonlinear analysis, a good physical and theoretical understanding is most important.

PHYSICAL INSIGHT $\longrightarrow$ MATHEMATICAL FORMULATION

Best Approach:

• Use reliable and generally applicable finite elements.

• With such methods, we can establish models that we understand.

• Start with simple models (of nature) and refine these as need arises

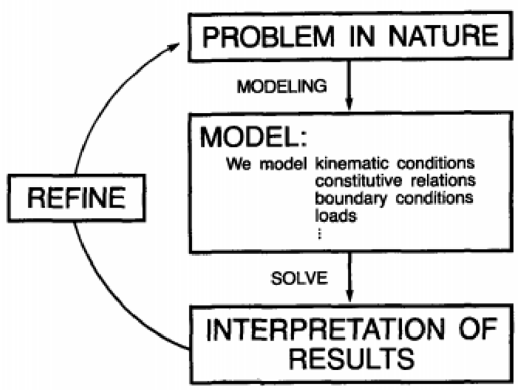
To perform a nonlinear analysis:

• Stay with relatively small and reliable models.

• Perform a linear analysis first.

• Refine the model by introducing nonlinearities as desired.

• Important: - Use reliable and well-understood models. - Obtain accurate solutions of the models. (Springer, 2006)



Start with Understanding the response of SDOF systems to harmonic excitation because understanding the response of structures to harmonic excitation provides insight into how the system will respond to other types of forces. (Springer, 2006)

By arbitrary trials, it shown that restricted structures go further more displacement amplitudes than free structure in free side fig (3.3), but this observation is not valid for all cases of excitations period, so a parametric graph of displacements for different links stiffness values and different excitation periods is needed.

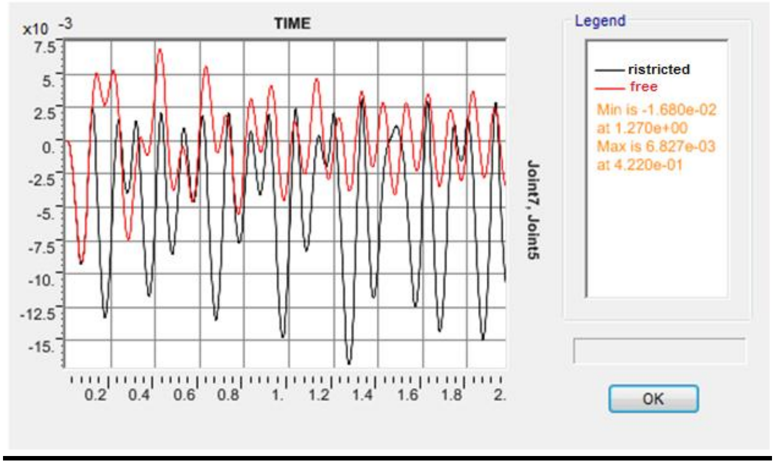


Figure (3. 4): response of free and restricted structures to arbitrary sine excitation period.

For a system has other masses than the linked one, it's considerable to make a physical sense of its behavior to can make judge of software results.

The basic concept which could determine the expected drift of the system is the movement of the free masses due to the direction of impact for linked mass, fig (3.5) illustrates this concept.

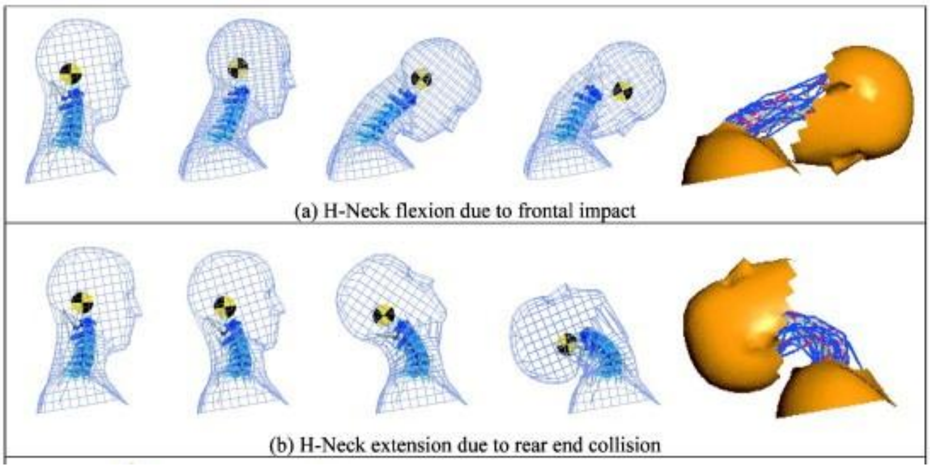


Figure (3. 5): Illustration of free mass movement due to impact force.

The free masses drift in the direction of impact so it's clear as shown later that the 1st mode where all masses drift in the same direction is the critical:

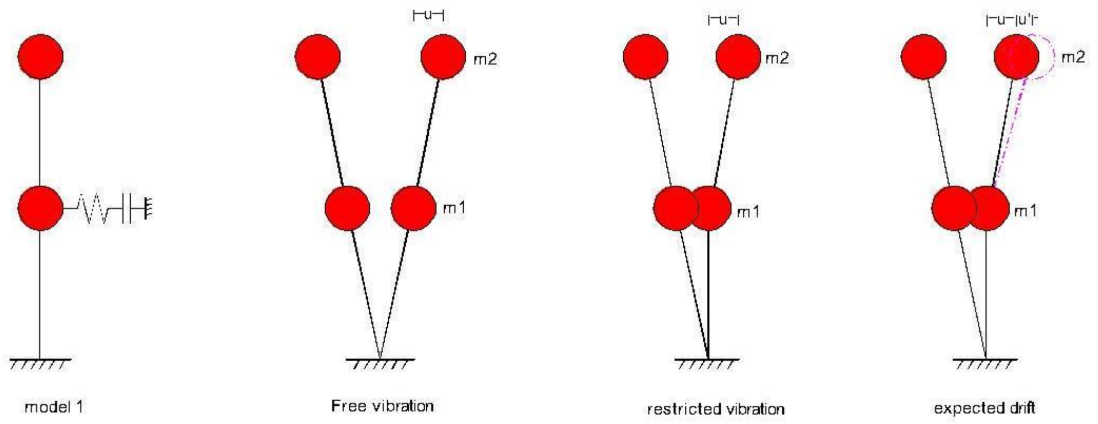


Figure (3. 6): 2dof 1st mode vibration due to link at level 1.

For the system of two masses with a link attached to the lower mass shown in fig (3.6), if "u" represents the relative displacement between the two masses in the unrestricted structure, then the upper mass of restricted structure expected to go further more distance in that direction equal " u+u' " due to impact.

For the same system but the link attached to the upper mass fig (3.7), whatever the lower mass in restricted model goes a relative displacement more or less than the unrestricted structure, it's clear that it drifts much more than the upper mass, which would create a negative shear in the upper storey, and the joint between two stories resists opposite shear forces.

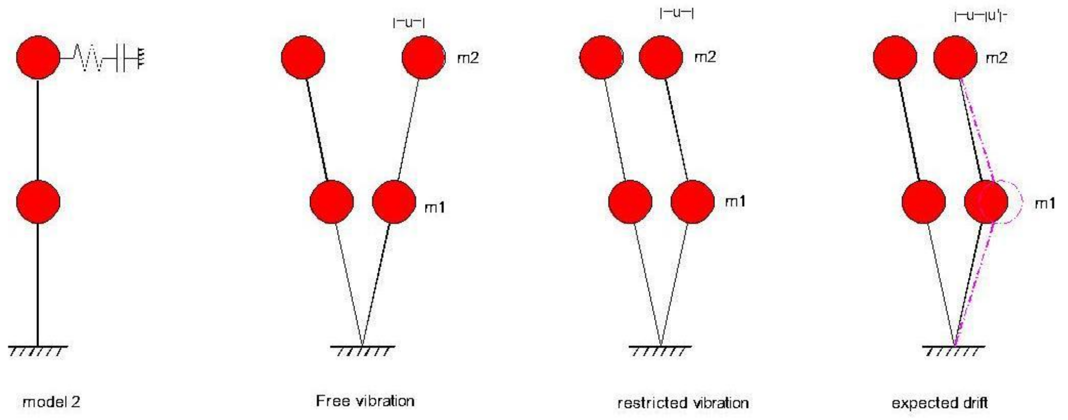


Figure (3. 7): 2DOF 1st mode vibration due to link at level 2.

Fig (3.8) shows a description of first mode drift for free and linked masses due to applying sine harmonic excitation to the system.

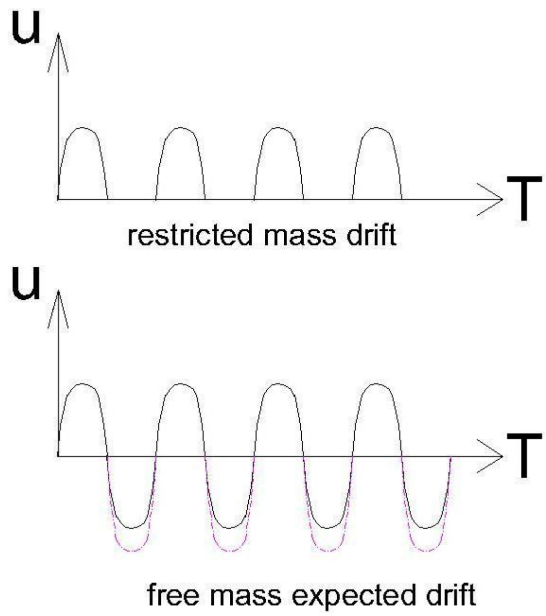


Figure (3. 8): 2DOF 1st mode restricted performance.

For the system of two masses with a link attached to the lower mass shown in fig(3.9), the second mode of vibration where the two masses drift in opposite directions, if "u" represents the relative displacement between the two masses in the restricted structure, then the upper mass of restricted structure expected to go less distance in that direction equal " u-u' " due to impact.

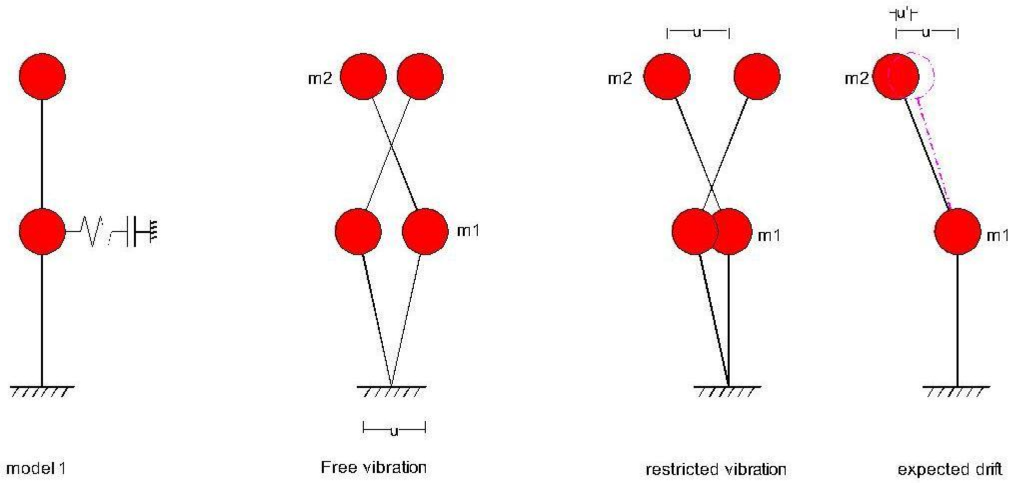


Figure (3. 9): 2DOF 2nd mode vibration due to link at level 1.

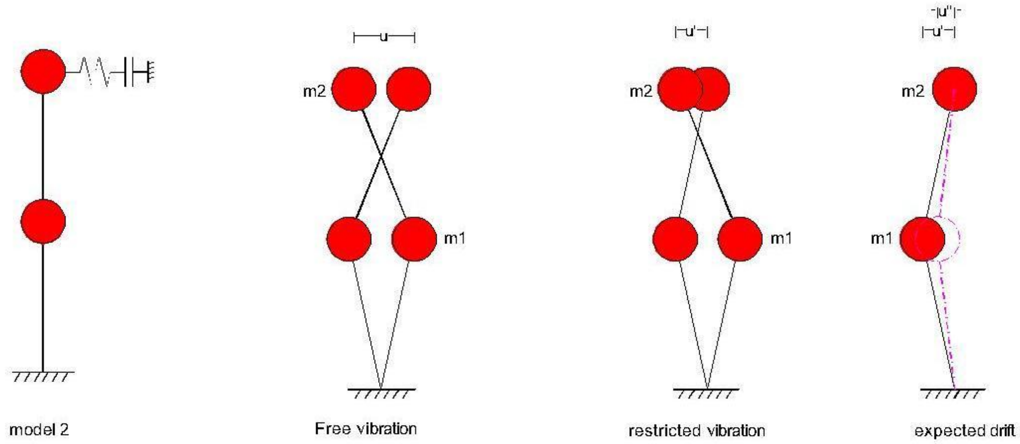


Figure (3. 10): 2DOF 2nd mode vibration due to link at level 2.

For the same system but the link attached to the upper mass fig (3.10), by applying the same concept, it’s clear that the relative displacement in the restricted structure is less than that of unrestricted model.

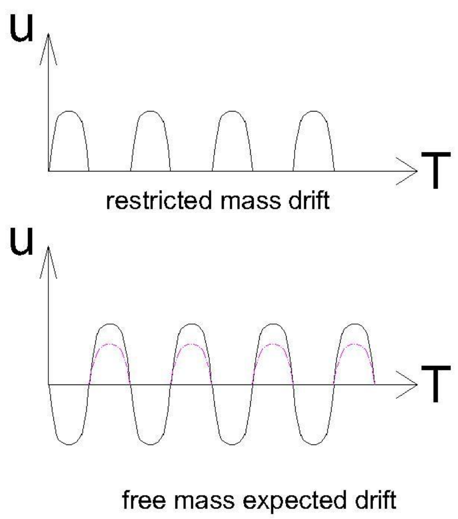


Figure (3. 11): 2DOF 2nd mode restricted performance.

Fig (3.11) shows a description of second mode drift for free and linked masses due to applying sine harmonic excitation to the system**.**

For three degrees of freedoms structures and more, a new case of free masses above and below the linked mass will be illustrated here:

Fig (3.12) below shows the possible three modes of vibration for 3DOF structures.

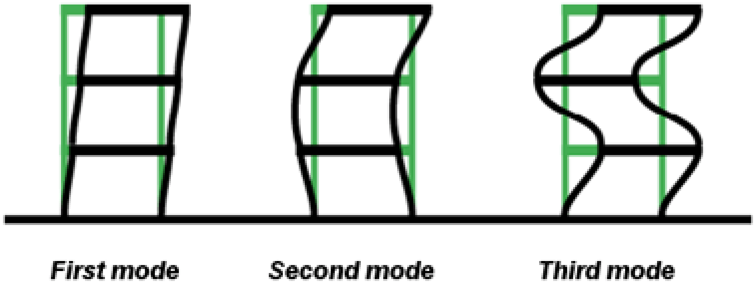


Figure (3. 12): The three modes of vibration for 3DOF model.

The first mode of vibration may suffer more relative displacement as shown in fig (3.13) below.

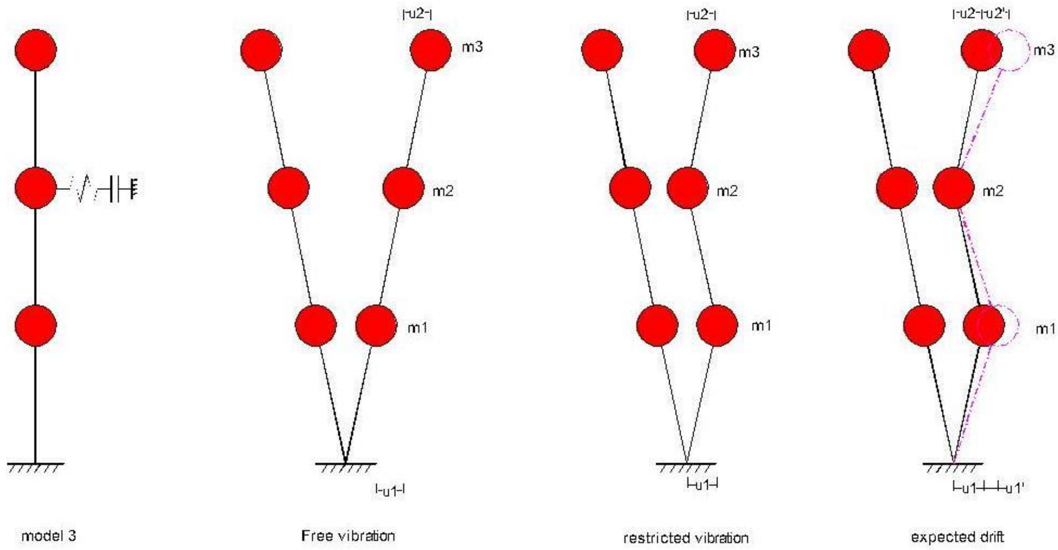


Figure (3. 13): 3DOF 1st mode vibration due to link at level 2.

For the second mode of vibration of this model, the lower mass expected to do more relative displacement while the upper mass doing less.

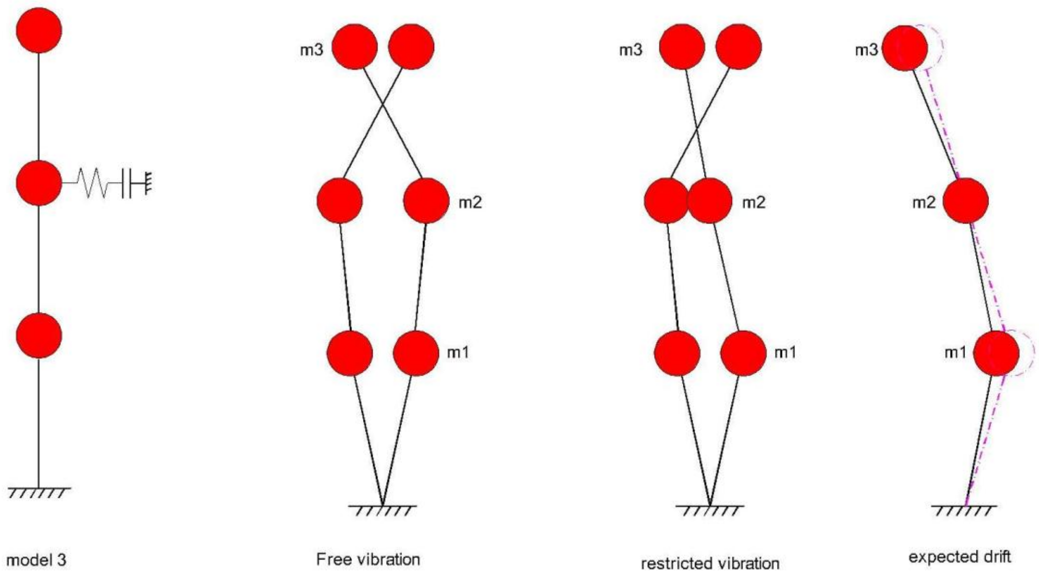


Figure (3. 14): 3DOF 2nd mode of vibration due to link at level 2.

The 3rd mode would be conservative as shown below.

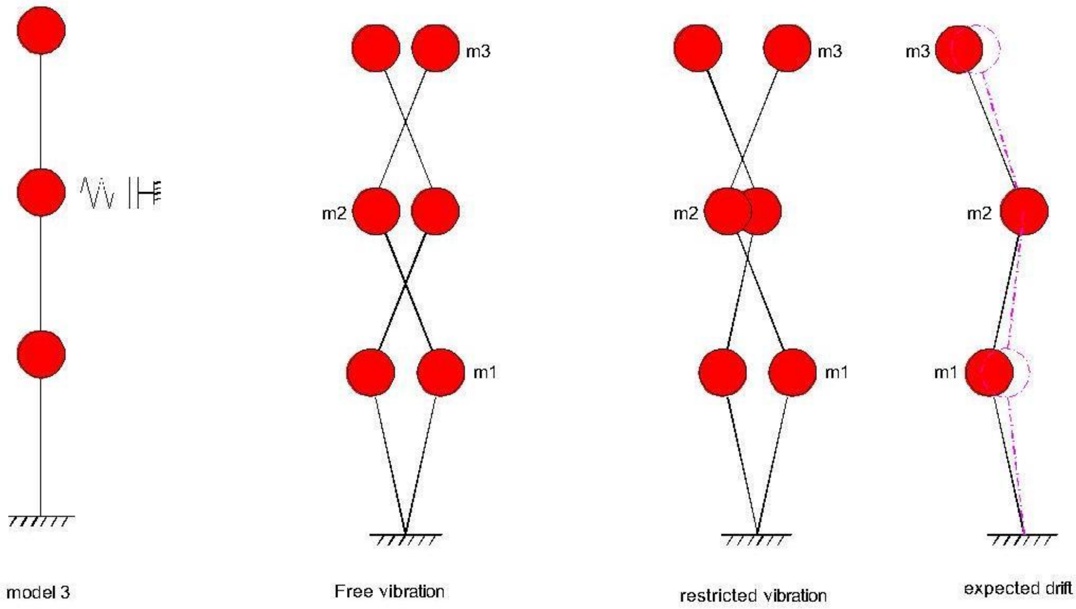


Figure (3. 15): 3DOF 3rd mode of vibration due to link at level 2.

Chapter Four

Structural Modeling

4.1 General

Structural analysis is a process to analyze a structural system to predict its responses and behaviors by using physical laws and mathematical equations. The main objective of structural analysis is to determine internal forces, stresses and deformations of structures under various load effects.

Structural modeling is a tool to establish three mathematical models, including (1) a structural model consisting of three basic components: structural members or components, joints (nodes, connecting edges or surfaces), and boundary conditions (supports and foundations); (2) a material model; and (3) a load model.

This chapter summarizes the guidelines and principles for structural analysis and modeling used for this study.

For the purpose of this study, the structural software (SAP2000, 2018) has been chosen, a linear and non-linear static and dynamic analysis and design program for three dimensional structures. The application has many features for solving a wide range of problems from simple 2-D trusses to complex 3-D structures.

Creation and modification of the model, execution of the analysis, and checking and optimization of the design are all done through this single interface. Graphical displays of the results, including real-time animations of time-history displacements, are easily produced.

SAP2000 will be used to create a generic model of the structure and special links. All the required data will be obtained from local cases and literature and to be used as input data to develop the model. Properties of link will be dependent of each case. The boundary conditions, adequate mesh size, load steps, and analysis type will be calibrated during the validation process.

Fig (4.1) shows the ability of sap program to deal with different types of links such as gap links which will be used through our work:

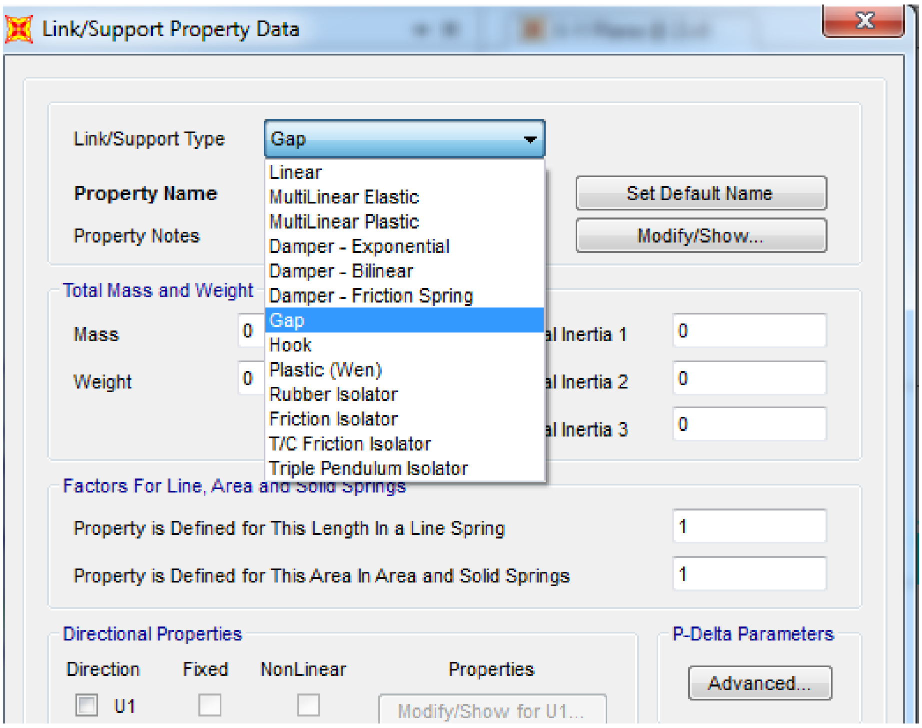


Figure (4. 1): types of links used in SAP2000.

Link to the ground is specified in a Gap Element Model In order to calculate impact force between building and stiff ground during seismic excitation, a gap element needs to connect between them. Gap elements have 2 nodes i and j, or one node only for one node link.

The stiffness of the gap element is generally adopted as 102 to 104 time the stiffness of the adjacent connected element for created models to show the probability of impact, but axial stiffness of link could be calculated for known sections, usually gap element only active in compression phase and it becomes inactive in tension phase. The gap element is active when the gap becomes zero as shown in Fig (4.2)

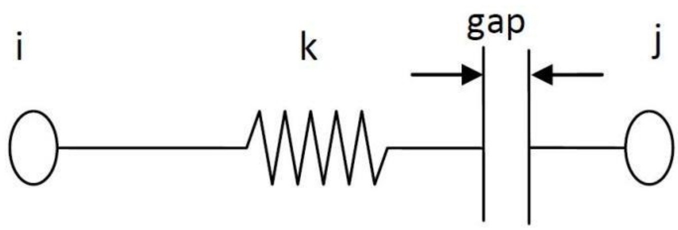


Figure (4. 2): simple model of gap link.

Link performance:

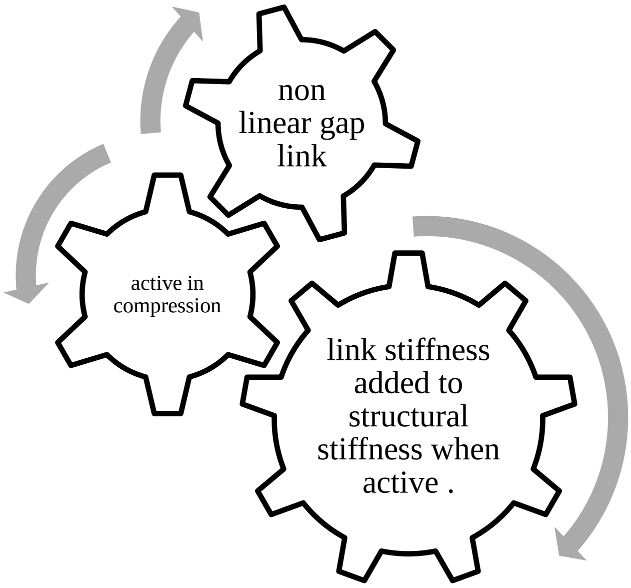


Figure (4. 3) gap link performance

The following charts illustrate the performance of free unrestricted model and restricted model to various types of excitations:

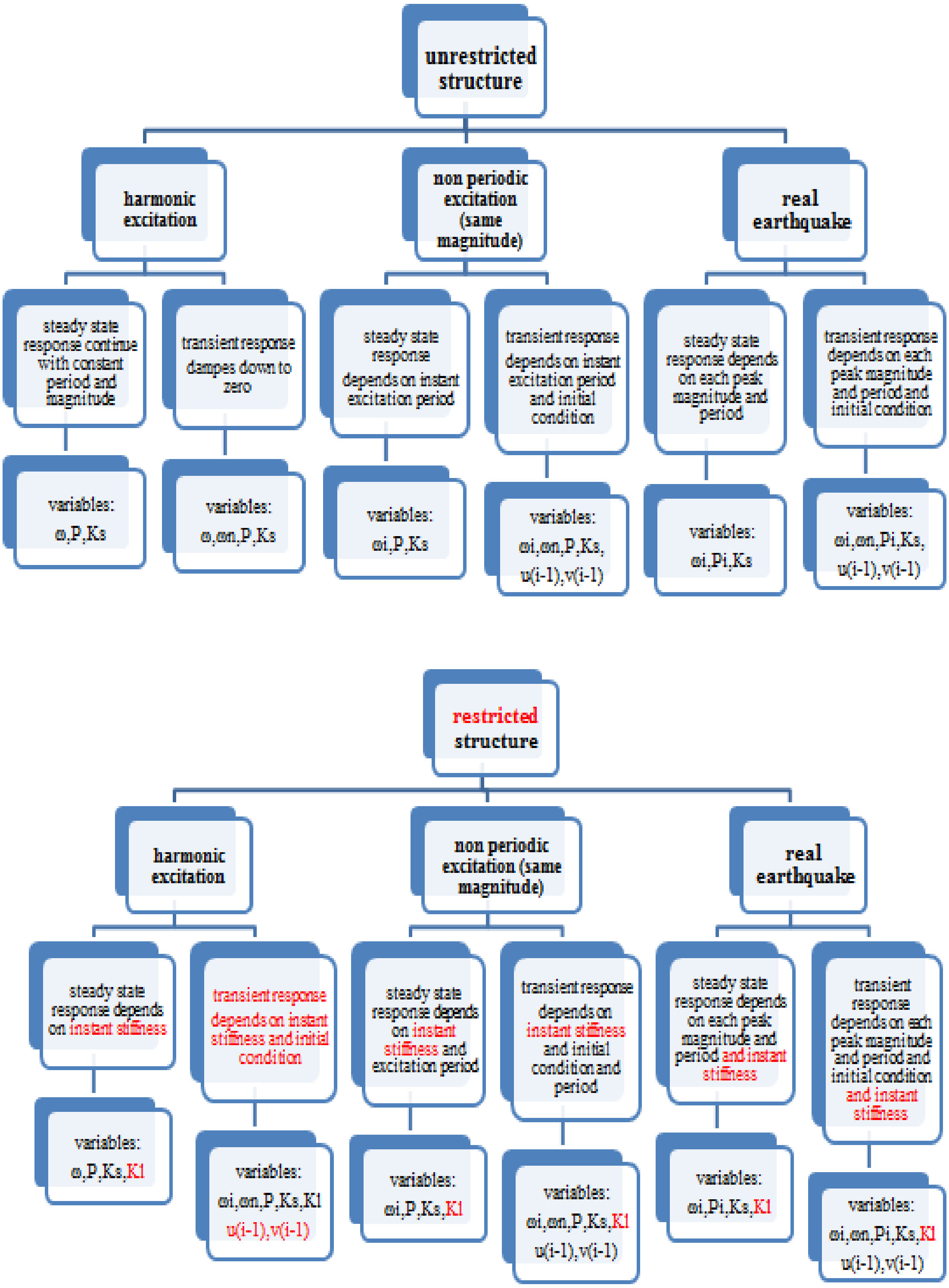


Figure (4. 4): Illustration charts of free and restricted models to various types of excitations.

4.2 Gap elements

Gap elements are two-node elements formulated in three-dimensional space. This element type is only available in a static stress analysis with linear material models.

Two end nodes specified in three-dimensional space define gap elements. Only the axial forces of the element are calculated for each element, and depending on the settings, only compressive forces or only tensile forces are generated. No element-based loading is defined for gap elements.

A compression gap is not activated until the gap is closed; a tension gap is not activated until the gap is opened. Therefore, the structural behavior of a finite element model associated with gap elements is always nonlinear because of its indeterminate condition. Whether the gaps are closed or opened is not known in advance. An iterative solution method is used to determine the status (opened or closed) of the gap elements.

Since the analysis is linear and small deflection theory is used, only motion in the direction of the original gap element orientation is considered. Sideways motion does not affect the status of the gap element.

Stiffness of gap element

Many studies have been carried which suggest various assumptions for assigning stiffness to the spring element. These are illustrated as under:

Wada et al. incorporated a gap element with stiffness equal to the axial stiffness of the beams and slab at the impact level (Wada, A., Shinozaki, Y. and Nakamura, N, 1984).

Anagnostopoulos suggested gap element with stiffness coefficient equal to twenty times the lateral stiffness of the more rigid SDOF system (Anagnostopoulos, 1988).

Maison and Kasai proposed a stiffness value corresponding to the axial stiffness of the floor level at the assumed level of contact (Maison, B.F. and Kasai, K., 1992).

In this study the (Wada et al) suggestion is adopted and the stiffness for the link element is calculated as beam axial stiffness.

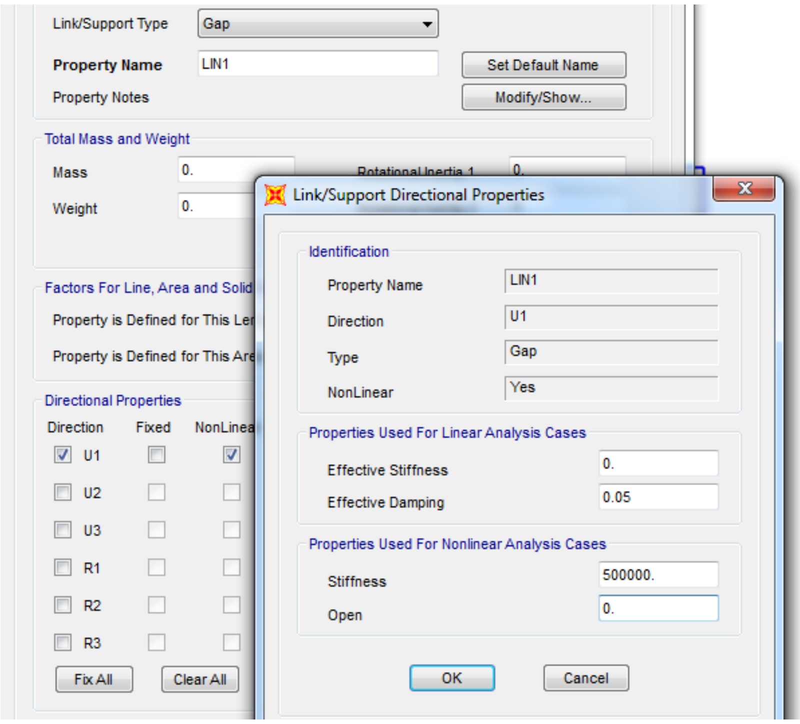


Figure (4. 5): Stiffness of gap element.

4.3 Materials Properties

The strength of plain concrete and steel bars typically, expressed in terms of compressive strength of concrete ($f_{c'}$), and yielding stress of steel (f_y).

For all structural elements composing the assessed models, concrete strength of $f_{c'}$=30MPa, and steel strength of f_y=420MPa are used.

4.4 Loads on the Building

Dead load in addition to seismic loads acting in the horizontal direction will only be considered during the analysis and design of models.

DL is taken as the weight of the structure itself, plus the SDL. The weight of the structure is determined by the foreknowledge of the dimensions of structural members and unit weights. The structural components of models are inherently RC. SDL computed to adjust the natural period of frames for each model.

The harmonic excitation having peak ground acceleration (PGA) of 1.0g is chosen for the time history analysis as shown in Figure (4.6).

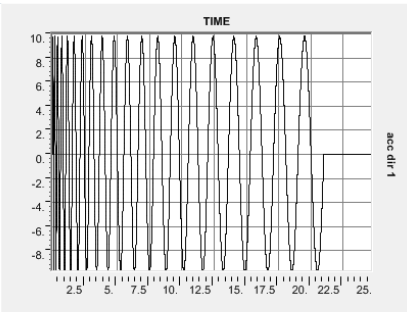


Figure (4. 6): sine harmonic excitation.

4.5 Ground Motion Input Parameters

The time variation of ground acceleration is the most common way of identifying the seismic intensity of earthquakes (Chopra, 2012). In earthquake engineering, ground motion parameters are often defined by the most predicted destructive potential of an earthquake ground motion, i.e. the peak values. Hence, the horizontal peak ground acceleration (PGA) seems a reasonable metric of the ground shaking. PGA is usually given in forms of the seismic zone factor (Z). Z is a dimensionless coefficient of the expected horizontal PGA as (SII, 2009): $Z= PGAg$ Where:

PGA is what experienced by a particular station on rock during an earthquake.

g is the standard acceleration due to gravity ($9.81m/s2$).

According to the NIBS (2012), the ASCE/SEI 7-10 defines the hazard of seismic action based on three parameters. The first two values are dimensionless coefficients (SS,1) of spectral accelerations quantified in terms of 2% of being exceeded in 50 years; 2475-years return period (CHARNEY, 2015). The third value is the spectral time period (TL) that expresses the commencement of long period behavior.

Nevertheless, the basic ground motion parameters (SS,$S1$) corresponding to 10% probability occurs of being exceeded in 50 years (475-years return period) is closer to the low to high seismicity of Palestine. This trend is also prevalent in a number of building codes as in Israel (AMIT, R.,

SALAMON, A., NETZER-COHEN, C., ZILBERMAN, E. & COHEN, M., 2015), Jordan (JIMENEZ, M., AL-NIMRY, H., KHASAWNEH, A., AL-HADID, T. & KAHHALEH, K. 2008, 2008), Saudi Arabia (SBCNC, 2007).

Figure 3.12, however, marks a definite value of Z on the rock for various communities of Palestine, with a reference exceedance probability of 10% in 50 years.

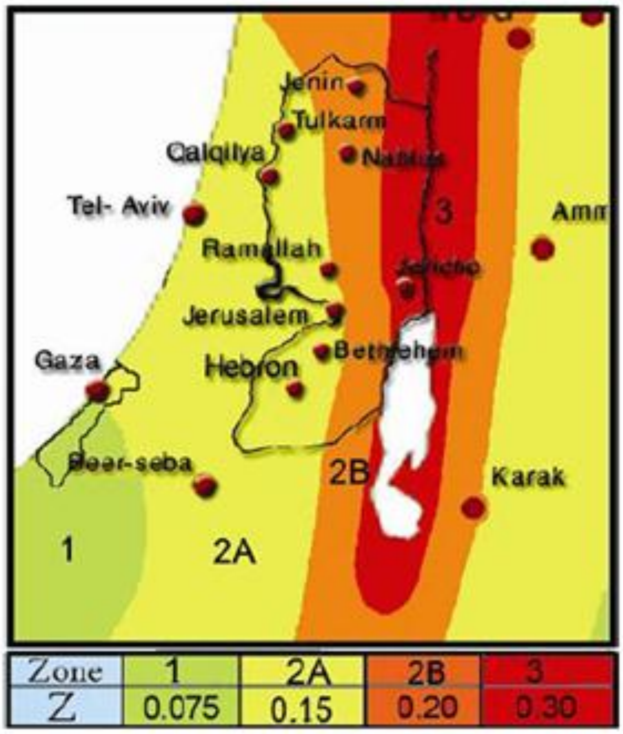


Figure (4. 7): seismic zonation map of Palestine.

SS is the 5% damped, dimensionless coefficient of short time period (T = 0.2sec) horizontal spectral acceleration for rock or site class B (ASCE, 2010).

$S1$ is the 5% damped, dimensionless coefficient of one second period horizontal spectral acceleration for rock or site class B (ASCE, 2010).

TL is a long-transition period in seconds resembles the onset of the constant-displacement spectral plateau (SUCUOGLU, 2015). For Palestinian Territories, TL could be taken as 4.0sec. (SII, 2009)

For this study a time history analysis is applied to the models with a PGA=1.0, because we comparing the behavior of structures rather than measuring values.

4.6 Members Stiffness

Modeling member stiffness upon uncracked section properties deems convenient when analyzing RC framed structures contra gravity loads; cracks propagation under service-vertical loads is somewhat trivial, member forces are inconsiderably affected (PRIESTLEY, M. & PAULAY, T., 1992). In the case of seismic analysis, the conventional design situation is to minimize the moment of inertia of members by a reduction factors inside codes (NIBS, 2009) Section 12.7.3(a) of the ASCE/SEI 7-10 Standards, calls to incorporate the effect of cracking in modeling, even so, neither standards (NIBS, 2012), nor the modern world seismic codes (BOSCO, M., GHERSI, A. & LEANZA, S., 2008) recommend explicit parameters to express the effective stiffness of the members. (PIQUE, J. & BURGOS, M., 2008), (PRIESTLEY, M. & PAULAY, T., 1992) confirmed that the reduction factors inside codes are still inappropriate to visualize the realistic stiffness of members as they do not consider the effect of axial and bending reinforcement. (BOSCO, M., GHERSI, A. & LEANZA, S., 2008)indicated that the role of the coded reduction factors is still doubtful; they lead to non-conservative results. Reduction factors result in decreasing of seismic loads, and, as a result, internal forces in members will be decreased further. On top of this, (BOSCO, M., GHERSI, A. & LEANZA, S., 2008) claimed that (PAULAY, 1997) called to sweep these factors since they do not stand on reliable basis.

In final consideration, the typical practice procedure accepts to utilize members stiffness based on the gross uncracked section properties (PIQUE, J. & BURGOS, M., 2008).

4.7 Base Fixity

In seismic analysis problems, ground motion is presupposed to be recognized and not depending on the response of the structure. This is analogues to say that “foundation soil is rigid, implying no soil-structure interaction”, except where the structure is constructed on “very flexible soil” where the vibration of structure affects the base motion (Chopra, 2012). In the final analysis, the targeted soil profile types in the research are compatible with the assumption of fixed-base models.

4.8 Modeling Phase

Only structural components are involved in modeling, all beams and columns are modeled using line elements.

- Axial, shear, flexural, and torsional deformations are involved.
- All columns are fully fixed with foundations.
- Self-weights of slabs, beams, and columns are not added, the software considers them automatically.
- SDL contributions are represented by entering a uniformly distributed line load.

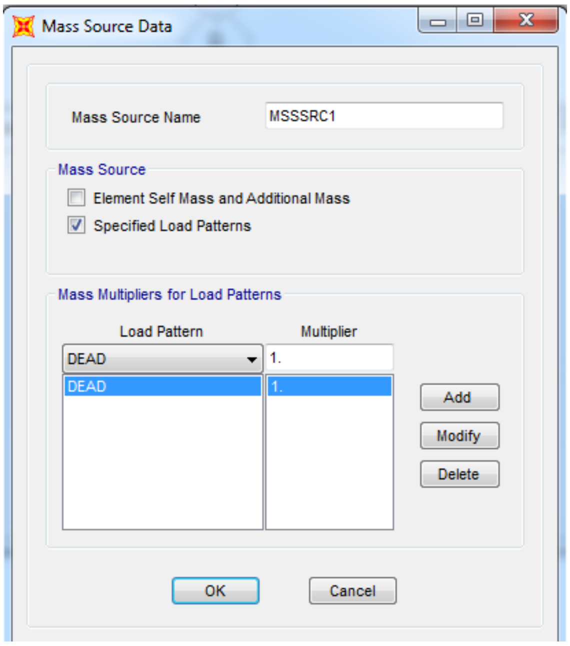


Figure (4. 8): mass source data.

4.9 Finite Element Mesh Sensitivity Analysis

Operating the finite element method (FEM) for analysis, displays inaccuracies between the supposed answers and the upcoming results. The accuracy of results depends mainly on the mesh density or elements size. Nevertheless, high mesh densities complicate the model, and time-consuming. However, it is advisable to balance between the accuracy related to meshing and the time it takes to run, and to analyze the model (CORONADO, C., REIGLES, D., BAE, S. & MUNSHI, J, 2011).

For this reason, mesh sensitivity study is performed to detect the appropriate level of meshing able to produce static and dynamic parameters within a reasonable domain of error.

To do that, frame elements will be subdivided into 10 elements for each columns and beams.

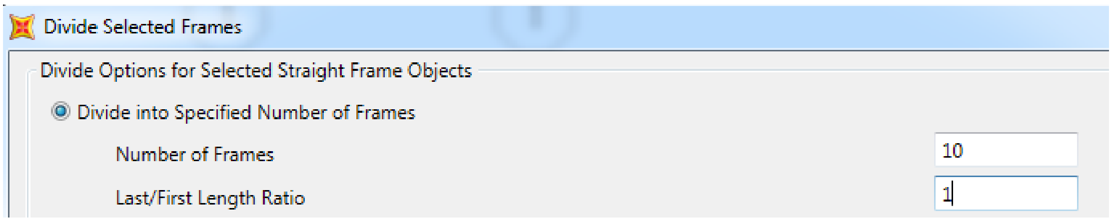


Figure (4. 9): frame subdivision.

4.10 Models Checking Process

By the universality of analysis and design of building structures, increased demand is placed on the computer software. "Whichever analysis method is adopted during design, it must always be controlled by the designer, i.e. not a computer!" (MCKENZIE, 2013). Thus, computerized results obtained with reliance on non-checked models have to be rejected, even if they look as pretty answers.

Honestly, the producers of SAP2000 specified an acceptance criterion (CSI, 2017a) for any independent value compared to that obtained by the program as follows:

- External forces and moments. The difference shall not exceed 5% between an exact and approximate solution.

- Internal forces and moments. The difference shall not exceed 10% between two approximate solutions having similar hypothesis.

- For experimental values. The difference shall not exceed 25% between two approximate solutions having dissimilar hypothesis.

These percentages, however, should not be exceeded during the verification of the computerized answers. Otherwise, one should look for reasons!

4.11 Earthquake Consequences on Structures

The response of a structure to a ground motion activity depends on its natural period (T_n) and damping ratio (ζ) (Chopra, 2012). Therefore, the determination of these two parameters is the first step towards any earthquake analysis and design process.

4.12 The Fundamental Natural Period

Natural period Tn is the time taken by undamped system to complete one cycle during free vibration. The fundamental time period ($T1$) of building skeletons refers to the first mode period which is always the longest modal time of vibration in the horizontal direction of interest. Time periods for the first mode and the subsequent modes of 2D models are gained from most structural analysis computer software. Periods calculated by a rigorous mathematical modeling of RC structures are, obviously, highly sensitive to stiffness assumptions. To make sure that significant low design base shear is not due to a doubtful long time period caused by either unrealistic stiffness reduction factors (GHOSH, S. K. & FANELLA, D. A., 2003), or unduly modeling simplifications (NIBS, 2012), or undetected modeling errors (NIBS, 2009), building codes impose a limit on the fundamental periods produced by rational structural analysis.

4.13 Damping

Once the seismic activity on building decays, the amplitude of vibration dies away steadily with time. This form of energy dissipation is called damping. For civil engineering structures, ζ is a unit less measure of damping (Chopra, 2012) with a value less than 10%. A near-universal assumption, yet, is that ζ=5% (WILLIAMS, 2016). This percent is also explicitly applied for each mode inside SAP2000.

For this study =5%, for each frames and links.

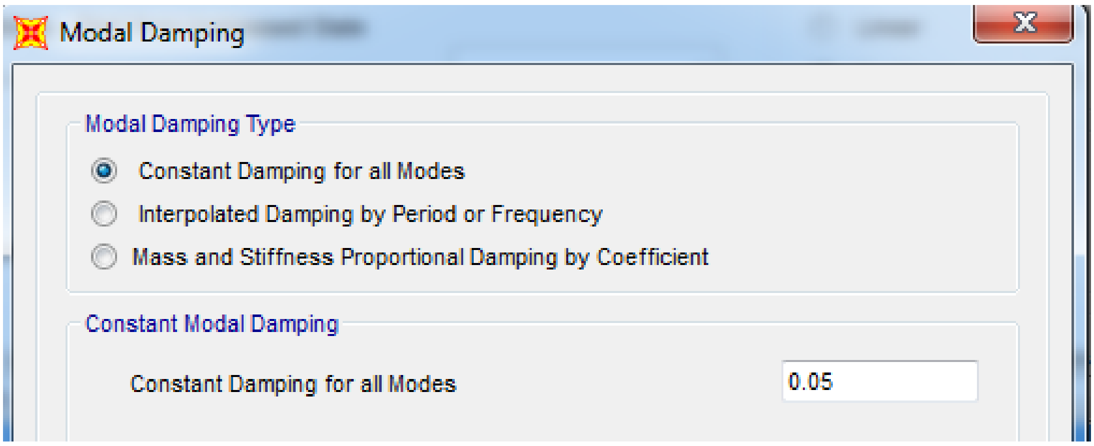


Figure (4. 10): modal damping.

4.14 Minimum Number of Modes

In general, it is not necessary to carry all the higher modes for the superposition process. According to Section 12.9.1 of the (ASCE/SEI7, 2010) Standards, the minimum number of modes required to analyze the MDOF system is such that their accumulated effective modal mass account for up to 90 percent of the actual mass, separately in X and Y directions.

In this study, the sample models restricted to 2D frames, the number of modes chosen for each model equals its number of degrees of freedom plus1, neglecting the Z direction.

For each model the modes where checked to have the same shapes as suggested in chapter 3.

For single degree of freedom model, 2 modes used in the time history nonlinear analysis, to consider the link behavior.

For higher degrees of freedom, there is no option to choose arbitrary mode to use in analysis, but one can restrict the number of modes respecting its order, in other words, you can use the first mode only, or the first and second modes only, or the three modes.

This procedure allowed us to check the behavior of first mode alone as illustrated in chapter 3, and the effect of adding multi modes to the analysis would be checked in the sensitivity study later.

Chapter Five
Model Verification

5.1 General

Physical modeling, advanced mathematics and interpretation of results are some demands of the dynamic analysis compared to those of static analysis which in most often are hand-based techniques. Therefore, the dependency on software developed solutions to structural dynamics is inevitable and unavoidable. Nevertheless, the above reasoning does not exempt from an evidencing of results.

In this chapter, the behavior of link will be verified by verifying the results of the single degree of freedom model, then a check to fundamental modes will be done to the multi degrees of freedom models.

It's considerable to note that a complete verification of gap links is available by CSI document "Example 6-003" shown in figure below. (CSI, 2003)

Software Verification

PROGRAM NAME: SAP2000
REVISION NO.: 15

EXAMPLE 6-003

LINK – GAP ELEMENT

PROBLEM DESCRIPTION

This example uses a single-bay, single-story rigid frame to test the gap link element. This link element carries compression loads only; it has zero stiffness when subjected to tension. The gap element is paced at the bottom of the right-hand column in the frame. The frame is then loaded with a gravity load P (10 kips) at the center of the beam. Once the full load P is applied, a lateral load V (20 kips) is applied, pushing the frame from right to left. The compression load in the gap element after the full load P has been applied and the uplift at the gap after the full load V has been applied are compared with independent hand calculated results.

Figure (5. 1): CSI software verification document.

5.2 Verification of Fundamental Periods

5.2.1 Single Degree Of Freedom Model

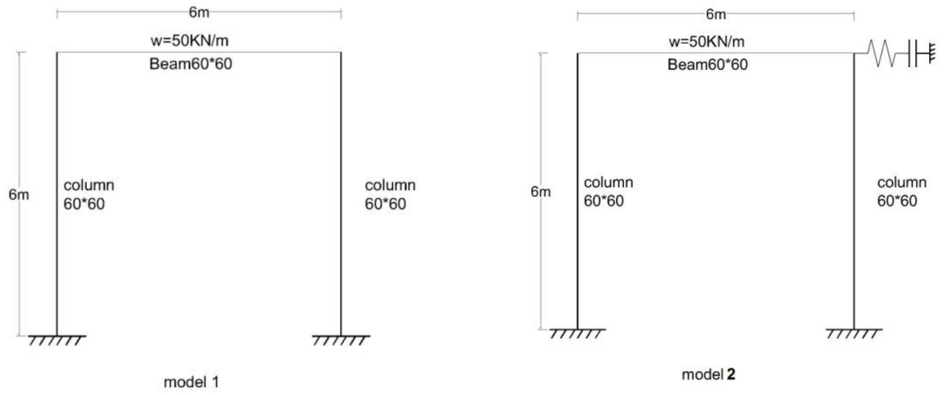


Figure (5. 2): single degree of freedom models.

Mass of the structure:

W= Beam load+ weight of beam+ weight of columns

W=(50kN/m*6m) + (25kN/m^3*0.6m*0.6m*6m) +(25kN/m^3*0.6m*0.6m *6m*2) = 300+54+108 = 462 kN

M=46200 kg. =46200 (N.sec^2/m) = 46.2 (N.sec^2/mm)

Stiffness of structure:

Assume concrete 30MPa

$E_{concrete} = 4700*\sqrt{30}$ =25743 MPa

I_x (for 60*60 cm^2 column) = 1/12 * 600*6003 = $1.08*10^{10}$ mm^4

$K = 12EI/L^3 = 12*25743(N/mm^2)*10,800,000,000(mm^4)/6000^3(mm^3)$ $=15445.8$ N/mm

K_x (storey) = 2*15445.8 = 30891.6 N/mm

For model 1: (free vibration)

period $T_n=2*\pi \sqrt{\frac{m}{K}} = 2*\pi \sqrt{\frac{46.2}{30891.6}} = 0.243$ sec

Natural period computed by sap2000 =0.23253 sec.

For model 2: (restricted vibration)

1st period $T_n=2*\pi \sqrt{\frac{m}{K}} = 2*\pi \sqrt{\frac{46.2}{30891.6}} = 0.243$ sec.

For link stiffness = 500000 N/mm

2nd period $T_n=2*\pi \sqrt{\frac{m}{K}} = 2*\pi \sqrt{\frac{46.2}{530891.6}} = 0.0586$ sec.

5.2.2 Two Degrees Of Freedom Model

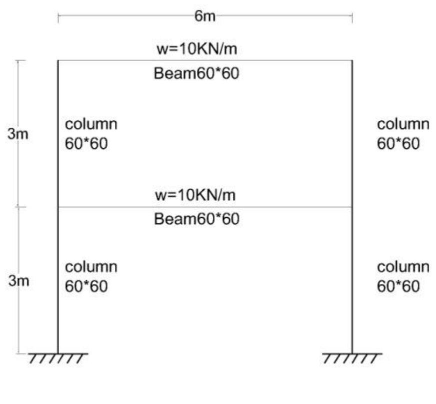


Figure (5. 3): two degree of freedom model.

Mass of the structure:

W1(1^{st} storey) = Beam load+ weight of beam+ weight of columns

=(10kN/m*6m) + (25kN/m^3*0.6m*0.6m*6m) +(25kN/m^3*0.6m*0.6m *3m*2) = 60+54+54 = 168 kN

M1 = 16800 kg. =16800 (N.sec^2/m) = 16.8 (N.sec^2/mm)

W2(2^{nd} storey) = Beam load+ weight of beam+ 0.5 * weight of columns

W2=60+54+27=141 kN

M2=14100 kg. = 14100 (N.sec^2/m) = 14.1 (N.sec^2/mm)

Stiffness of structure:

Assume concrete 30MPa

$E_{concrete}$ = 4700*$\sqrt{30}$ =25743 MPa

I_y (for 60*60 cm^2 column) = 1/12 * 600*600^3 = 1.08*10^{10} mm^4

K = 12EI/L^3 = 12*25743(N/mm^2)*10800000000(mm^4)/3000^3(mm^3) =123566.4 N/mm

K_x (for typical storey) = 2*123566.4 = 247132.8 N/mm

$$M=\begin{Bmatrix} m1 & 0 \\ 0 & m2 \end{Bmatrix}=\begin{bmatrix} 16.8 & 0 \\ 0 & 14.1 \end{bmatrix}$$

$$K=\begin{Bmatrix} k1+k2 & -k2 \\ -k2 & k2 \end{Bmatrix}=\begin{bmatrix} 4.942656\ 10^5 & -2.471328\ 10^5 \\ -2.471328\ 10^5 & 2.471328\ 10^5 \end{bmatrix}$$

Eigen values = $V := \mathrm{Re}(evalf(Eigenvectors(k, m, output = values)))$

$$V := \begin{bmatrix} 40596.7321159453 \\ 6350.98824879637 \end{bmatrix}$$

Eigen Vectors = $ivec := \mathrm{Re}(evalf(Eigenvectors(k, m, output = vectors)))$

$$ivec := \begin{bmatrix} 1. & 0.637648526184995 \\ -0.759751435454462 & 1. \end{bmatrix}$$

Natural periods of structure:

$$T1 := \frac{2\pi}{(6350.98824879637)^{0.5}} = 0.07884228707 \text{ sec.}$$

$$T2 := \frac{2\pi}{(40596.7321159453)^{0.5}} = 0.03118418013 \text{ sec.}$$

Table (5. 1): Sap2000 results for 2DOF model.

OutputCase	StepType Text	StepNum Unitless	Period Sec	Frequency Cyc/sec	CircFreq rad/sec	Eigenvalue rad2/sec2
MODALritz	Mode	1	0.078339	12.7649540...	80.2045719...	6432.77335...
MODALritz	Mode	2	0.030725	32.5471283...	204.499638...	41820.1023...

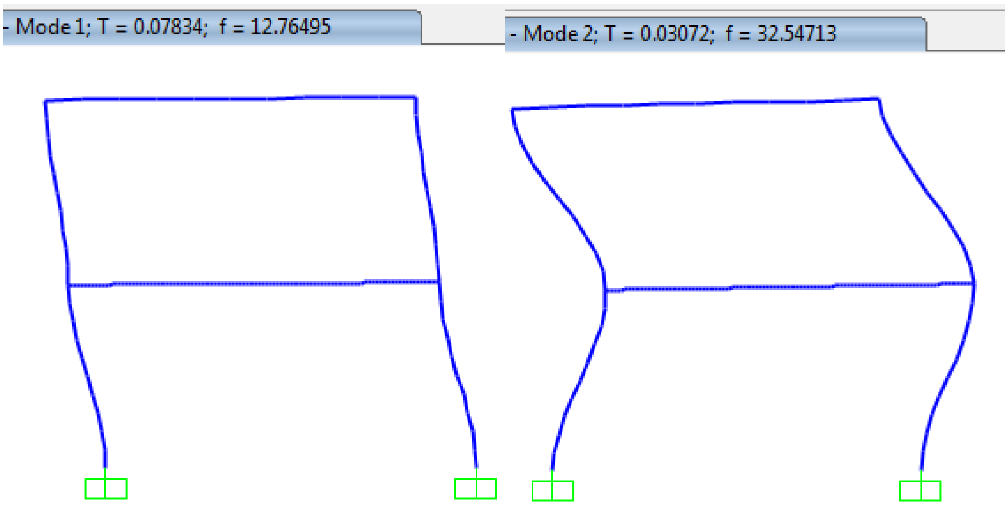


Figure (5. 4): mode shapes of 2DOF model for 1st and 2nd modes.

5.2.3 Three Degrees of Freedom Model

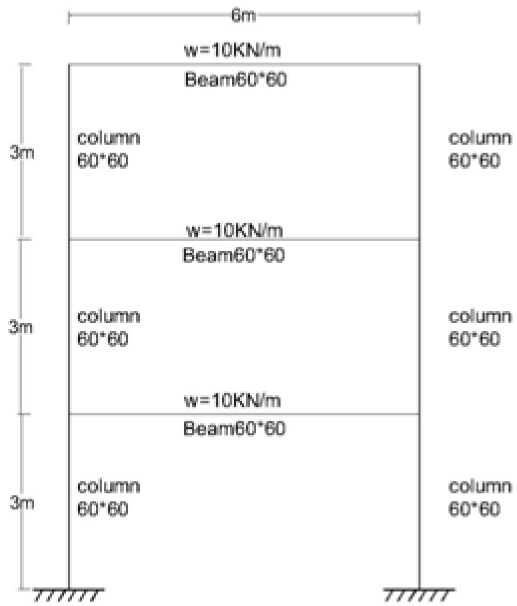


Figure (5. 5): three degrees of freedom model.

W1=W2 = Beam load+ weight of beam+ weight of columns

=(10kN/m*6m) + (25kN/m^3*0.6m*0.6m*6m) +(25kN/m^3*0.6m*0.6m *3m*2) = 60+54+54 = 168 kN

M1=M2 = 16800 kG =16800 (N.sec^2/m) = 16.8 (N.sec^2/mm)

W3= Beam load+ weight of beam+ 0.5 * weight of columns

W3=60+54+27=141 kN

M3=14100 kg. = 14100 (N.sec^2/m) = 14.1 (N.sec^2/mm)

$$\text{Mass matrix} = M = \begin{bmatrix} m1 & 0 & 0 \\ 0 & m2 & 0 \\ 0 & 0 & m3 \end{bmatrix} = \begin{bmatrix} 16.800 & 0 & 0 \\ 0 & 16.800 & 0 \\ 0 & 0 & 14.100 \end{bmatrix}$$

Assume concrete 30MPa

$E_{concrete}$ = 4700*$\sqrt{30}$ =25743 MPa

I_x (for 60*60 cm^2 column) = 1/12 * 600*600^3 = $1.08*10^{10}$ mm^4

K = $12EI/L^3$ = 12*25743(N/mm^2)*10800000000(mm^4)/3000^3(mm^3) =123566.4 N/mm

K_x (for typical storey) = 2*123566.4 = 247132.8 N/mm

Stiffness matrix = K= $\begin{bmatrix} k1+k2 & -k2 & 0 \\ -k2 & k2+k3 & -k3 \\ 0 & -k3 & k3 \end{bmatrix} = \begin{bmatrix} 494265.6 & -247132.8 & 0 \\ -247132.8 & 494265.6 & -247132.8 \\ 0 & -247132.8 & 247132.8 \end{bmatrix}$

Eigen Values = V := Re(evalf(Eigenvectors(k, m, output = values)))

$$V := \begin{bmatrix} 48816.9510165801 \\ 24362.2621239693 \\ 3189.07865276367 \end{bmatrix}$$

Eigen Vectors = ivec := Re(evalf(Eigenvectors(k, m, output = vectors)))

$$ivec := \begin{bmatrix} 0.758403682890389 & -1. & -0.458751534353617 \\ -1. & -0.343862070584380 & -0.818049206726230 \\ 0.560155314857160 & 0.881758876413423 & -1. \end{bmatrix}$$

Natural Periods of Structure:

$$T1 := \frac{2\pi}{(3189)^{0.5}} = 0.1112634722 \text{ sec.}$$

$$T2 := \frac{2\pi}{(24362)^{0.5}} = 0.04025533073 \text{ sec.}$$

$$T3 := \frac{2\pi}{(48816.95)^{0.5}} = 0.02843770518 \text{ sec.}$$

Table (5. 2): SAP2000 results for 3DOF model.

OutputCase	StepType Text	StepNum Unitless	Period Sec	Frequency Cyc/sec	CircFreq rad/sec	Eigenvalue rad2/sec2
MODAL	Mode	1	0.118649	8.42823640...	52.9561711...	2804.35606...
MODAL	Mode	2	0.042928	23.2950098...	146.366863...	21423.2588...
MODAL	Mode	3	0.030326	32.9753503...	207.190237...	42927.7943...

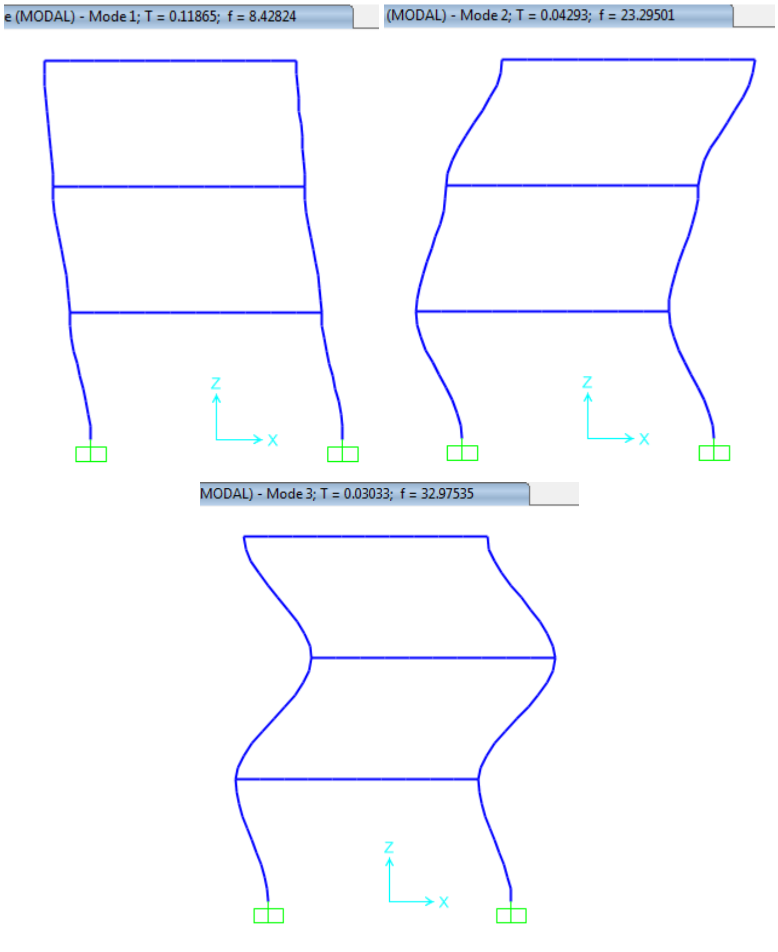


Figure (5. 6): mode shapes of 3DOF model for 1st, 2nd and 3rd modes.

5.3 Verification Of Displacement Results

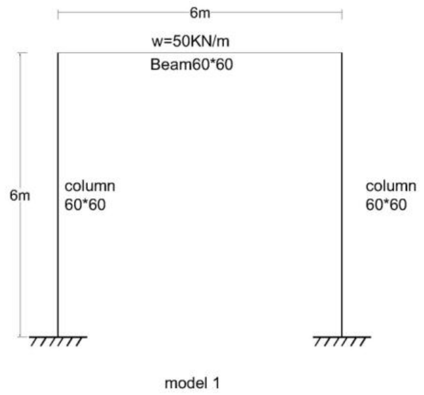


Figure (5. 7): Single degree of freedom model.

For unrestricted vibration: there is one natural period of the structure depends on its stiffness and mass, displacement contains two distinct vibration components:

The sin(ωt) term, giving an oscillation at the forcing or exciting frequency;And the sin(ω_nt) and cos(ω_nt) terms, giving an oscillation at the natural frequency of the system.

$$F := P0\cdot\sin(\omega\cdot t)$$

$$u := e^{-\xi\cdot\omega n\cdot t}\cdot(A\cdot\cos(\omega D\cdot t) + B\cdot\sin(\omega D\cdot t)) + C\cdot\sin(\omega\cdot t) + d\cdot\cos(\omega\cdot t)$$

$$\omega D = \omega n\cdot\left(\sqrt{1-\xi^2}\right)$$

$$uc := e^{-\xi\cdot\omega n\cdot t}\cdot(A\cdot\cos(\omega D\cdot t) + B\cdot\sin(\omega D\cdot t))$$

$$up := C\cdot\sin(\omega\cdot t) + d\cdot\cos(\omega\cdot t)$$

By substituting these variables into the complementary component

$$subs\left(A = u0, B = \frac{v0}{\omega n} - \frac{P0}{k}\cdot\left(\frac{\frac{\omega}{\omega n}}{1-\left(\frac{\omega}{\omega n}\right)^2}\right), \omega D = \omega n\cdot\left(\sqrt{1-\xi^2}\right), uc\right)$$

$$uc := e^{-\xi\,\omega n\,t}\left(u0\cos\left(\omega n\sqrt{-\xi^2+1}\;t\right) + \left(\frac{v0}{\omega n} - \frac{P0\,\omega}{k\cdot\omega n\left(1-\frac{\omega^2}{\omega n^2}\right)}\right)\sin\left(\omega n\sqrt{-\xi^2+1}\;t\right)\right):$$

$$C := \frac{P0}{k}\cdot\frac{\left(1-\left(\frac{\omega}{\omega n}\right)^2\right)}{\left(1-\left(\frac{\omega}{\omega n}\right)^2\right)^2+\left(2\cdot\xi\cdot\left(\frac{\omega}{\omega n}\right)\right)^2}:$$

$$d := \frac{P0}{k}\cdot\frac{\left(-2\cdot\xi\cdot\left(\frac{\omega}{\omega n}\right)\right)}{\left(1-\left(\frac{\omega}{\omega n}\right)^2\right)^2+\left(2\cdot\xi\cdot\left(\frac{\omega}{\omega n}\right)\right)^2}:$$

By substituting these variables into the particular component

$$up := \frac{P0\left(1-\frac{\omega^2}{\omega n^2}\right)\sin(\omega\,t)}{k\left(\left(1-\frac{\omega^2}{\omega n^2}\right)^2+\frac{4\,\xi^2\,\omega^2}{\omega n^2}\right)} - \frac{2\,P0\,\xi\,\omega\cos(\omega\,t)}{\omega n\left(\left(1-\frac{\omega^2}{\omega n^2}\right)^2+\frac{4\,\xi^2\,\omega^2}{\omega n^2}\right)k}$$

The first of these is the free vibration or transient vibration, which depends on the initial displacement and velocity.

The latter is the forced vibration or steady-state vibration, for it is present because of the applied force no matter what the initial conditions.

By substituting model properties into these equations:

$$uc1 := subs\left(\omega = 20\,\pi, \xi = 0.05, u0 = 0, v0 = 0\,, \omega n = \frac{2\,\pi}{0.23253}, P0 = 453.32, k = 30891.6, uc\right)$$

$$uc1 := 0.007742807077\,e^{-1.351048318\,t}\sin(26.98716902\,t)$$

$vc1 := diff(uc1, t)$

$vc1 := -0.01046090648\,e^{-1.351048318\,t}\sin(26.98716902\,t) + 0.2089564433\,e^{-1.351048318\,t}\cos(26.98716902\,t)$

$up1 := subs\left(\omega = 20\,\pi, \xi = 0.05, \omega n = \dfrac{2\,\pi}{0.23253}, P0 = 453.32, k = 30891.6, up\right)$

$up1 := -0.003320565526\sin(20\,\pi\,t) - 0.0001752048063\cos(20\,\pi\,t)$

$vp1 := diff(up1, t)$

$vp1 := -0.06641131052\,\pi\cos(20\,\pi\,t) + 0.003504096126\,\pi\sin(20\,\pi\,t)$

$plot((up1 + uc1), t = 0..1)$

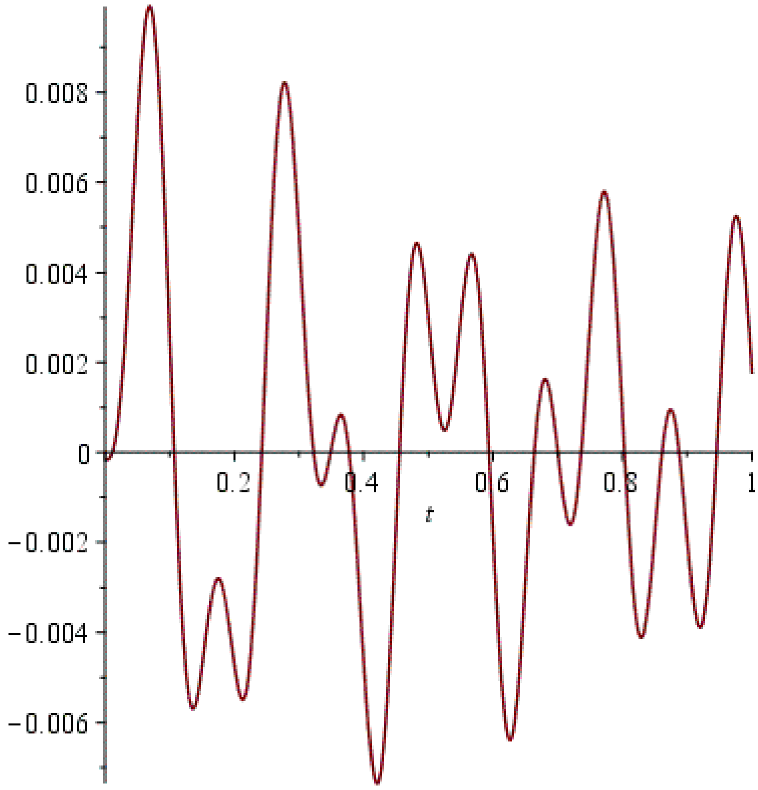


Figure (5. 8): plot of the sum of particular and complementary components of displacement.

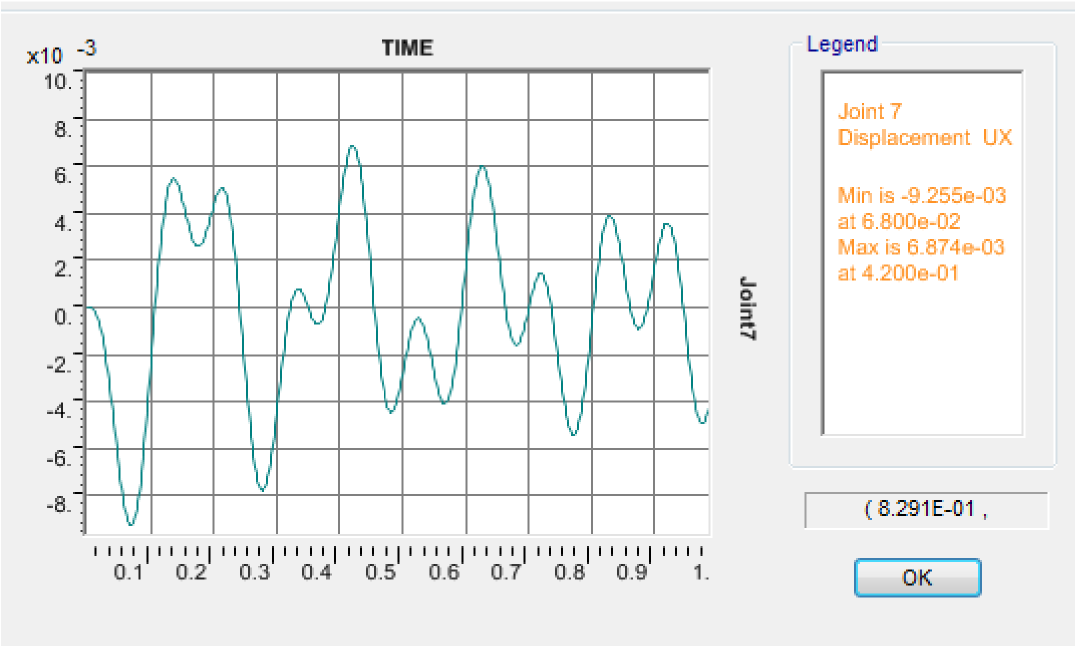


Figure (5. 9): displacement results as per SAP2000 (opposite sign).

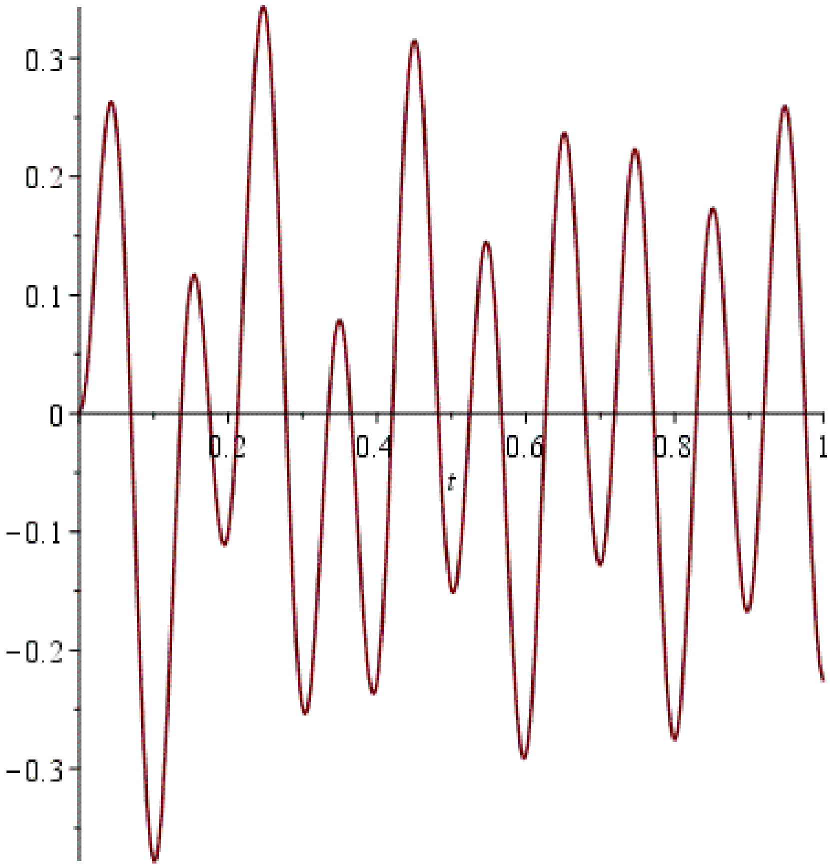


Figure (5. 10): plot of the sum of particular and complementary components of velocity.

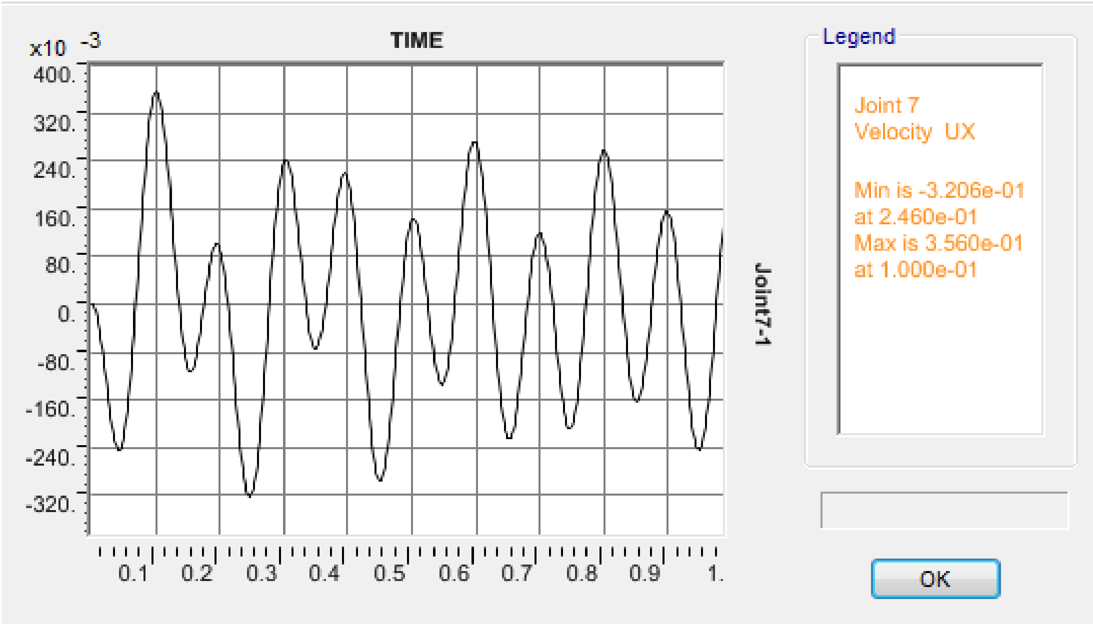


Figure (5. 11): velocity results as per SAP2000 (opposite sign).

5.4 Verification Of Link Behavior

For restricted vibration: stiffness changes related to change in the contact condition of link:

First the structure performs in its natural period out of the link and have displacement equal to free vibration case, because of same parameters and zero initial conditions in the two cases. At the instant the structure returns to its origin and start to drift towards the connection, the structure will perform in new period because of the additional stiffness added, starting with initial velocity equals the velocity of the structure at the end of phase 1, When the compression force in the link released, and the structure tend to vibrate out of the link again, it vibrates in its original period with initial velocity gained from the previous phase.

By substituting the results from 1st step into displacement equation:

$$uc2 := subs\left(\omega = 20\,\pi, \xi = 0.05, u0 = 0, v0 = 0.356\,, \omega n = \frac{2\,\pi}{0.0586}, P0 = 453.32, k = 530891.6, uc\right)$$

$$uc2 := 0.002558159826\,e^{-5.361079615\,t}\sin(107.0874814\,t)$$

$$vc2 := diff(uc2, t)$$

$$vc2 := -0.01371449850\,e^{-5.361079615\,t}\sin(107.0874814\,t) + 0.2739468928\,e^{-5.361079615\,t}\cos(107.0874814\,t)$$

$$up2 := subs\left(\omega = 20\,\pi, \xi = 0.05, \omega n = \frac{2\,\pi}{0.0586}, P0 = 453.32, k = 530891.6, up\right)$$

$$up2 := 0.001290179185\sin(20\,\pi\,t) - 0.0001151447452\cos(20\,\pi\,t)$$

$$vp2 := diff(up2, t)$$

$$vp2 := 0.02580358370\,\pi\cos(20\,\pi\,t) + 0.002302894904\,\pi\sin(20\,\pi\,t)$$

$$plot((up2 + uc2), t = 0\,..1)\ ,((vc2+vp2),t=0..1)$$

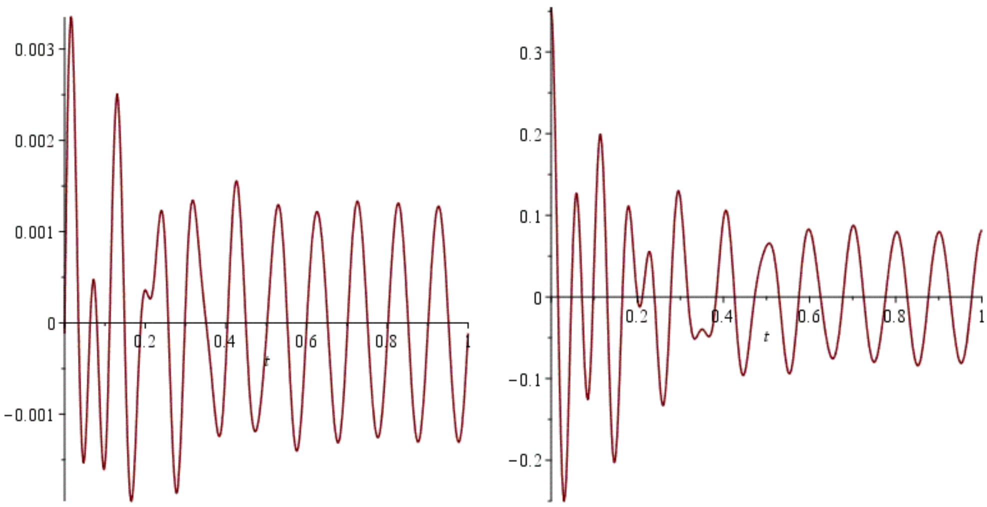


Figure (5. 12): displacement (left) and velocity (right) results for 2nd step.

By substituting the results from 2nd step into displacement equation:

$$uc3 := subs\left(\omega = 20\,\pi, \xi = 0.05, u0 = 0, v0 = 0.2, \omega n = \frac{2\,\pi}{0.23253}, P0 = 453.32, k = 30891.6, uc\right)$$

$$uc3 := 0.01514446686\,e^{-1.351048318\,t}\sin(26.98716902\,t)$$

$$vc3 := diff(uc3, t)$$

$$vc3 := -0.02046090648\,e^{-1.351048318\,t}\sin(26.98716902\,t) + 0.4087062869\,e^{-1.351048318\,t}\cos(26.98716902\,t)$$

$$up3 := subs\left(\omega = 20\,\pi, \xi = 0.05, \omega n = \frac{2\,\pi}{0.23253}, P0 = 453.32, k = 30891.6, up\right)$$

$$up3 := -0.003320565526\sin(20\,\pi\,t) - 0.0001752048063\cos(20\,\pi\,t)$$

$$vp3 := diff(up3, t)$$

$$vp3 := -0.06641131052\,\pi\cos(20\,\pi\,t) + 0.003504096126\,\pi\sin(20\,\pi\,t)$$

$$plot((up3 + uc3), t = 0..1)\ , ((vc2+vp2),t=0..1)$$

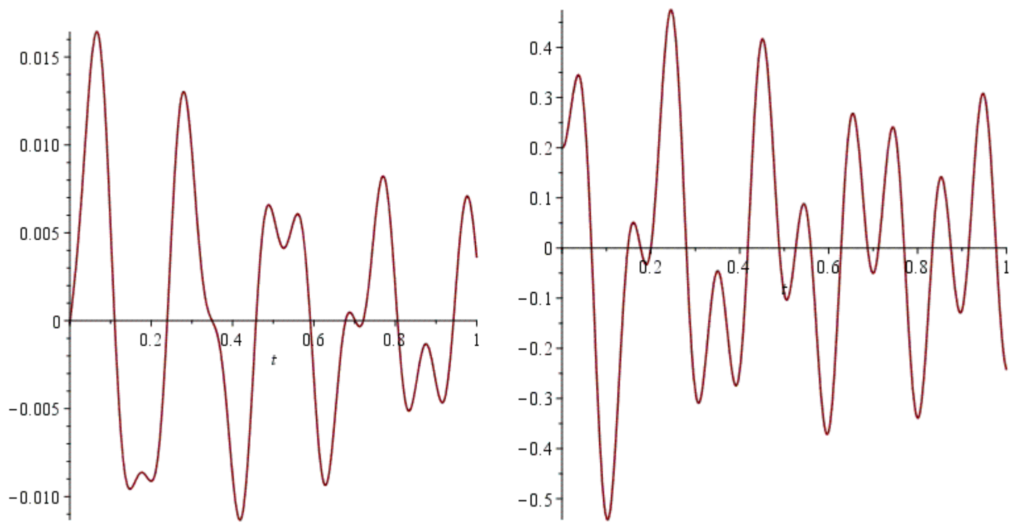


Figure (5. 13): displacement (left) and velocity (right) results for 3rd step.

By applying superposition to the three steps above, the results will match the plot of displacement as per SAP2000 program.

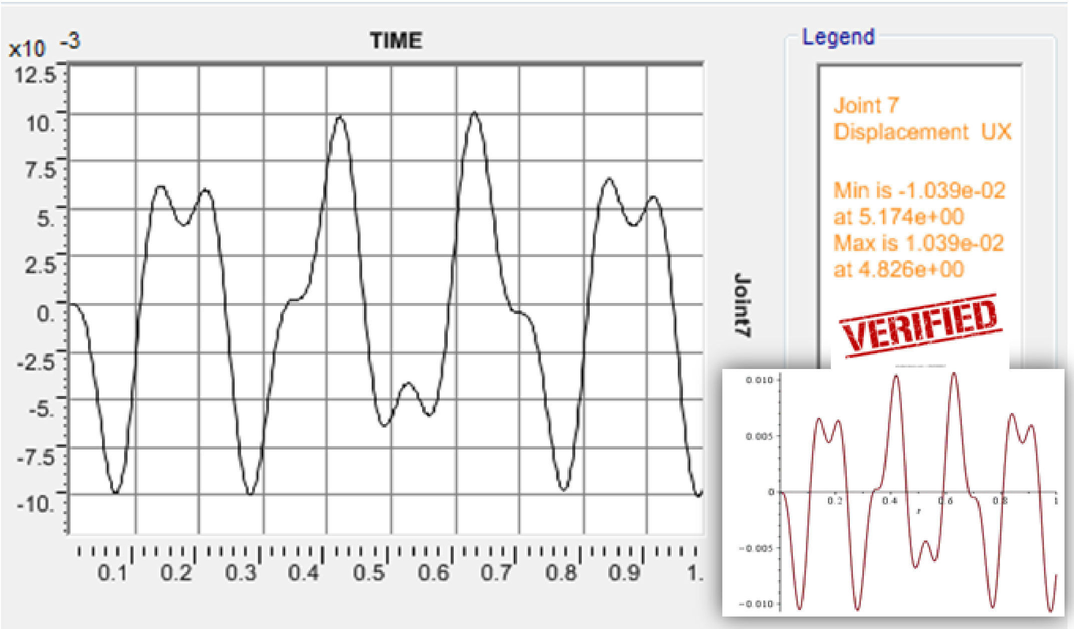


Figure (5. 14): displacement results verification of unrestricted case.

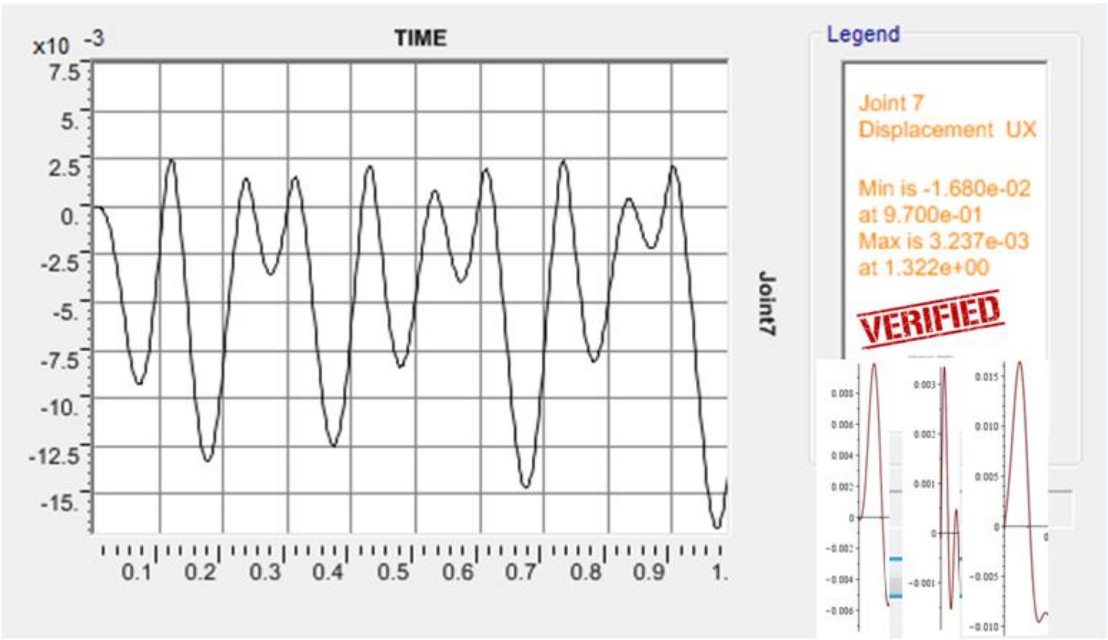


Figure (5. 15): verification of displacement results for restricted case.

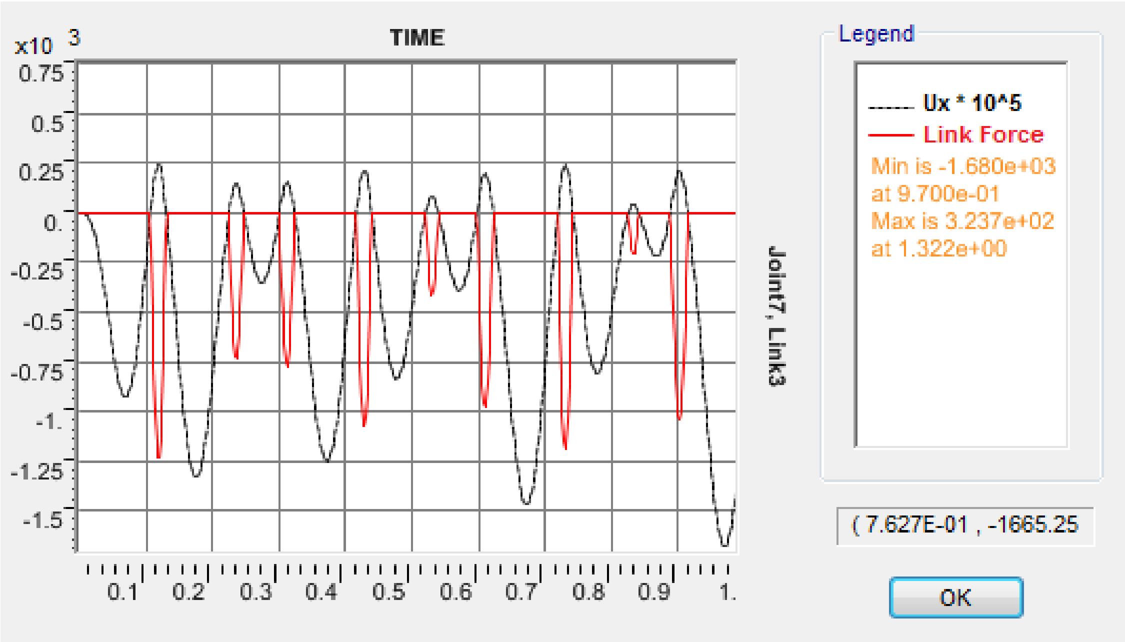


Figure (5. 16): links axial force due to impact over displacement plot.

The above results ensure that the link will have the same performance as expected in previous chapter.

Chapter Six
Structural Analysis

6.1 General

For the purpose of this study, two measurements approaches will be applied to all possible configurations for link location for single, two and three degrees of freedoms models:

- Response spectrum approach: The procedure to compute the peak response of an *N*-story building with plan symmetric about two orthogonal axes to earthquake ground motion along an axis of symmetry, characterized by a response spectrum or design spectrum.

This procedure measuring the response of a structure to a specific excitation, to give this approach a physical meaning: a multi periodic harmonic sine wave containing periods from 0.1sec to 2sec with steps of 0.1 sec with same amplitudes will be applied to each model of the same configuration with different natural periods, where 0.1sec< T_n <1sec.

- Frequency-response approach: A plot of the amplitude of a response quantity against the excitation frequency is called a frequency-response curve.

This procedure measures the response of a structure to each excitation period individually, then it would give a plot of each response to the ratio of T/T_n, the variable here is the excitation period which would be applied to a specific model with determined unrestricted natural period.

A plot of each response would be created to the ratio $0.1<T/T_n<2$.

6.2 Description of the Studied models:

A series of frames models would be analyzed in the two approaches as the table below:

Table (6. 1): total models used in analysis.

Frame type	configuration	Response spectrum approach	Frequency-response approach
SDOF	unrestricted	25 models	25 models
SDOF	Link at 1st storey	50 models	125 models
2DOF	unrestricted	10 models	16 models
2DOF	Link at 1st storey	10 models	16 models
2 DOF	Link at 2nd storey	10 models	16 models
3 DOF	unrestricted	10 models	16 models
3 DOF	Link at 1st storey	10 models	16 models
3 DOF	Link at 2nd storey	10 models	16 models
3 DOF	Link at 3rd storey	10 models	16 models

In all models the frame is 6m bay length with 3m height of each storey, except for SDOF models the height is 6m to avoid floating of program for short periods <0.05 sec. In every model, beams and columns are kept in the same size 60cm*60cm. It should be noted that the dimensions have been gotten after a number of iterations so that, they are expected to realize the forthcoming requirements and checks.

6.3 Single Degree Of Freedom Frames

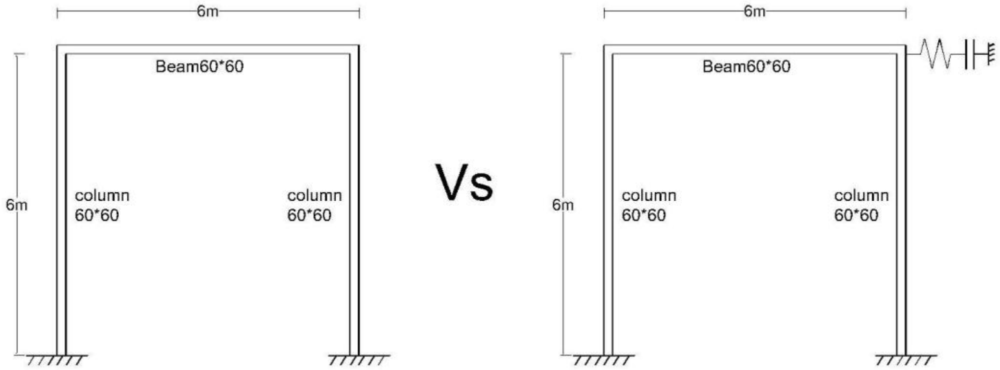


Figure (6. 1): unrestricted Vs restricted models.

6.3.1 Response Spectrum Analysis

Ideal harmonic multi periodic excitation and real earthquake excitation will be analyzed here.

Functions are discussed here, as well the response of the system, regarding time history, and frequency response. In first instance, SDOF system with damping ratio 5% will be analyzed.

1. Multi periodic harmonic excitation:

The two frames would be analyzed here for multi periodic sine wave:

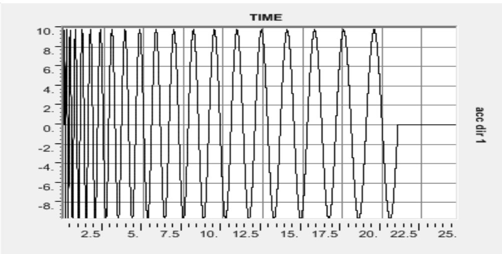
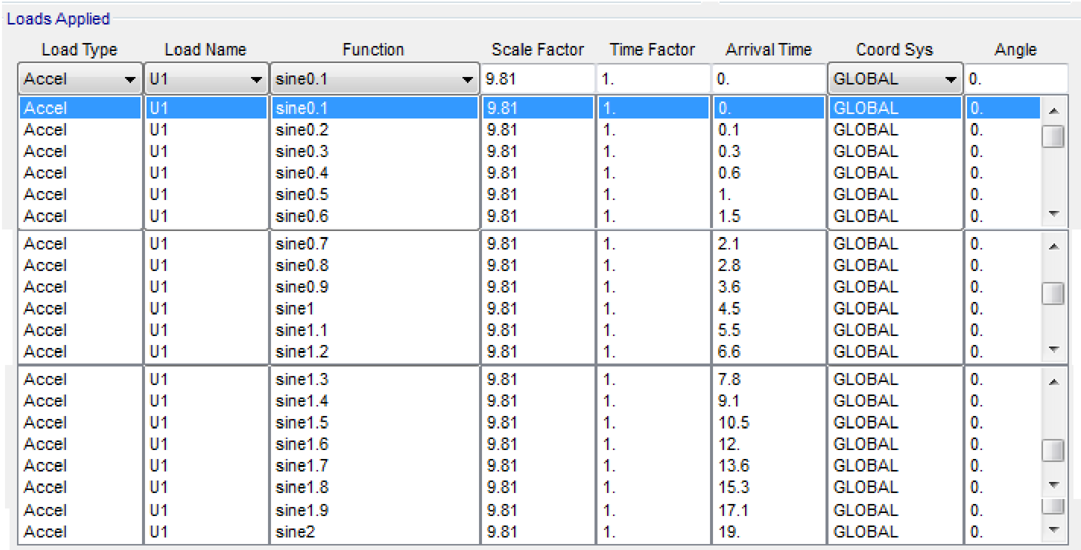


Figure (6. 2): multi periodic sine excitation.



Loads Applied

Load Type	Load Name	Function	Scale Factor	Time Factor	Arrival Time	Coord Sys	Angle
Accel	U1	sine0.1	9.81	1.	0.	GLOBAL	0.
Accel	U1	sine0.1	9.81	1.	0.	GLOBAL	0.
Accel	U1	sine0.2	9.81	1.	0.1	GLOBAL	0.
Accel	U1	sine0.3	9.81	1.	0.3	GLOBAL	0.
Accel	U1	sine0.4	9.81	1.	0.6	GLOBAL	0.
Accel	U1	sine0.5	9.81	1.	1.	GLOBAL	0.
Accel	U1	sine0.6	9.81	1.	1.5	GLOBAL	0.
Accel	U1	sine0.7	9.81	1.	2.1	GLOBAL	0.
Accel	U1	sine0.8	9.81	1.	2.8	GLOBAL	0.
Accel	U1	sine0.9	9.81	1.	3.6	GLOBAL	0.
Accel	U1	sine1	9.81	1.	4.5	GLOBAL	0.
Accel	U1	sine1.1	9.81	1.	5.5	GLOBAL	0.
Accel	U1	sine1.2	9.81	1.	6.6	GLOBAL	0.
Accel	U1	sine1.3	9.81	1.	7.8	GLOBAL	0.
Accel	U1	sine1.4	9.81	1.	9.1	GLOBAL	0.
Accel	U1	sine1.5	9.81	1.	10.5	GLOBAL	0.
Accel	U1	sine1.6	9.81	1.	12.	GLOBAL	0.
Accel	U1	sine1.7	9.81	1.	13.6	GLOBAL	0.
Accel	U1	sine1.8	9.81	1.	15.3	GLOBAL	0.
Accel	U1	sine1.9	9.81	1.	17.1	GLOBAL	0.
Accel	U1	sine2	9.81	1.	19.	GLOBAL	0.

Figure (6. 3): Excitation acceleration definition.

The results of top displacement, velocity and acceleration for unrestricted and restricted models are summarized in the table below:

Table (6. 2) SDOF response results.

Model		Free			Restricted		
ωn	**Tn**	**PSD (m)**	**PSV (m/sec)**	**PSA (m/sec2)**	**PSD (m)**	**PSV (m/sec)**	**PSA (m/sec2)**
43.66	0.14	0.01336	0.583298	25.46677	0.006405	0.2796423	12.20918
37.3	0.168	0.02273	0.847829	31.62402	0.01299	0.484527	18.07286
31.31	0.2	0.0378	1.183518	37.05595	0.0207	0.648117	20.29254
20.88	0.3	0.1065	2.22372	46.43127	0.05252	1.0966176	22.89738
15.69	0.4	0.2116	3.320004	52.09086	0.1192	1.870248	29.34419
12.54	0.5	0.3703	4.643562	58.23027	0.2159	2.707386	33.95062
10.466	0.6	0.5559	5.818049	60.89171	0.3003	3.1429398	32.89401
8.97	0.7	0.788	7.06836	63.40319	0.4844	4.345068	38.97526
7.85	0.8	1.071	8.40735	65.9977	0.6653	5.222605	40.99745
6.979	0.9	1.406	9.812474	68.48126	0.8128	5.6725312	39.5886
6.22	1	1.809	11.25198	69.98732	1.076	6.69272	41.62872
5.71	1.1	2.209	12.61339	72.02246	1.348	7.69708	43.95033
5.23	1.2	2.682	14.02686	73.36048	1.66	8.6818	45.40581
4.83	1.3	3.212	15.51396	74.93243	2.102	10.15266	49.03735
4.487	1.4	3.762	16.88009	75.74098	2.404	10.786748	48.40014
4.187	1.5	4.379	18.33487	76.76811	2.665	11.158355	46.72003
3.9249	1.6	5.062	19.86784	77.9793	3.189	12.516506	49.12603
3.69449	1.7	5.792	21.39849	79.05649	3.724	13.758281	50.82983

3.48999	1.8	5.555	19.38689	67.66007	4.115	14.361309	50.12082
3.3055	1.9	5.004	16.54072	54.67536	4.903	16.206867	53.5718
3.138442	2	4.954	15.54784	48.796	5.426	17.029186	53.44511
2.991789	2.1	4.366	13.06215	39.07919	5.769	17.259629	51.63716
2.854861	2.2	3.854	11.00263	31.41098	6.593	18.822097	53.73446
2.731196	2.3	3.488	9.526412	26.0185	7.61	20.784402	56.76628
2.617087	2.4	3.195	8.361591	21.88301	8.022	20.994268	54.94382
2.513091	2.5	2.975	7.476447	18.78899	9.268	23.291331	58.53324

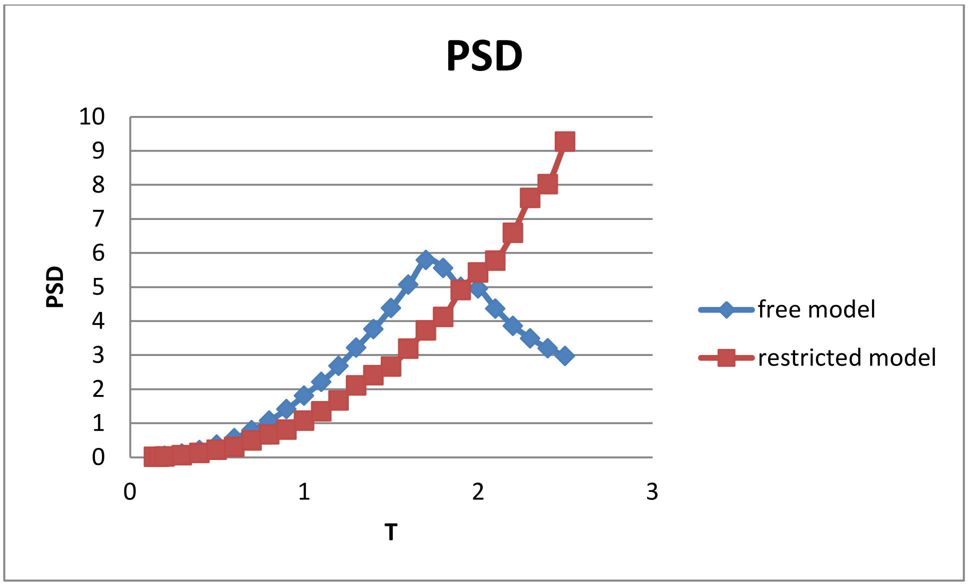


Figure (6. 4): deformation response spectrum.

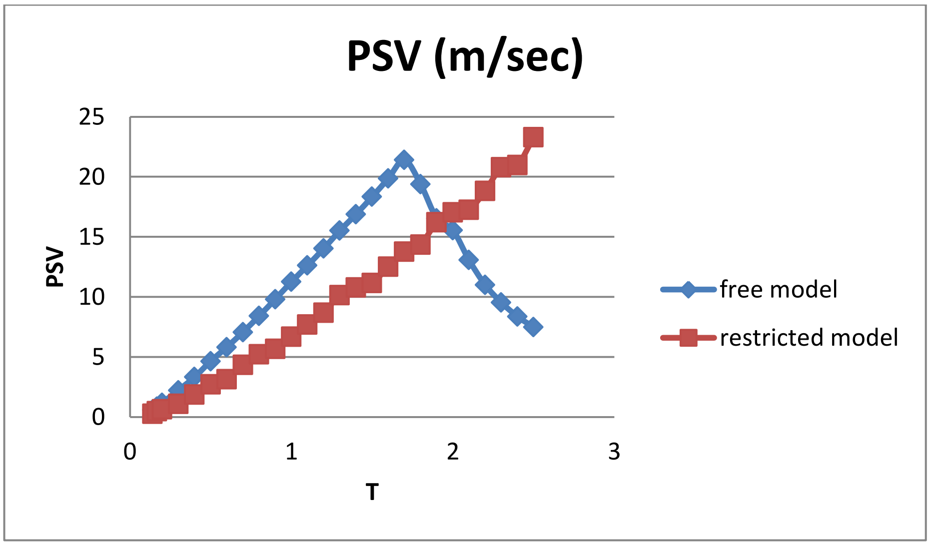


Figure (6. 5): pseudo-velocity response spectrum.

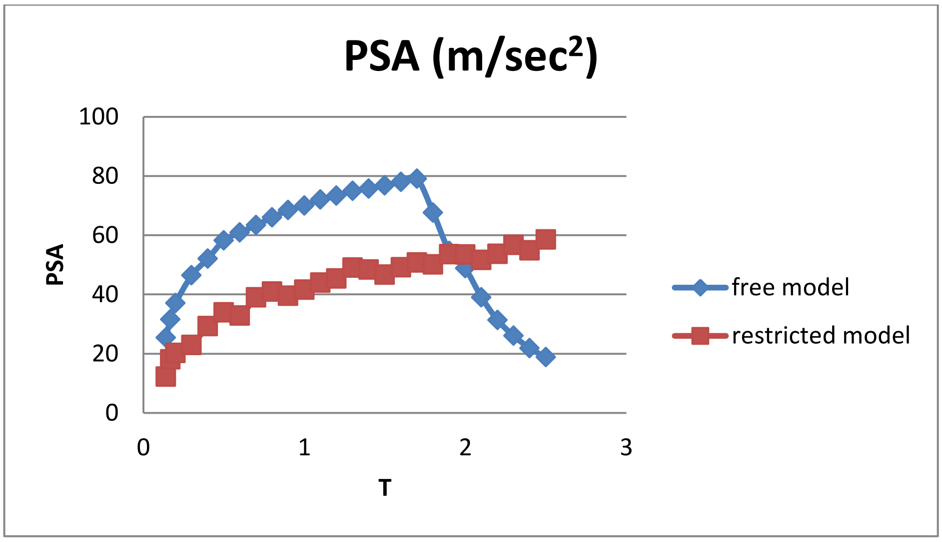


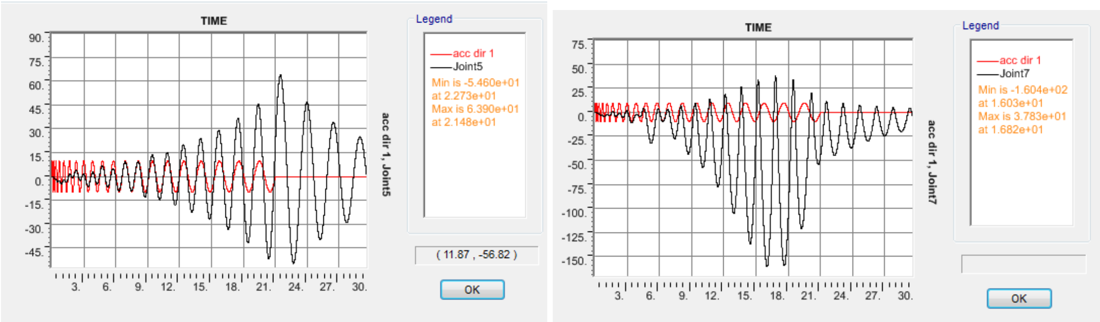
Figure (6. 6): pseudo-acceleration response spectrum.

This behavior introduces the value of considering link stiffness carefully in time history analysis, because of two main reasons will be shown in the next sections:

1. The real earthquake time history has a lag of some periods which could resonate the free structure, or it may contain a period could resonate linked structure and has no effect to the free one.

2. For monotonic harmonic excitation with increasing or decreasing periods, the free structure exactly resonate at the same excitation period equals its natural period that means for long periods out of the time history domain the free structure has no peak, where when considering the link stiffness in the model the structure still resonates in this domain.

The figures below illustrate the time of maximum response for both free and restricted models under same excitation:



(a) free structure. (b)restricted structure.

Figure (6. 7): time of maximum response of structure.

This figure shows that for the same model inputs with same excitation shown in red plot, the link decreased the period of structure, and has its maximum response at lower period.

2. Real Earthquake Excitation

Same models will be analyzed here for Elcentro earthquake:

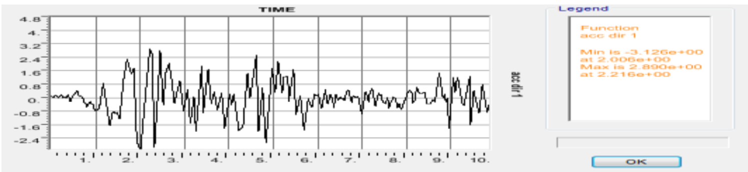


Figure (6. 8): Elcentro earthquake excitation

Load Type	Load Name	Function	Scale Factor	Time Factor	Arrival Time	Coord Sys	Angle
Accel	U1	elcentro	9.81	1.	0.	GLOBAL	0.
Accel	U1	elcentro	9.81	1.	0.	GLOBAL	0.

Figure (6. 9): excitation definition.

Table (6. 3): analysis results for joint at beam level.

model		Free			Restricted		
ωn	Tn	SD (m)	PSV(m/sec)	PSA(m/sec^2)	SD (m)	PSV(m/sec)	PSA(m/sec^2)
43.66	0.14	0.001982	0.086534	3.77808	0.001493	0.065184	2.84595
37.3	0.168	0.003863	0.14409	5.374553	0.002559	0.095451	3.560311
31.31	0.2	0.008596	0.269141	8.426797	0.003739	0.117068	3.665402
20.88	0.3	0.01623	0.338882	7.075865	0.01625	0.3393	7.084584
15.69	0.4	0.03628	0.569233	8.931269	0.02868	0.449989	7.060331
12.54	0.5	0.06785	0.850839	10.66952	0.03301	0.413945	5.190875
10.466	0.6	0.07306	0.764646	8.002785	0.05299	0.554593	5.804374
8.97	0.7	0.06686	0.599734	5.379616	0.07942	0.712397	6.390205
7.85	0.8	0.08244	0.647154	5.080159	0.1011	0.793635	6.230035
6.979	0.9	0.1075	0.750243	5.235942	0.1234	0.861209	6.010375
6.22	1	0.1216	0.756352	4.704509	0.1027	0.638794	3.973299
5.71	1.1	0.1049	0.598979	3.42017	0.1139	0.650369	3.713607
5.23	1.2	0.1027	0.537121	2.809143	0.1035	0.541305	2.831025
4.83	1.3	0.1057	0.510531	2.465865	0.1775	0.857325	4.14088
4.487	1.4	0.1516	0.680229	3.052188	0.1825	0.818878	3.674303
4.187	1.5	0.1302	0.545147	2.282532	0.212	0.887644	3.716565
3.9249	1.6	0.1384	0.543206	2.13203	0.1817	0.713154	2.799059
3.69449	1.7	0.2064	0.762543	2.817207	0.1611	0.595182	2.198895
3.48999	1.8	0.2489	0.868659	3.03161	0.1628	0.56817	1.982909
3.3055	1.9	0.2709	0.89546	2.959943	0.2184	0.721921	2.386311
3.138442	2	0.2532	0.794654	2.493974	0.1638	0.514077	1.6134
2.991789	2.1	0.2414	0.722218	2.160723	0.2023	0.605239	1.810747
2.854861	2.2	0.2859	0.816205	2.330151	0.3298	0.941533	2.687946
2.731196	2.3	0.3357	0.916863	2.504131	0.2923	0.798329	2.180392
2.617087	2.4	0.3728	0.97565	2.55336	0.2529	0.661861	1.732148
2.513091	2.5	0.3939	0.989907	2.487726	0.2317	0.582283	1.463331

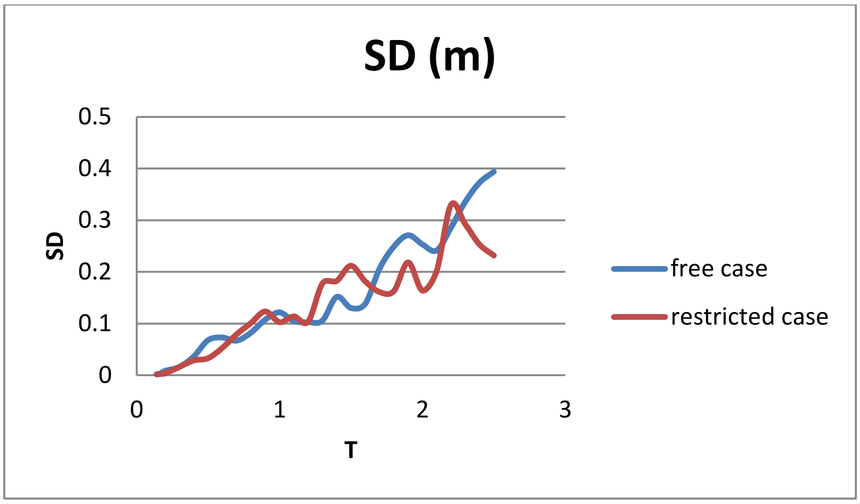


Figure (6. 10): deformation response spectrum.

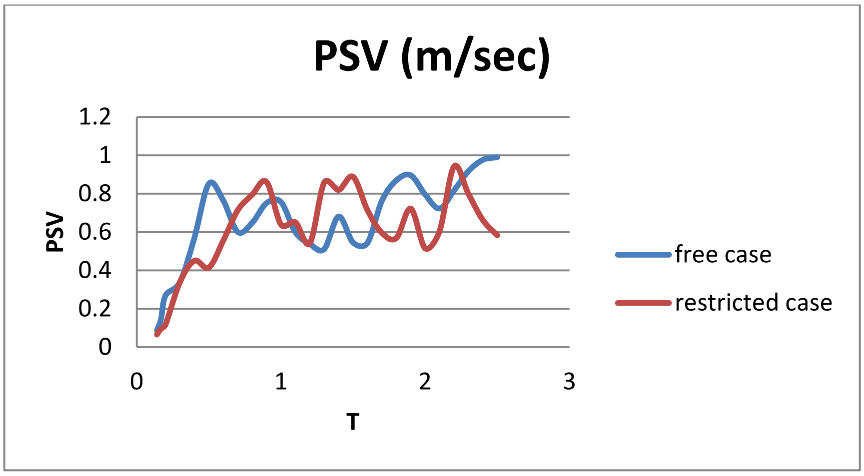


Figure (6. 11): pseudo-velocity response spectrum.

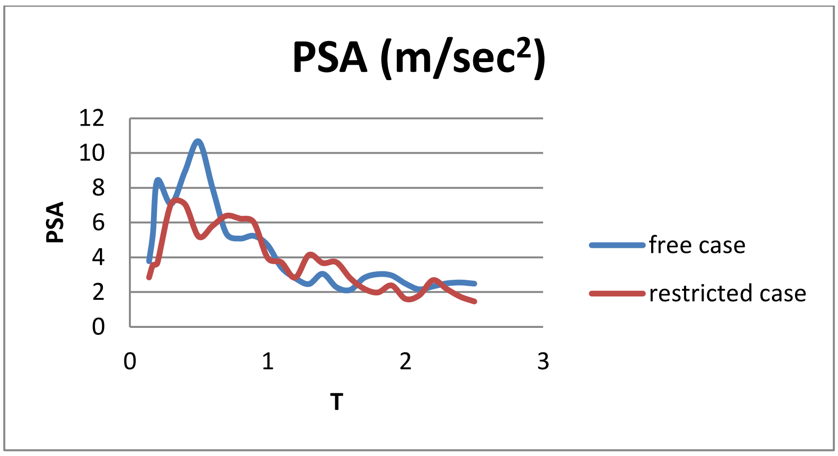


Figure (6. 12): pseudo-acceleration response spectrum.

This response spectrum will depend on the damping ratio and the ground motion selected.

In order to perform the seismic analysis and design of a structure to be built at a particular location, the actual time history record is required.

However, it is not possible to have such records at each and every location.

Further, the seismic analysis of structures cannot be carried out simply based on the peak value of the ground acceleration as the response of the structure depend upon the frequency content of ground motion and its own dynamic properties.

Response spectrum is an important tool in the seismic analysis and design of structures. It describes the maximum response of damped single degree of freedom system to a particular input motion at different natural periods.

Response spectrum method of analysis is advantageous as it considers the frequency effects and provides a single suitable horizontal force for the design of structure.

From these figures, one can directly read the maximum relative displacement of any structure of natural period T for a particular value of ξ as damping

It’s shown above that for certain natural periods the linked structure may have a larger response for a certain ground motion.

6.3.2 Frequency-Response Analysis

For the purpose of this analysis a frame of free natural period =1.0 sec and damping ratio 5% compared to the same models of frame connected to link with (link stiffness / structure stiffness) =15,as shown in table (6.5) below.

Table (6. 4): models parameters.

Tn	**1.0**		**1.0**
damping	0.05		0.05
ω	6.283499		6.283499
Klink	0		500000
Tn2	1.0		0.218207
tnavg	1.0		0.609079

Table (6. 5): analysis results for top displacements

		Kl	**0**	**500000**
Tn	**ω**	**T/Tn**	**kl/k=0**	**kl/k=15**
1	6.283499	0.1	0.02516	0.02856
1	6.283499	0.15	0.04053	0.1075
1	6.283499	0.2	0.05116	0.2288
1	6.283499	0.3	0.09938	0.4676
1	6.283499	0.4	0.1507	0.2872
1	6.283499	0.5	0.1997	0.3887
1	6.283499	0.6	0.3205	1.904
1	6.283499	0.65	0.3366	1.353
1	6.283499	0.7	0.4606	0.7733
1	6.283499	0.8	0.7252	0.5329
1	6.283499	0.85	0.9306	0.4928
1	6.283499	0.9	1.245	0.4187
1	6.283499	1	2.362	0.4191
1	6.283499	1.1	1.501	0.3754
1	6.283499	1.2	1.047	0.4719
1	6.283499	1.3	0.8191	0.5128
1	6.283499	1.4	0.7053	0.4366
1	6.283499	1.5	0.6125	0.418
1	6.283499	1.6	0.5704	0.4068
1	6.283499	1.7	0.5247	0.4069
1	6.283499	1.8	0.477	0.46
1	6.283499	1.9	0.429	0.4035
1	6.283499	2	0.4005	0.4005
1	6.283499	2.1	0.3968	0.3968
1	6.283499	2.2	0.3926	0.4028
1	6.283499	2.3	0.388	0.388
1	6.283499	2.4	0.3831	0.3831
1	6.283499	2.5	0.3794	0.378

Fig (6.13) below shows the frequency response curves for unrestricted and restricted frames with stiffness of link =15 times the stiffness of structure.

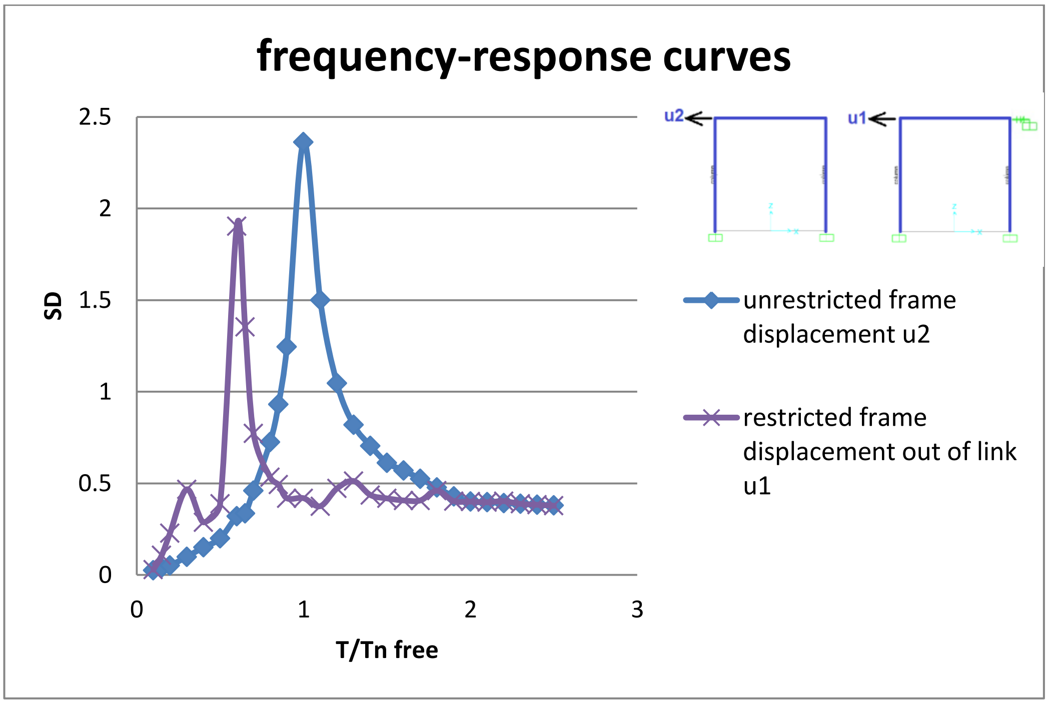


Figure (6. 13): frequency-response curves for restricted and unrestricted frames.

This graph shows that unrestricted structure has its maximum displacement exactly at excitation period equals the natural period "T/T_n =1" which matches resonance theory perfectly.

For restricted models, the structure resonates at lower period between free natural period and combined link-structure period, and it is exactly at the average of both.

The effect of link stiffness will be clarified in the parametric study chapter for different stiffness values.

6.4 Two Degrees of Freedom Frames (2DOF)

There are two possible configurations depending on attached link's level, the two configurations will be analyzed for response spectrum and frequency response approaches.

6.4.1 The Link At 1st Storey

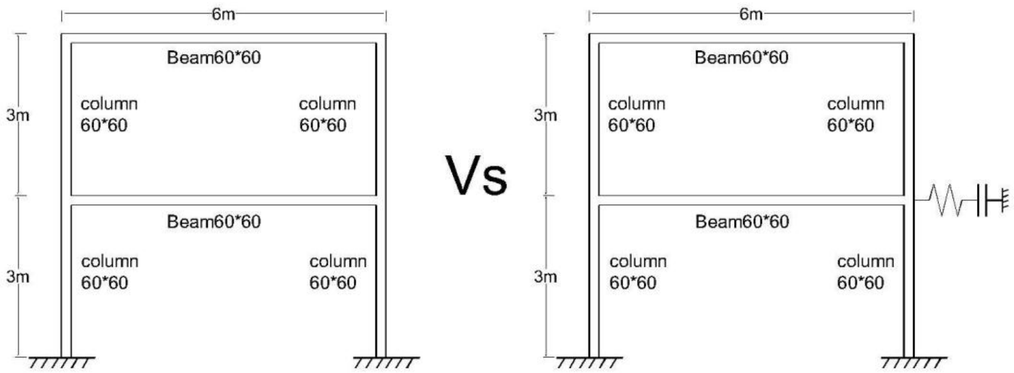


Figure (6. 14): 2DOF models unrestricted vs. restricted with link at 1st storey.

6.4.1.1 Response Spectrum Analysis

The two frames would be analyzed here for multi periodic sine wave has periods from 0.1sec to 2sec with same magnitude:

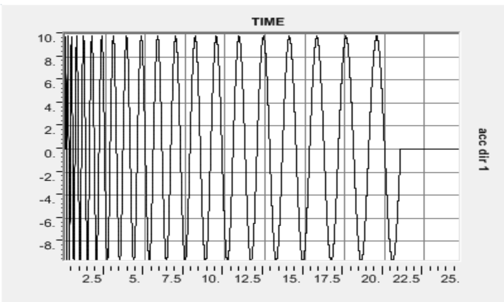


Figure (6. 15): multi periodic sine excitation.

The results of 20 models with free natural periods from 0.1sec to 1sec summarized in the table below.

Table (6. 6) displacement results for 1st and 2nd stories of two frames.

Wd	**15**	**140**	**300**	**500**	**800**	**1200**	**1600**	**2070**	**2600**	**3200**
Tn	**0.09618**	**0.21955**	**0.31316**	**0.4005**	**0.50389**	**0.61528**	**0.70939**	**0.80605**	**0.90272**	**1.00095**
out of link										
L U1	0.004855	0.02966	0.06988	0.1278	0.2123	0.3316	0.4642	0.6180	0.8046	1.002
L U2	0.006964	0.04582	0.1075	0.1909	0.3182	0.4845	0.6700	0.9086	1.200	1.500
F U3	0.005971	0.03587	0.08205	0.1440	0.2513	0.3897	0.5400	0.7111	0.9157	1.143
F U4	0.009594	0.05972	0.1355	0.2394	0.4173	0.6508	0.9008	1.189	1.531	1.914
Δ1	0.002109	0.01616	0.03762	0.0631	0.1059	0.1529	0.2058	0.2906	0.3954	0.4980
Δ2	0.003623	0.02385	0.05345	0.0954	0.166	0.2611	0.3608	0.4779	0.6153	0.7710
in link side										
L U5	0.001531	0.01085	0.002374	0.04033	0.06681	0.1078	0.1492	0.2000	0.2567	0.3135
L U6	0.005509	0.03744	0.08693	0.1527	0.2625	0.4229	0.5770	0.7637	0.9905	1.241
Δ3	0.003978	0.02659	0.08456	0.11237	0.19569	0.3151	0.4278	0.5637	0.7338	0.9275

Fig (6.16) below shows the displacements results of the unrestricted model due to multi period harmonic excitation described above, where u3 and u4 represents the displacement of 1st and 2nd stories respectively.

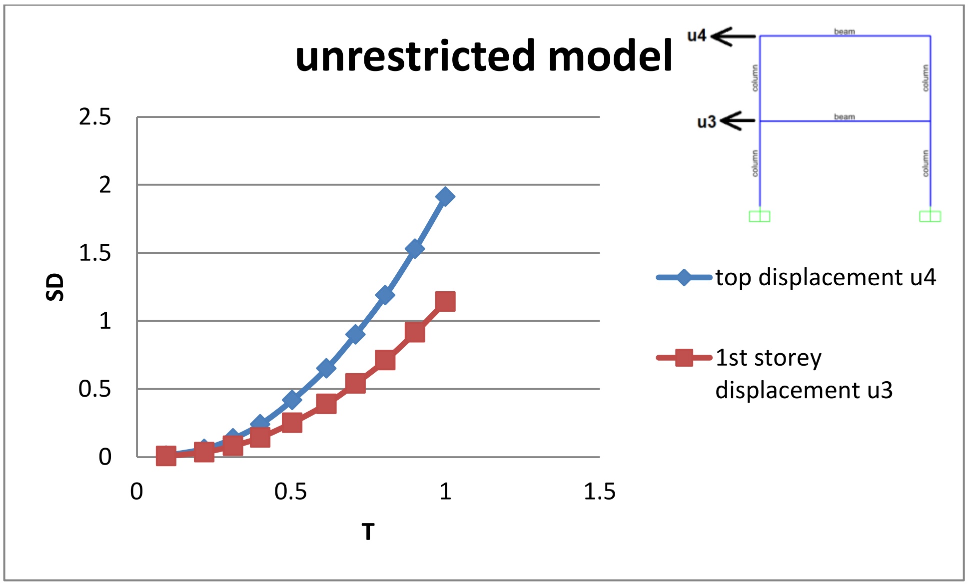


Figure (6. 16): displacement results of unrestricted model.

Fig (6.17) below shows the displacements results of the restricted model due to multi period harmonic excitation described above, where u1 and u2 represents the displacement out of link of 1st and 2nd stories respectively, u5 and u6 represents the displacement in link side of 1st and 2nd stories respectively. Where fig (6.18) shows shear comparison for top stories.

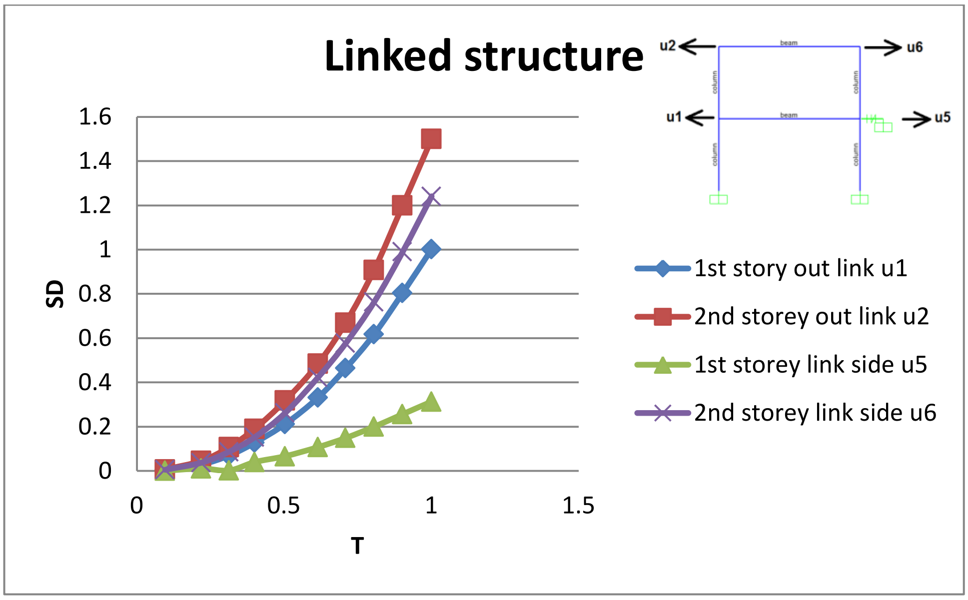


Figure (6. 17): displacement results of unrestricted model.

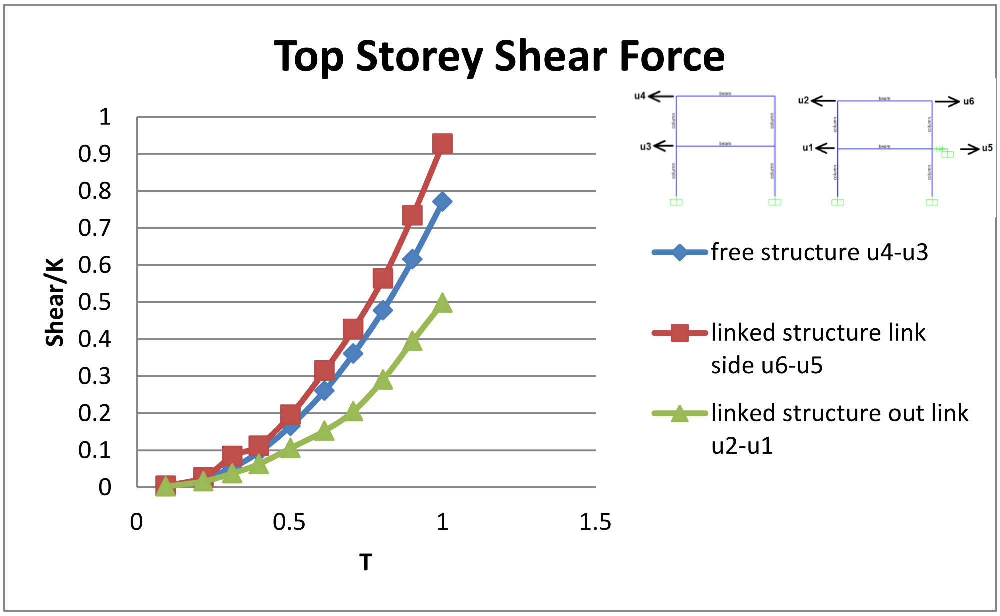


Figure (6. 18): shear comparison of unrestricted and restricted models.

Current seismic design criteria in the United States are based on story shear strength patterns developed from well-established dynamic analysis concepts (IBC, 2009)

These shear strength patterns represent the expected distribution of the maximum inertia forces that a system experiences when it is subjected to seismic excitations. The shape of the code-compliant shear strength distributions takes into account the most important dynamic characteristics that influence the behavior of multi-story buildings (e.g., higher mode effects)

Frame structures subjected to strong ground shaking are generally designed with sufficient deformation capacity to undergo significant levels of inelastic behavior. However, the inelastic dynamic behavior of structures is not very well understood, and the designer has limited control over the extent of damage that a system will experience and its distribution in the structure. Results from this study suggest that in some cases, designing frames using story shear strength patterns based on unrestricted vibration may not be the conservative to mitigate the occurrence and/or the extent of damage in frames that experience considerable levels of inelastic deformation. The problem becomes more complex when issues such as the P-Delta effects, structure over strength, cyclic deterioration and the contribution of nonstructural components to the response are present.

6.4.1.2 Frequency Response Analysis

For the purpose of this analysis a frame of free natural period =0.8 sec and damping ratio 5% will compared to the same models of frame connected to link at first storey with stiffness equal 500000 N/mm.

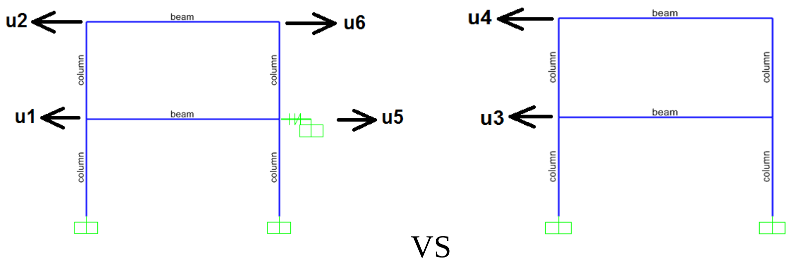


Figure (6. 19): degrees of freedom to be measured.

The table below summarizes the results of displacement for the two frames in two directions and the relative displacement between stories.

Table (6. 7): displacement (m) results for the two frames.

T_n	T	T/T_n	u1	u2	u3	u4	u5	u6	Δ1	Δ2	Δ3
0.8	0.1	**0.125**	0.0145	0.0249	0.0144	0.02493	0.00751	0.0163	0.0104	0.0105	0.00877
0.8	0.2	**0.25**	0.0596	0.0684	0.0382	0.05472	0.02867	0.0498	0.0088	0.0165	0.02109
0.8	0.3	**0.375**	0.0728	0.1316	0.0832	0.1052	0.04992	0.0732	0.0588	0.022	0.02323
0.8	0.4	**0.5**	0.089	0.154	0.0887	0.1542	0.03457	0.1407	0.065	0.0655	0.10613
0.8	0.5	**0.625**	0.182	0.285	0.1104	0.2081	0.06966	0.2384	0.103	0.0977	0.16874
0.8	0.6	**0.75**	0.3802	0.5523	0.2468	0.4388	0.1097	0.4821	0.1721	0.192	0.3724
0.8	0.7	**0.875**	1.028	1.526	0.4524	0.7811	0.3001	1.323	0.498	0.3287	1.0229
0.8	0.8	**1**	0.5258	0.7655	1.094	1.85	0.1783	0.6773	0.2397	0.756	0.499
0.8	0.9	**1.125**	0.3425	0.5369	0.6288	1.048	0.1291	0.4675	0.1944	0.4192	0.3384
0.8	1	**1.25**	0.2735	0.4276	0.4182	0.6898	0.0938	0.3704	0.1541	0.2716	0.2766
0.8	1.1	**1.375**	0.1959	0.303	0.3433	0.5628	0.08785	0.3488	0.1071	0.2195	0.26095
0.8	1.2	**1.5**	0.1877	0.3061	0.2753	0.4484	0.08151	0.3216	0.1184	0.1731	0.24009
0.8	1.3	**1.625**	0.1894	0.3071	0.2153	0.3477	0.08027	0.3064	0.1177	0.1324	0.22613
0.8	1.4	**1.75**	0.234	0.3781	0.1928	0.3099	0.08829	0.3013	0.1441	0.1171	0.21301
0.8	1.5	**1.875**	0.217	0.3502	0.1891	0.3045	0.06052	0.229	0.1332	0.1154	0.16848
0.8	1.6	**2**	0.1918	0.3058	0.1878	0.3017	0.05294	0.1986	0.114	0.1139	0.14566

The following figures will illustrate the behavior of each frame and compare the shear forces for the second storey.

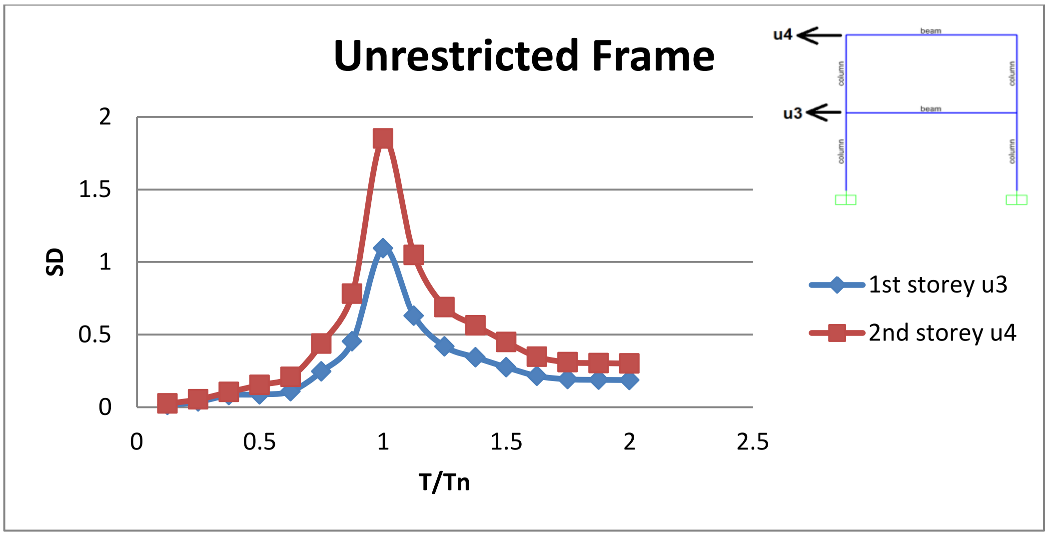


Figure (6. 20): displacement results for unrestricted frame.

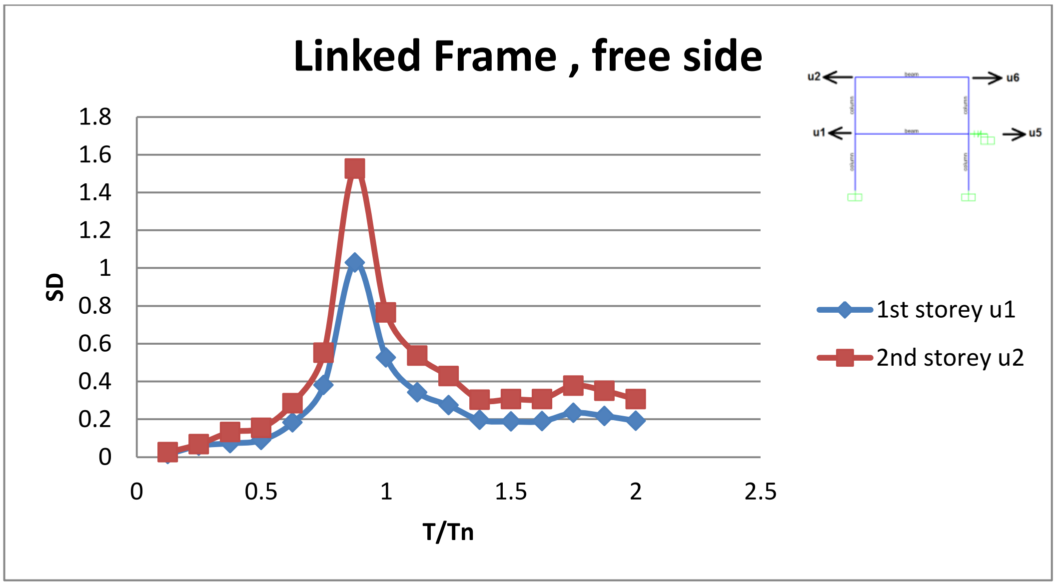


Figure (6. 21): displacement results for restricted frame in the opposite side of link.

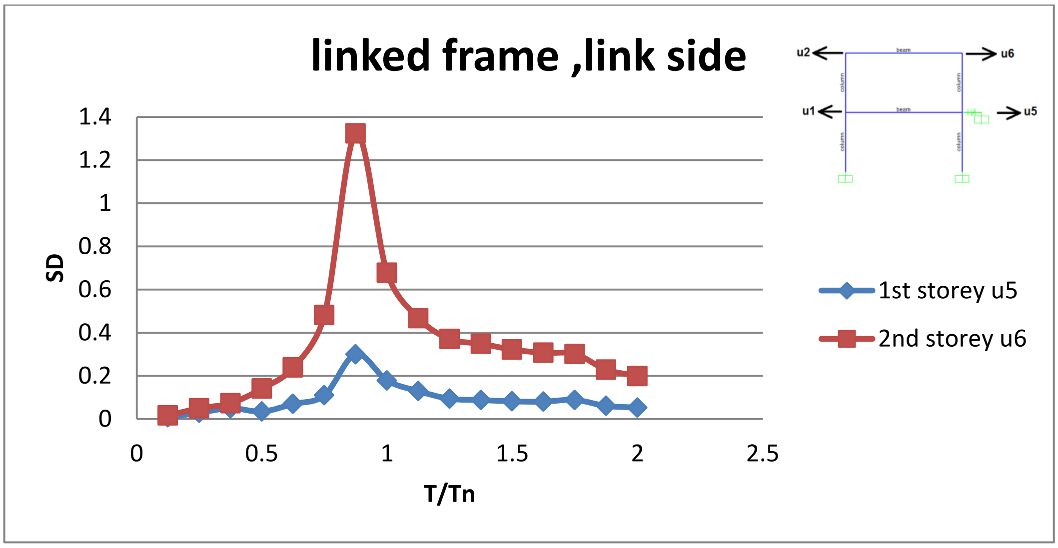


Figure (6. 22): displacement results for restricted frame in the link's side.

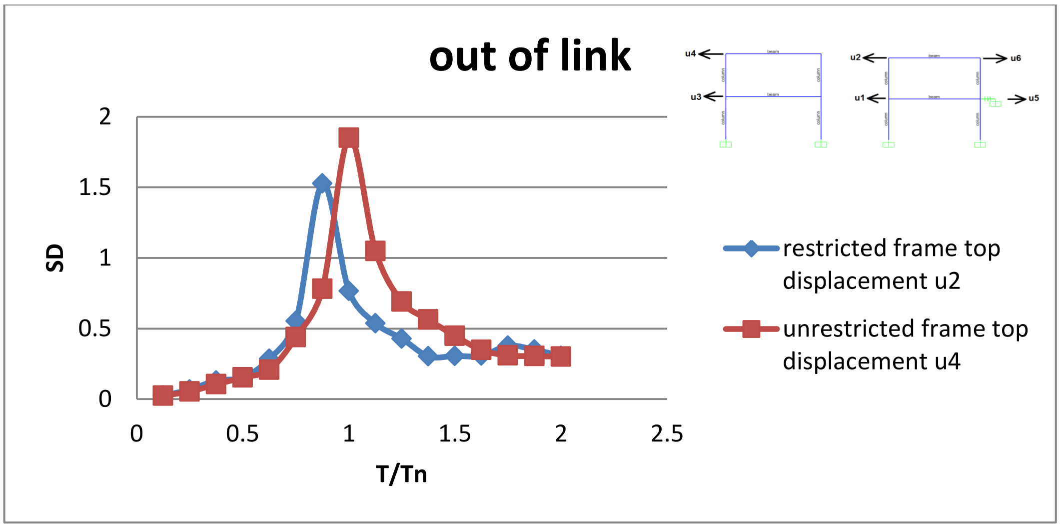


Figure (6. 23): top displacement results for two frames in the opposite side of link.

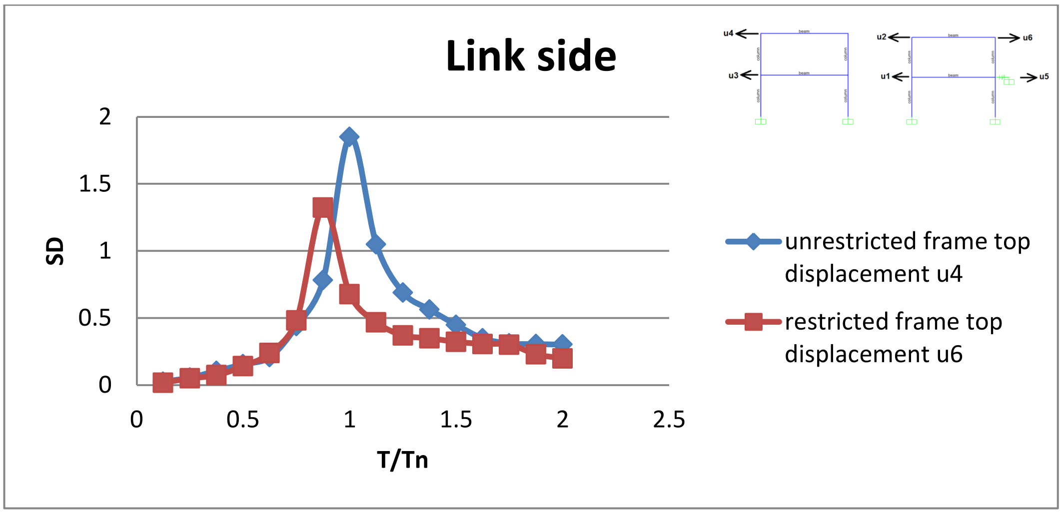


Figure (6. 24): top displacement results for two frames in the link's side.

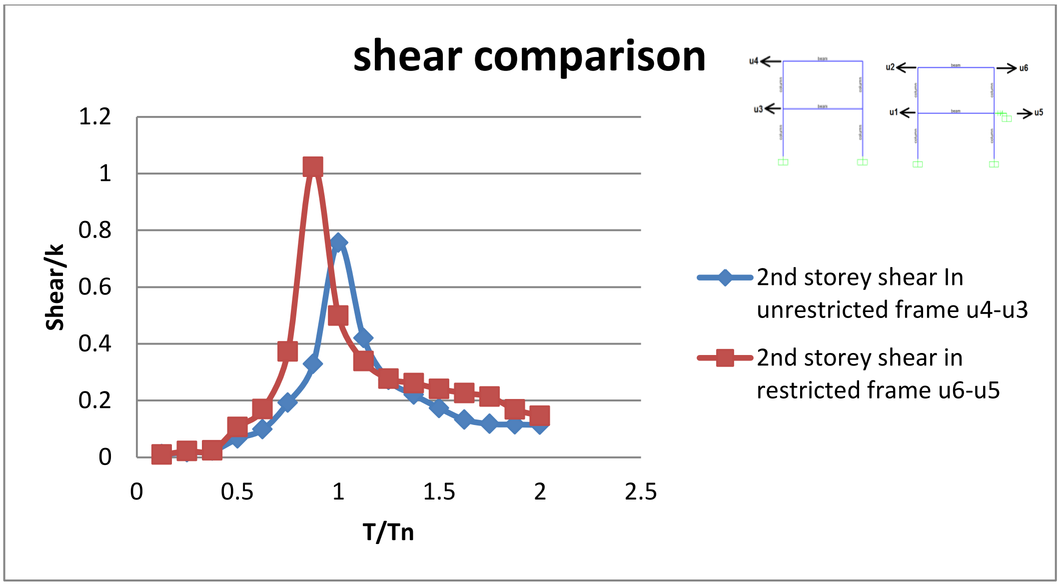


Figure (6. 25): shear forces in the 2nd storey for the two frames.

6.4.2 The Link At 2nd Storey

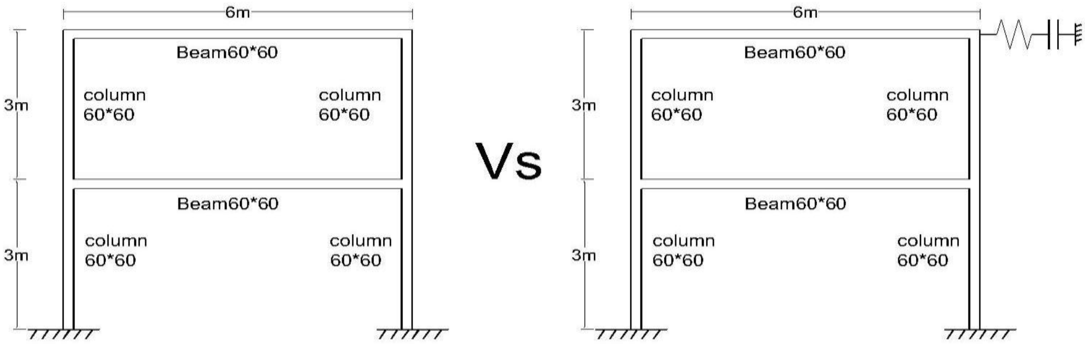


Figure (6. 26): 2DOF models unrestricted vs. restricted with link at 2nd storey.

6.4.2.1 Response Spectrum Analysis

The two frames would be analyzed here for multi periodic sine wave has periods from 0.1sec to 2sec with same magnitude:

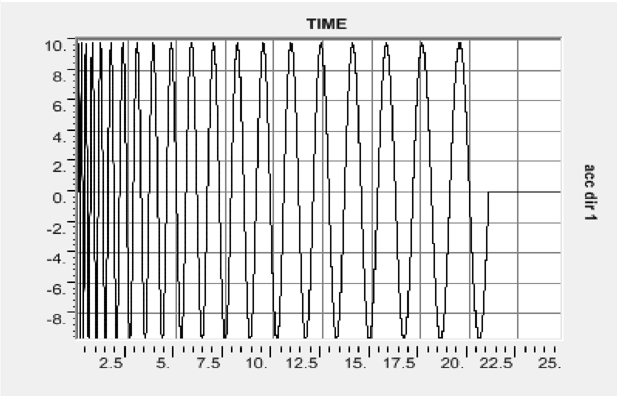


Figure (6. 27): multi periodic sine excitation.

The displacement results of 20 models with free natural periods from 0.1sec to 1sec summarized in the following table (6.8) for degrees of freedom in the following figure (6.29).

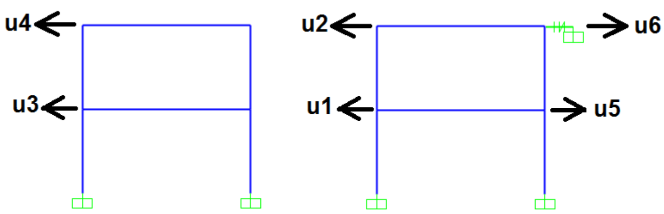


Figure (6. 28): degrees of freedom to be measured.

Table (6. 8): analysis results for the two frames.

Wd	15	140	300	500	800	1200	1600	2070	2600	3200
Tn	0.0962	0.21955	0.31316	0.4005	0.50389	0.6153	0.7094	0.8061	0.9027	1.001
out of link										
u1	0.003	0.02376	0.04381	0.0941	0.1576	0.2476	0.3414	0.4165	0.5716	0.7423
u2	0.0049	0.03973	0.07476	0.1552	0.2528	0.3981	0.5369	0.668	0.8766	1.132
u3	0.006	0.03245	0.08205	0.144	0.2513	0.3897	0.54	0.7111	0.9157	1.143
u4	0.0096	0.0539	0.1355	0.2394	0.4173	0.6508	0.9008	1.189	1.531	1.914
Δ1	0.0019	0.01597	0.03095	0.0611	0.0952	0.1505	0.1955	0.2515	0.305	0.3897
Δ2	0.0036	0.02145	0.05345	0.0954	0.166	0.2611	0.3608	0.4779	0.6153	0.771
in link side										
u5	0.0029	0.01591	0.04026	0.0833	0.1363	0.2154	0.2894	0.3572	0.4681	0.615
u6	0.0022	0.01416	0.02848	0.0527	0.08179	0.1353	0.1998	0.2509	0.3035	0.4114
Δ3	-7E-04	-0.0018	-0.0118	-0.0306	-0.0545	-0.0801	-0.0896	-0.106	-0.165	-0.204

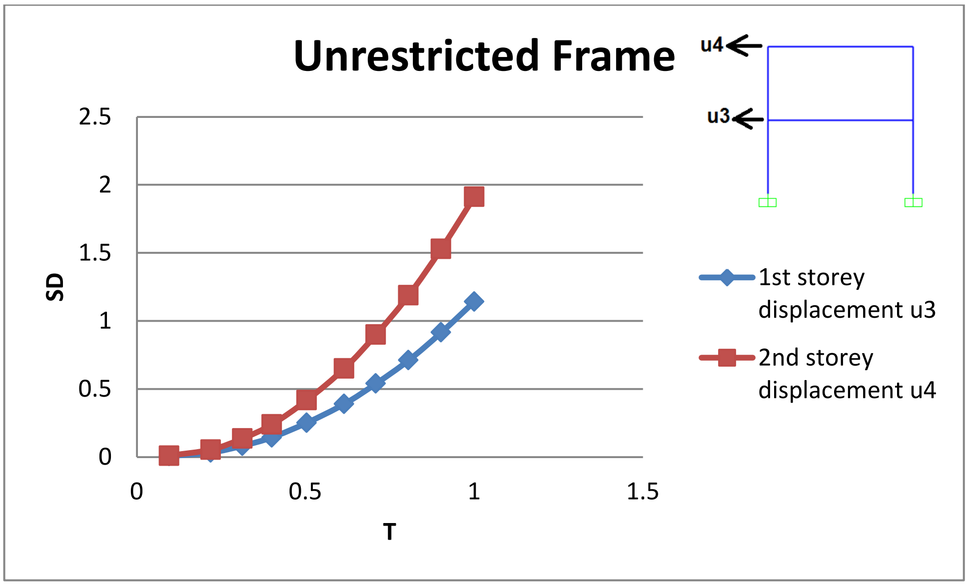


Figure (6. 29): displacement (m) results for unrestricted frame.

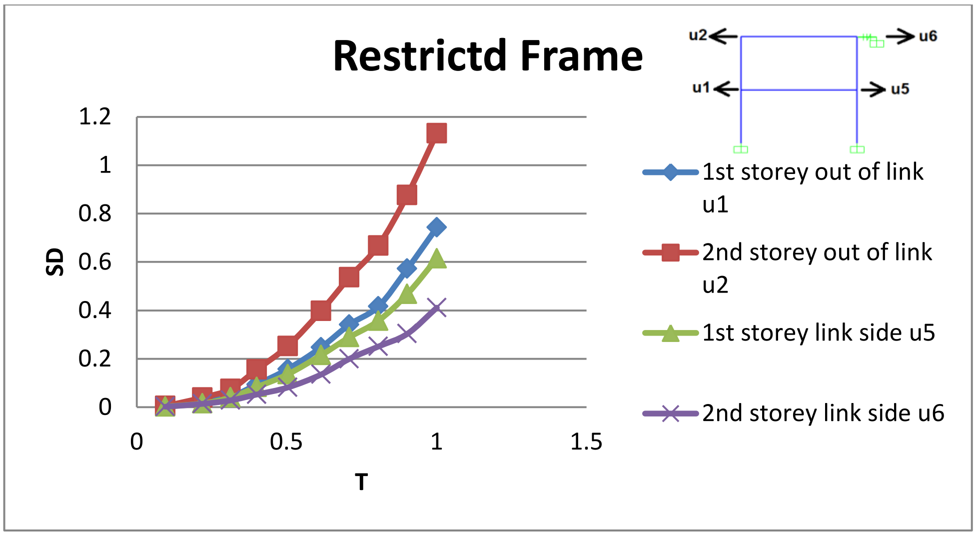


Figure (6. 30): displacement (m) results for restricted frame linked at 2nd storey.

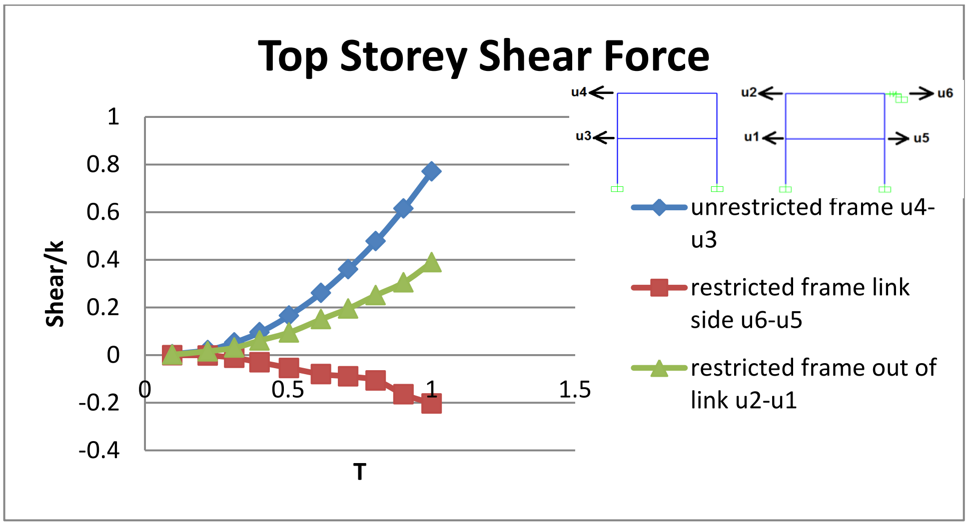


Figure (6. 31): top storey shear forces in the two frames.

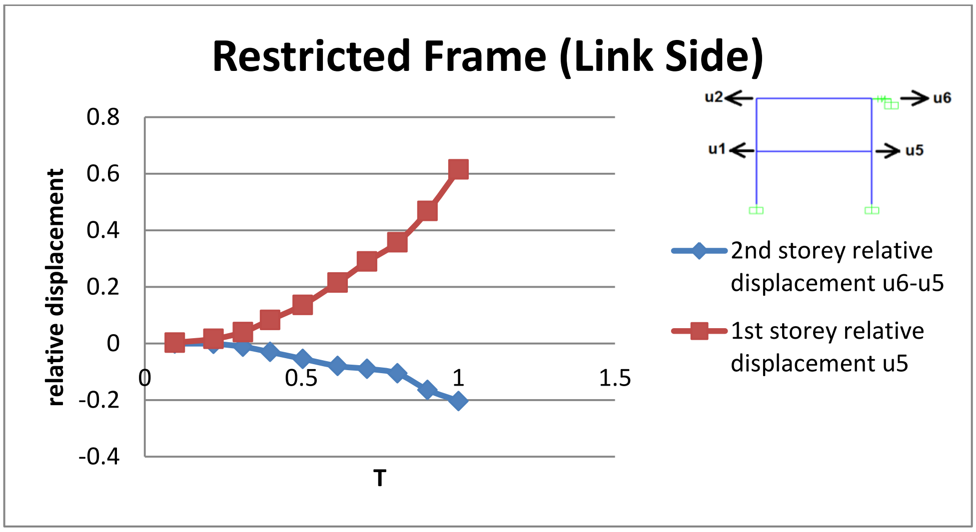


Figure (6. 32): relative displacements for restricted frame.

6.4.2.2 Frequency Response Analysis

For the purpose of this analysis a frame of free natural period =0.8 sec and damping ratio 5% will compared to the same models of frame connected to link at 2nd storey with stiffness equal 500000 N/mm.

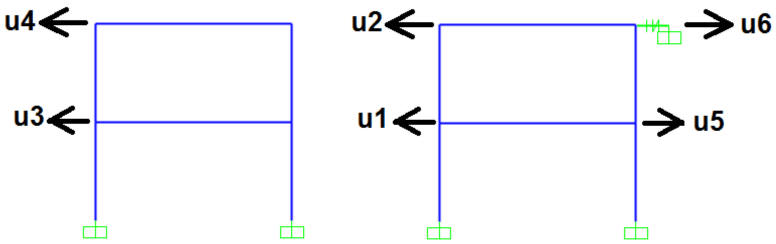


Figure (6. 33): degrees of freedom to be measured.

The table below summarizes the results of displacement for the two frames in two directions and the relative displacement between stories.

Table (6. 9): displacement (m) results for the two frames.

T_n	T	T/T_n	u1	u2	u3	u4	u5	u6	∆1	∆2	∆3
0.8	0.1	**0.125**	0.0144	0.02485	0.01442	0.02492	0.0112	0.007833	0.01044	0.0105	-0.00337
0.8	0.2	**0.25**	0.0382	0.05459	0.03819	0.05472	0.02331	0.01792	0.01643	0.01653	-0.00539
0.8	0.3	**0.375**	0.147	0.19	0.08316	0.1052	0.1293	0.05907	0.043	0.02204	-0.07023
0.8	0.4	**0.5**	0.1421	0.208	0.08873	0.1542	0.05104	0.07847	0.0659	0.06547	0.02743
0.8	0.5	**0.625**	0.3584	0.4459	0.1104	0.2081	0.1339	0.2019	0.0875	0.0977	0.068
0.8	0.6	**0.75**	0.4601	0.7034	0.2468	0.4389	0.3743	0.2416	0.2433	0.1921	-0.1327
0.8	0.7	**0.875**	0.3311	0.63	0.4524	0.7811	0.3261	0.2101	0.2989	0.3287	-0.116
0.8	0.8	**1**	0.2413	0.4351	1.095	1.853	0.2112	0.1682	0.1938	0.758	-0.043
0.8	0.9	**1.125**	0.2232	0.3142	0.6288	1.048	0.2076	0.1516	0.091	0.4192	-0.056
0.8	1	**1.25**	0.1788	0.2969	0.4182	0.6899	0.1835	0.1451	0.1181	0.2717	-0.0384
0.8	1.1	**1.375**	0.2106	0.3021	0.3433	0.5628	0.1718	0.1526	0.0915	0.2195	-0.0192
0.8	1.2	**1.5**	0.2512	0.3957	0.2753	0.4484	0.1592	0.1325	0.1445	0.1731	-0.0267
0.8	1.3	**1.625**	0.2213	0.3359	0.2152	0.3477	0.1432	0.1138	0.1146	0.1325	-0.0294
0.8	1.4	**1.75**	0.1896	0.3064	0.1928	0.3099	0.128	0.1022	0.1168	0.1171	-0.0258
0.8	1.5	**1.875**	0.1891	0.3045	0.1891	0.3045	0.1169	0.09077	0.1154	0.1154	-0.02613
0.8	1.6	**2**	0.1877	0.3017	0.1878	0.3017	0.1148	0.08274	0.114	0.1139	-0.03206

The following figures will illustrate the behavior of each frame and compare the relative displacement for restricted frame.

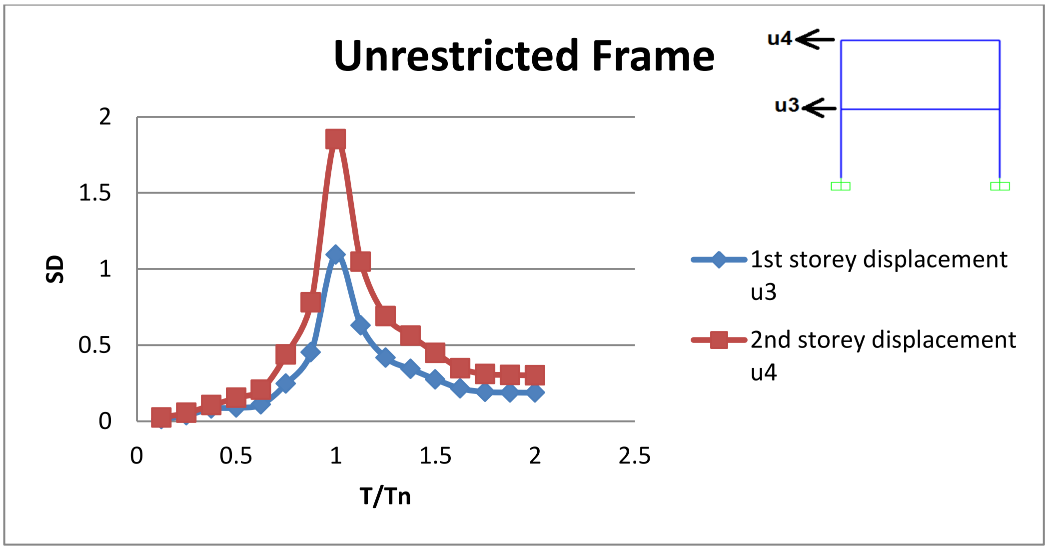


Figure (6. 34): displacement (m) results for unrestricted frame.

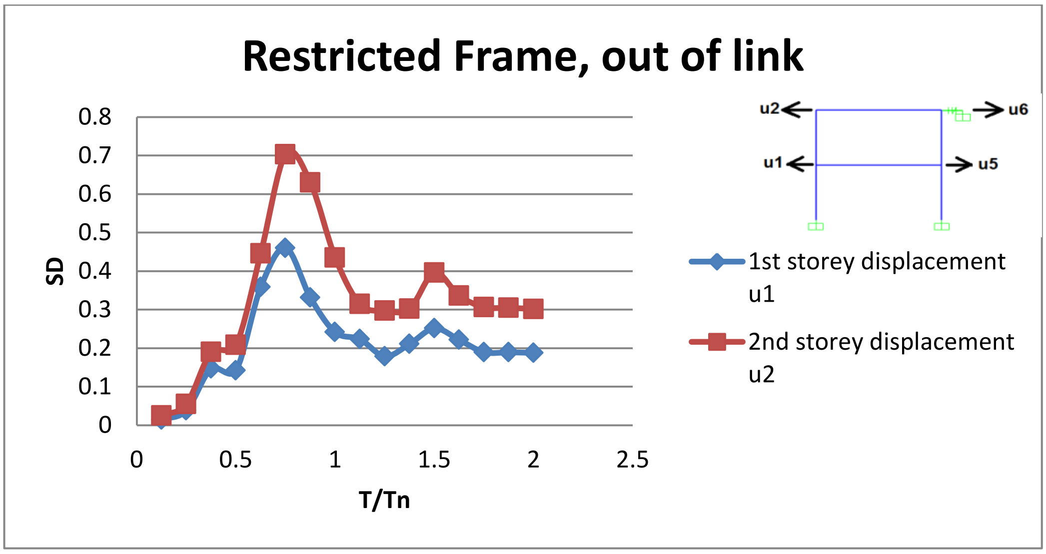


Figure (6. 35): displacement (m) results out of link for restricted frame linked at 2nd storey.

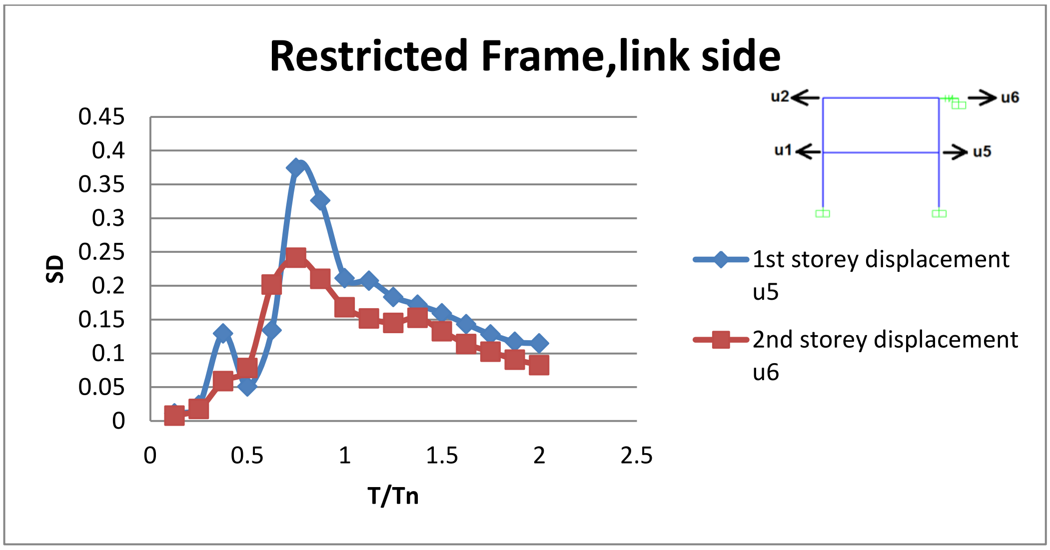


Figure (6. 36): displacement (m) results in link side for restricted frame linked at 2nd storey.

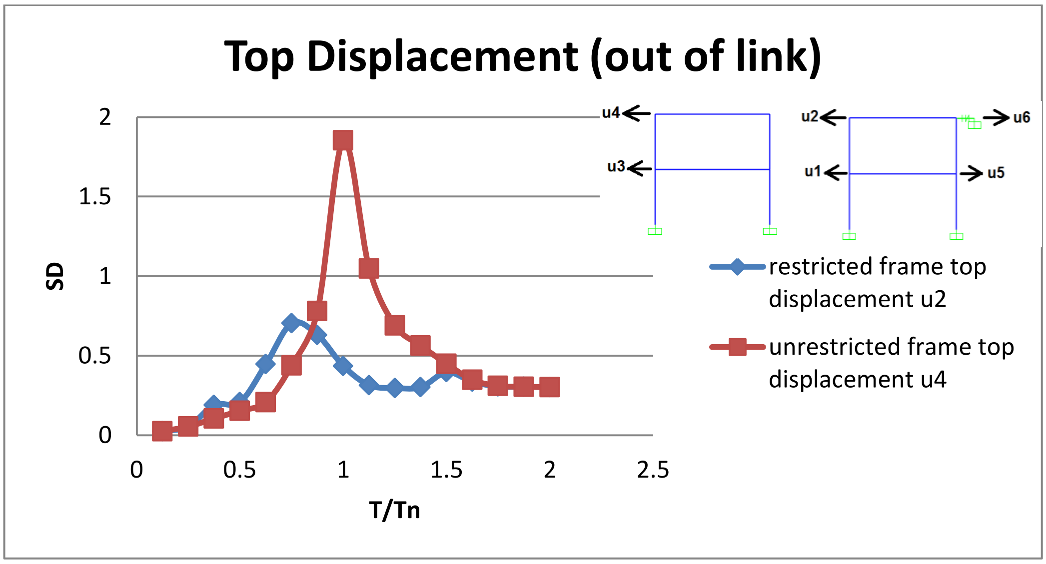


Figure (6. 37): top displacement (m) results out of link for two frames.

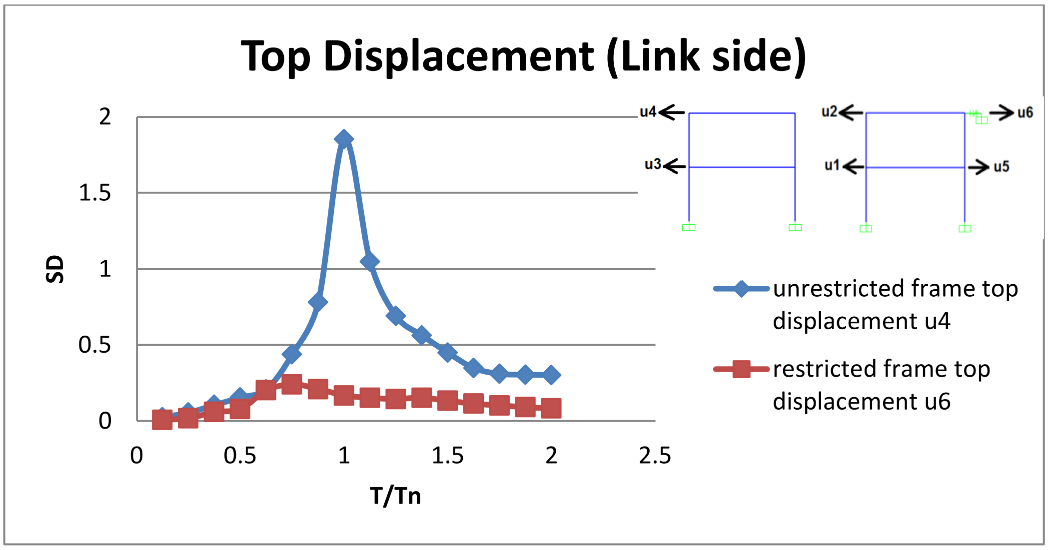


Figure (6. 38): top displacement (m) results in link side for two frames.

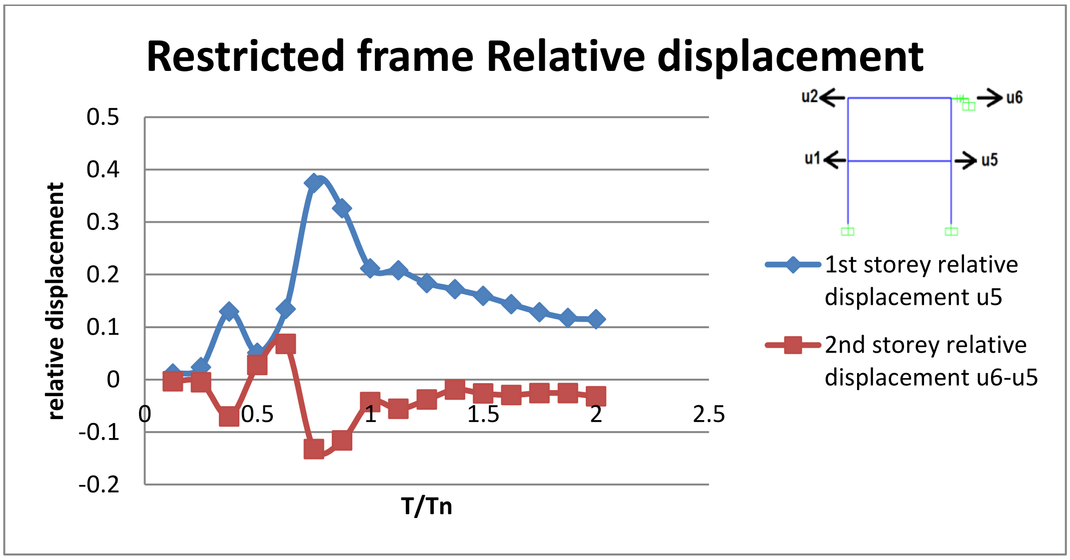


Figure (6. 39): relative displacements (m) in link side for restricted frame.

6.5 Three Degrees Of Freedom Frames (3DOF)

There are three possible configurations depending on attached link's level, the three configurations will be analyzed for response spectrum and frequency response approaches.

6.5.1 The Link At 1st Storey

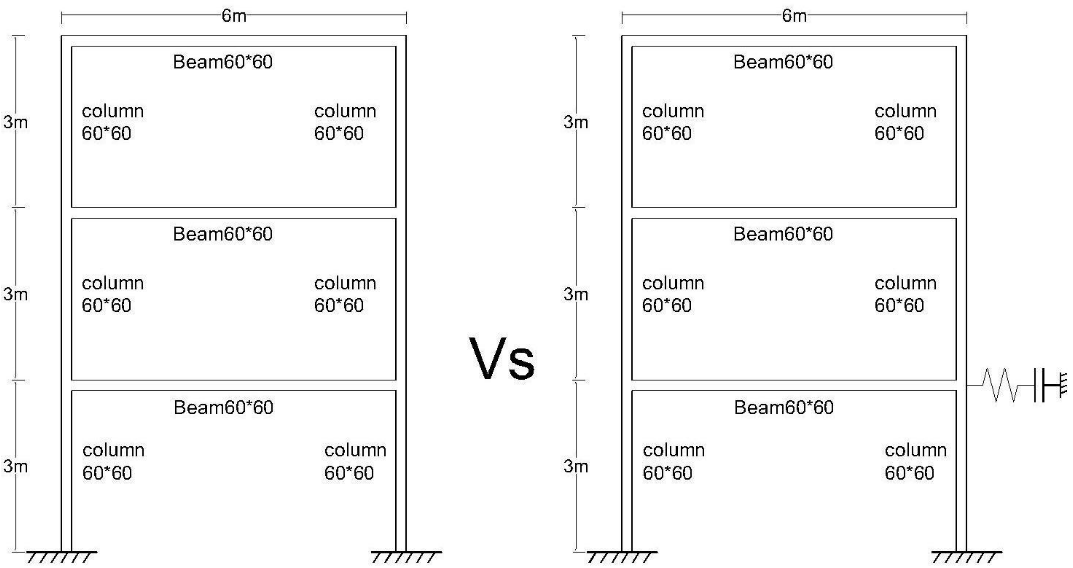


Figure (6. 40): 3DOF models, unrestricted vs. restricted with link at 1st storey.

6.5.1.1 Response Spectrum Analysis

The two frames would be analyzed here for multi periodic sine wave has periods from 0.1sec to 2sec with same magnitude:

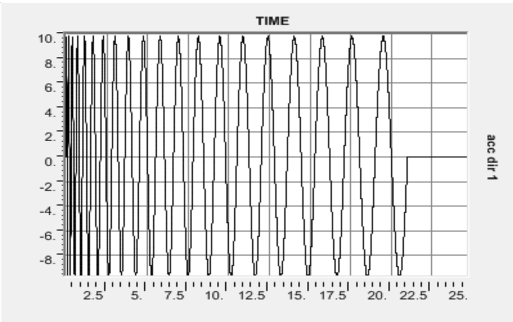


Figure (6. 41): multi periodic sine excitation.

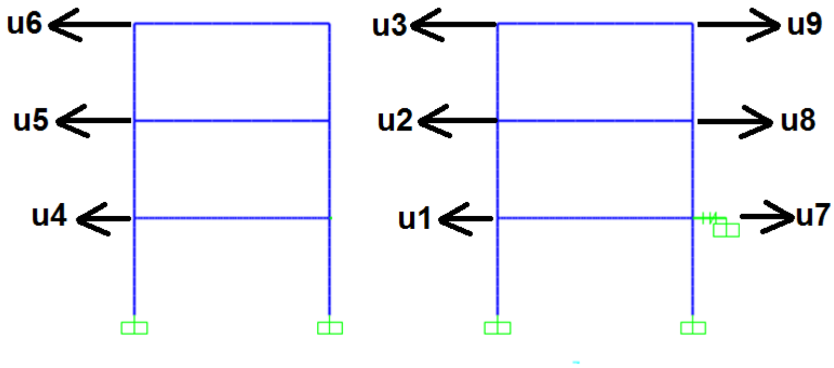


Figure (6. 42): degrees of freedom to be measured.

The table below summarizes the results of displacement for the two frames in two directions and the relative displacement between stories.

Table (6. 10): displacement (m) results for the two frames.

Wd	15	50	130	245	380	560	760	1000	1280	1600
T_n	**0.13862**	**0.20319**	**0.3029**	**0.4053**	**0.5001**	**0.6025**	**0.7002**	**0.8003**	**0.904**	**1.0095**
out of link										
u1	0.00839	0.0204	0.04974	0.0963	0.1539	0.2357	0.3295	0.4476	0.5868	0.7513
u2	0.01491	0.03765	0.09141	0.1774	0.2828	0.4351	0.6097	0.8296	1.087	1.39
u3	0.01788	0.04477	0.1088	0.2094	0.3335	0.512	0.7185	0.9771	1.28	1.635
u4	0.00829	0.02116	0.05185	0.1091	0.1779	0.2721	0.3811	0.51	0.6637	0.8317
u5	0.01534	0.03915	0.09657	0.2036	0.3324	0.5101	0.7148	0.9585	1.249	1.568
u6	0.01943	0.04967	0.1236	0.2607	0.4262	0.6555	0.9186	1.234	1.608	2.021
Δ1L	0.00839	0.0204	0.04974	0.0963	0.1539	0.2357	0.3295	0.4476	0.5868	0.7513
Δ2L	0.00652	0.01725	0.04167	0.0811	0.1289	0.1994	0.2802	0.382	0.5002	0.6387
Δ3L	0.00297	0.00712	0.01739	0.032	0.0507	0.0769	0.1088	0.1475	0.193	0.245
Δ1F	0.00829	0.02116	0.05185	0.1091	0.1779	0.2721	0.3811	0.51	0.6637	0.8317
Δ2F	0.00705	0.01799	0.04472	0.0945	0.1545	0.238	0.3337	0.4485	0.5853	0.7363
Δ3F	0.00409	0.01052	0.02703	0.0571	0.0938	0.1454	0.2038	0.2755	0.359	0.453
in link side										
u7	0.00288	0.00554	0.01507	0.032	0.04954	0.0795	0.1103	0.1508	0.1995	0.2574
u8	0.01156	0.02415	0.06393	0.1315	0.2083	0.3247	0.4513	0.6141	0.8085	1.037
u9	0.01755	0.03716	0.09863	0.2031	0.3208	0.5015	0.6982	0.9509	1.252	1.606
Δ1L+	0.00288	0.00554	0.01507	0.032	0.04954	0.0795	0.1103	0.1508	0.1995	0.2574
Δ2L+	0.00868	0.01861	0.04886	0.0995	0.15876	0.2452	0.341	0.4633	0.609	0.7796
Δ3L+	0.00599	0.01301	0.0347	0.0716	0.1125	0.1768	0.2469	0.3368	0.4435	0.569

The following figures will illustrate the behavior of each frame and compare the relative displacement for the two frames.

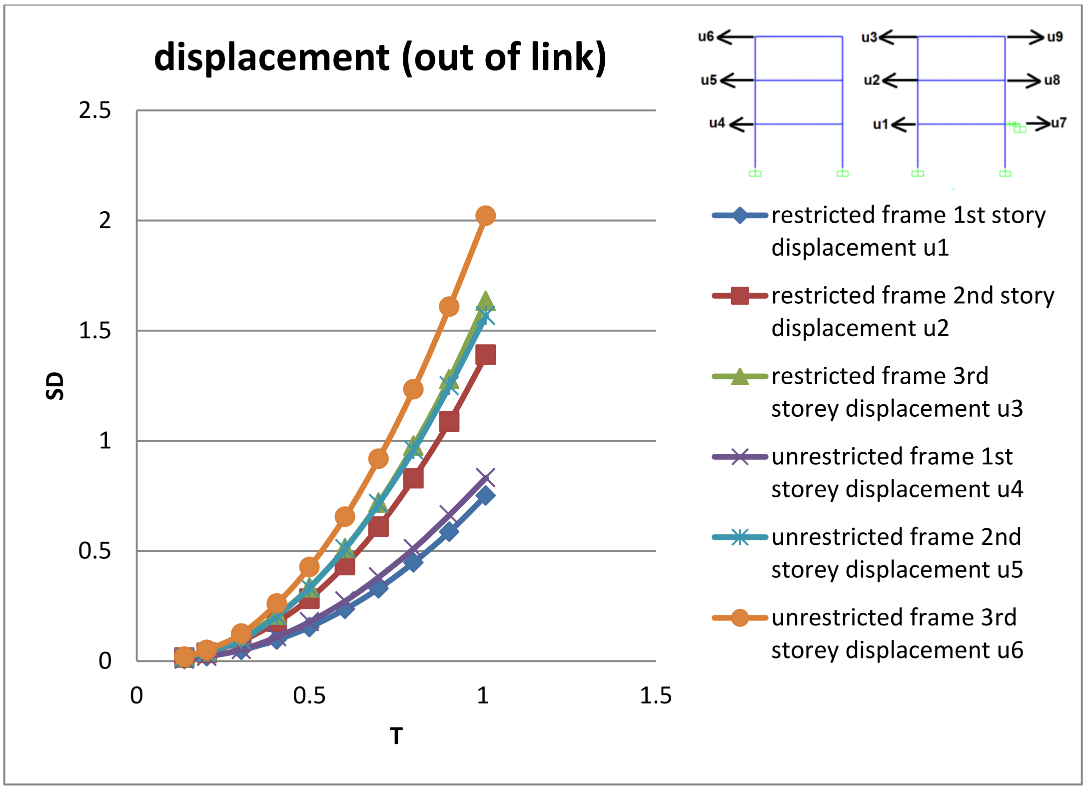


Figure (6. 43): displacement (m) results for the two frames out of link.

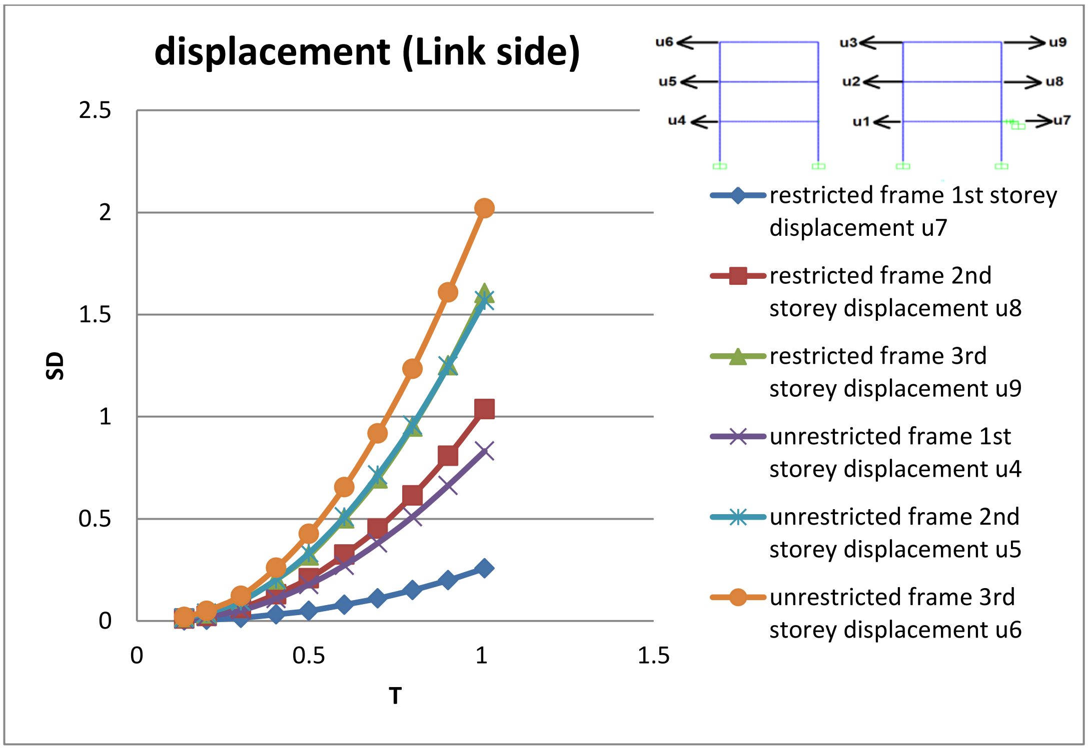


Figure (6. 44): displacement (m) results for the two frames in link side.

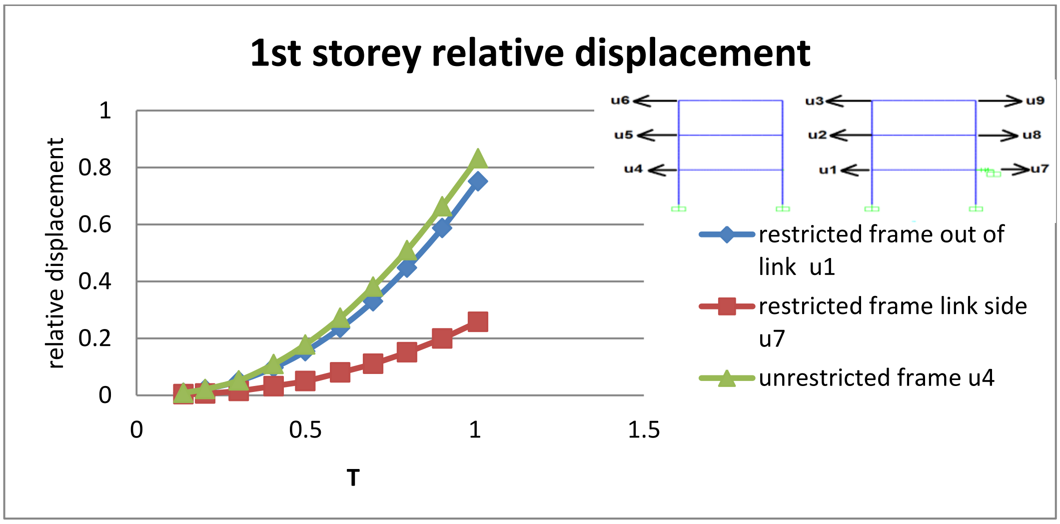


Figure (6. 45): 1st storey relative displacement (m) results for the two frames.

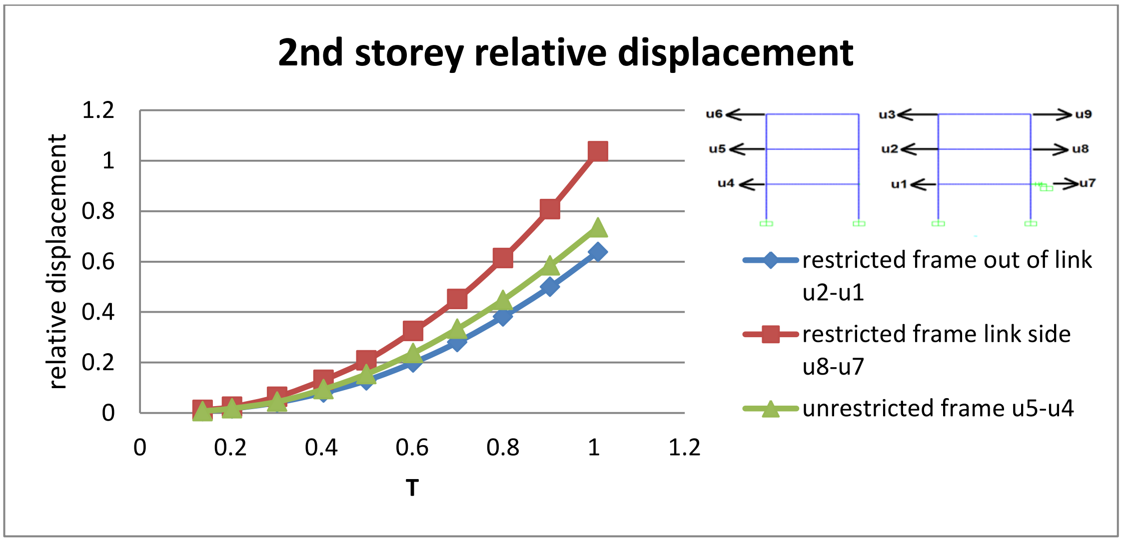


Figure (6. 46): 2nd storey relative displacement (m) results for the two frames.

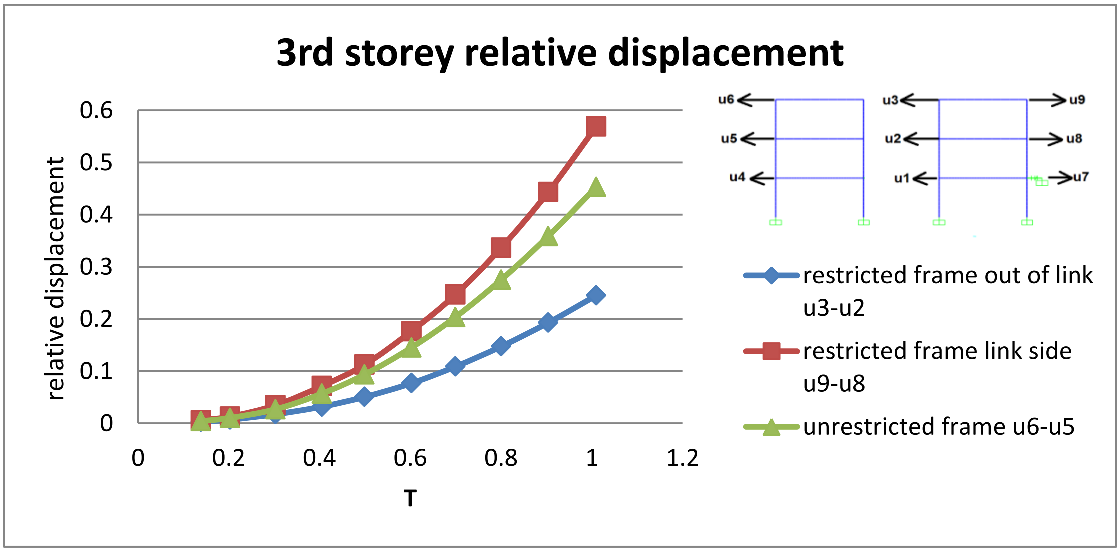


Figure (6. 47): 3rd storey relative displacement (m) results for the two frames.

6.5.1.2 Frequency Response Analysis

For the purpose of this analysis a frame of free natural period =0.8 sec and damping ratio 5% will compared to the same models of frame connected to link at 2nd storey with stiffness equal 500000 N/mm.

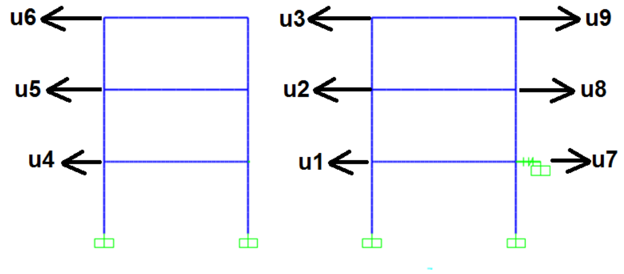


Figure (6. 48): degrees of freedom to be measured.

The table below summarizes the results of displacement for the two frames, and the following figures will illustrate the behavior of each frame.

Table (6. 11): displacement (m) results for the two frames.

Tn	T	T/Tn	u1	u2	u3	u4	u5	u6	u7	u8	u9
0.8	0.1	**0.125**	0.01125	0.01939	0.0266	0.0113	0.0193	0.0266	0.005595	0.01295	0.01804
0.8	0.2	**0.25**	0.02994	0.05293	0.0577	0.02991	0.0486	0.057	0.01458	0.03883	0.05043
0.8	0.3	**0.375**	0.04702	0.08604	0.1114	0.0646	0.086	0.1115	0.02749	0.0541	0.08878
0.8	0.4	**0.5**	0.06226	0.1209	0.1633	0.06203	0.1207	0.1634	0.01483	0.09109	0.1586
0.8	0.5	**0.625**	0.1203	0.2175	0.2893	0.07659	0.163	0.2266	0.03462	0.1687	0.2693
0.8	0.6	**0.75**	0.2406	0.4226	0.5517	0.1767	0.3504	0.4626	0.06993	0.3039	0.5065
0.8	0.7	**0.875**	0.5759	1.073	1.266	0.3319	0.6458	0.8452	0.2089	0.7523	1.194
0.8	0.7	**0.9063**	0.7285	1.344	1.583	0.4065	0.7861	1.026	0.252	0.9562	1.496
0.8	0.7	**0.9125**	0.7366	1.354	1.597	0.4272	0.8248	1.076	0.2522	0.9674	1.509
0.8	0.8	**0.9375**	0.6721	1.221	1.446	0.5128	0.9852	1.282	0.2231	0.8868	1.369
0.8	0.8	**1**	0.4537	0.8264	0.9899	0.7764	1.476	1.91	0.1458	0.6034	0.9352
0.8	0.9	**1.125**	0.276	0.5107	0.6126	0.4381	0.82	1.053	0.09646	0.3864	0.6051
0.8	1	**1.25**	0.2307	0.406	0.4988	0.2917	0.5403	0.6903	0.07407	0.2802	0.4332
0.8	1.1	**1.375**	0.1785	0.3048	0.3697	0.2438	0.4495	0.5729	0.07109	0.2666	0.4081
0.8	1.2	**1.5**	0.1363	0.2491	0.3155	0.1946	0.3566	0.4532	0.06637	0.2468	0.3762
0.8	1.3	**1.625**	0.1376	0.2503	0.3165	0.1567	0.2842	0.3592	0.06061	0.2235	0.3405
0.8	1.4	**1.75**	0.1401	0.2526	0.3209	0.1379	0.25	0.3158	0.06061	0.2226	0.3306
0.8	1.5	**1.875**	0.1555	0.2842	0.3646	0.1374	0.2486	0.3138	0.04795	0.174	0.2678
0.8	1.6	**2**	0.1418	0.262	0.3381	0.1364	0.2464	0.3108	0.04164	0.1499	0.23

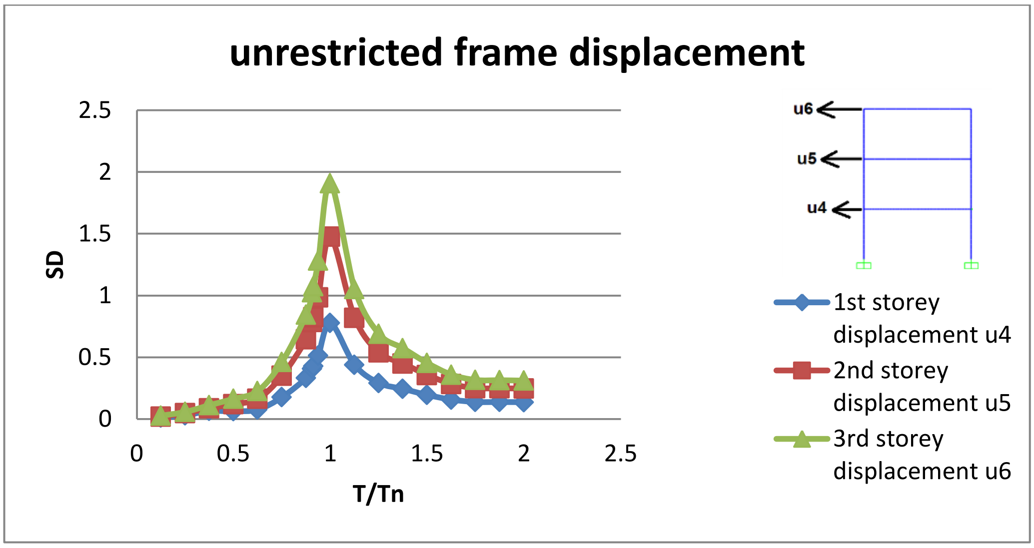


Figure (6. 49): displacement (m) results for unrestricted frame.

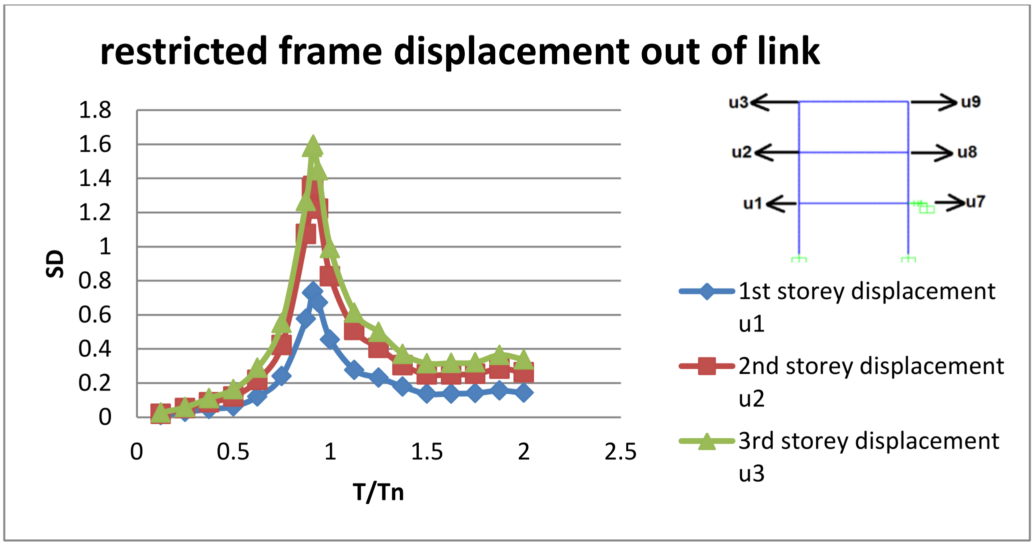


Figure (6. 50): displacement (m) results for restricted frame out of link.

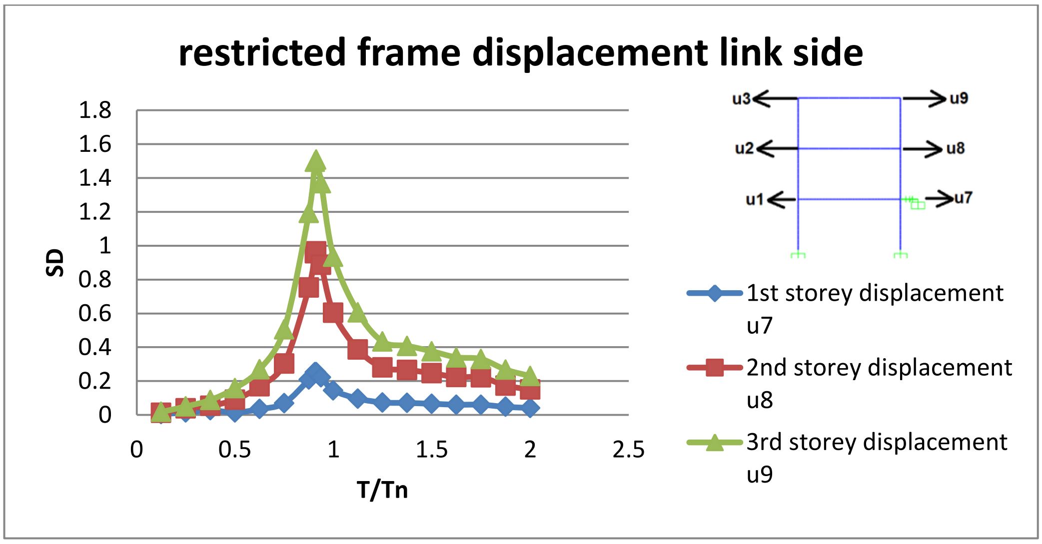


Figure (6. 51): displacement (m) results for restricted frame in link's side.

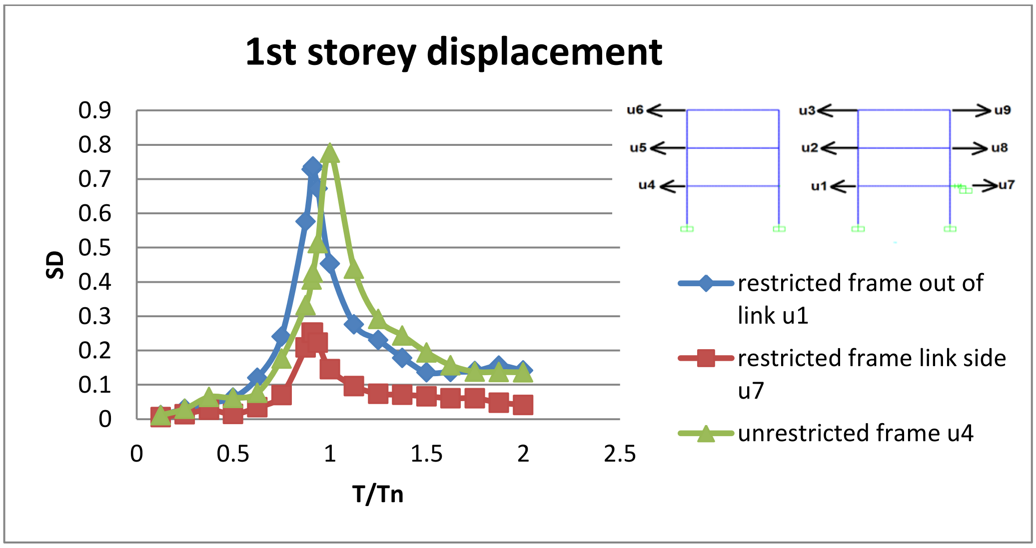


Figure (6. 52): 1st storey displacement (m) results for the two frames.

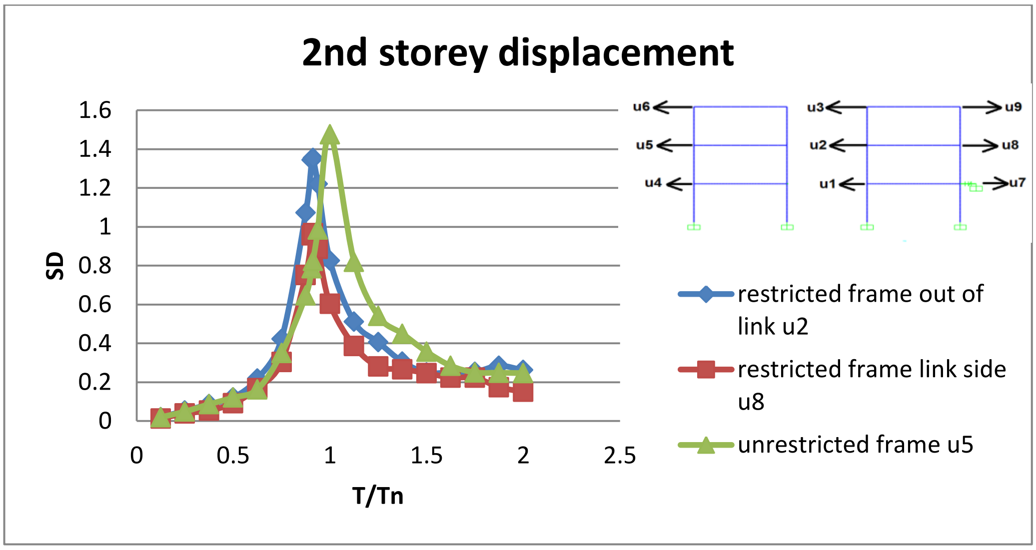


Figure (6. 53): 2nd storey displacement (m) results for the two frames.

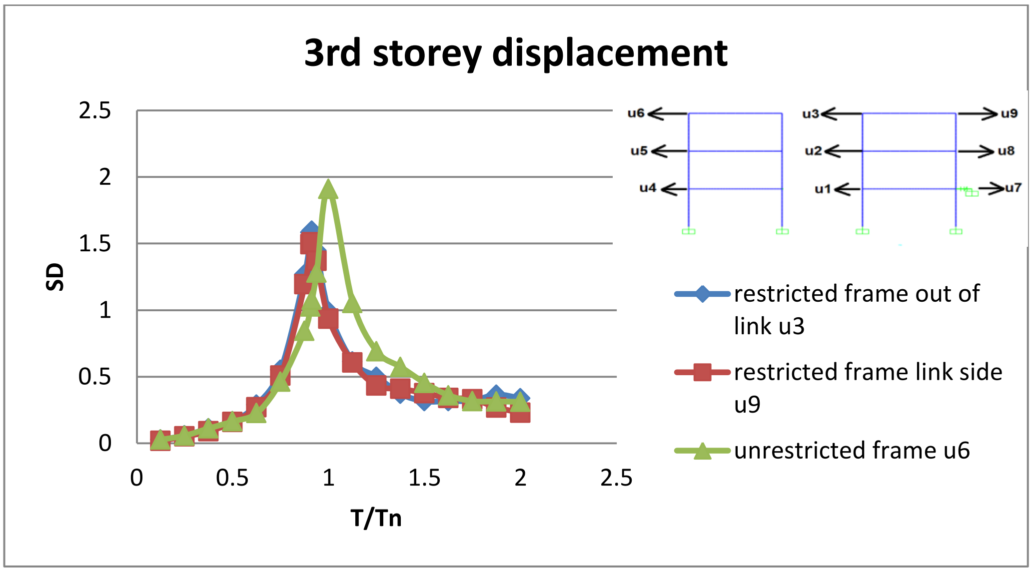


Figure (6. 54): 2nd storey displacement (m) results for the two frames.

The table and figures below summarize the computed results of relative displacement for the two frames.

Table (6. 12): relative displacement (m) results for the two frames.

Tn	T	T/Tn	u1	u2-u1	u3-u2	u4	u5-u4	u6-u5	u7	u8-u7	u9-u8
0.8	0.1	**0.125**	0.01125	0.00814	0.00717	0.0113	0.008	0.00726	0.0056	0.0074	0.00509
0.8	0.2	**0.25**	0.02994	0.02299	0.00477	0.0299	0.0187	0.00846	0.0146	0.0243	0.0116
0.8	0.3	**0.375**	0.04702	0.03902	0.02536	0.0646	0.0214	0.02549	0.0275	0.0266	0.03468
0.8	0.4	**0.5**	0.06226	0.05864	0.0424	0.062	0.0587	0.0427	0.0148	0.0763	0.06751
0.8	0.5	**0.625**	0.1203	0.0972	0.0718	0.0766	0.0864	0.0636	0.0346	0.1341	0.1006
0.8	0.6	**0.75**	0.2406	0.182	0.1291	0.1767	0.1737	0.1122	0.0699	0.234	0.2026
0.8	0.7	**0.875**	0.5759	0.4971	0.193	0.3319	0.3139	0.1994	0.2089	0.5434	0.4417
0.8	0.73	**0.9063**	0.7285	0.6155	0.239	0.4065	0.3796	0.2399	0.252	0.7042	0.5398
0.8	0.73	**0.9125**	0.7366	0.6174	0.243	0.4272	0.3976	0.2512	0.2522	0.7152	0.5416
0.8	0.75	**0.9375**	0.6721	0.5489	0.225	0.5128	0.4724	0.2968	0.2231	0.6637	0.4822
0.8	0.8	**1**	0.4537	0.3727	0.1635	0.7764	0.6996	0.434	0.1458	0.4576	0.3318
0.8	0.9	**1.125**	0.276	0.2347	0.1019	0.4381	0.3819	0.233	0.0965	0.2899	0.2187
0.8	1	**1.25**	0.2307	0.1753	0.0928	0.2917	0.2486	0.15	0.0741	0.2061	0.153
0.8	1.1	**1.375**	0.1785	0.1263	0.0649	0.2438	0.2057	0.1234	0.0711	0.1955	0.1415
0.8	1.2	**1.5**	0.1363	0.1128	0.0664	0.1946	0.162	0.0966	0.0664	0.1804	0.1294
0.8	1.3	**1.625**	0.1376	0.1127	0.0662	0.1567	0.1275	0.075	0.0606	0.1629	0.117
0.8	1.4	**1.75**	0.1401	0.1125	0.0683	0.1379	0.1121	0.0658	0.0606	0.162	0.108
0.8	1.5	**1.875**	0.1555	0.1287	0.0804	0.1374	0.1112	0.0652	0.048	0.1261	0.0938
0.8	1.6	**2**	0.1418	0.1202	0.0761	0.1364	0.11	0.0644	0.0416	0.1083	0.0801

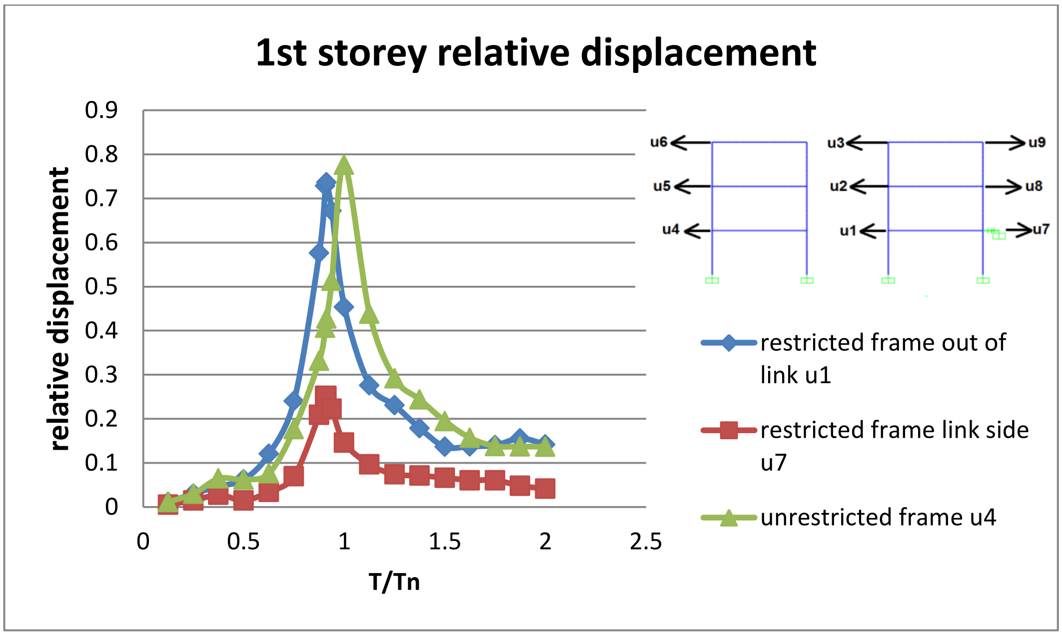


Figure (6. 55): 1st storey relative displacement (m) results for the two frames.

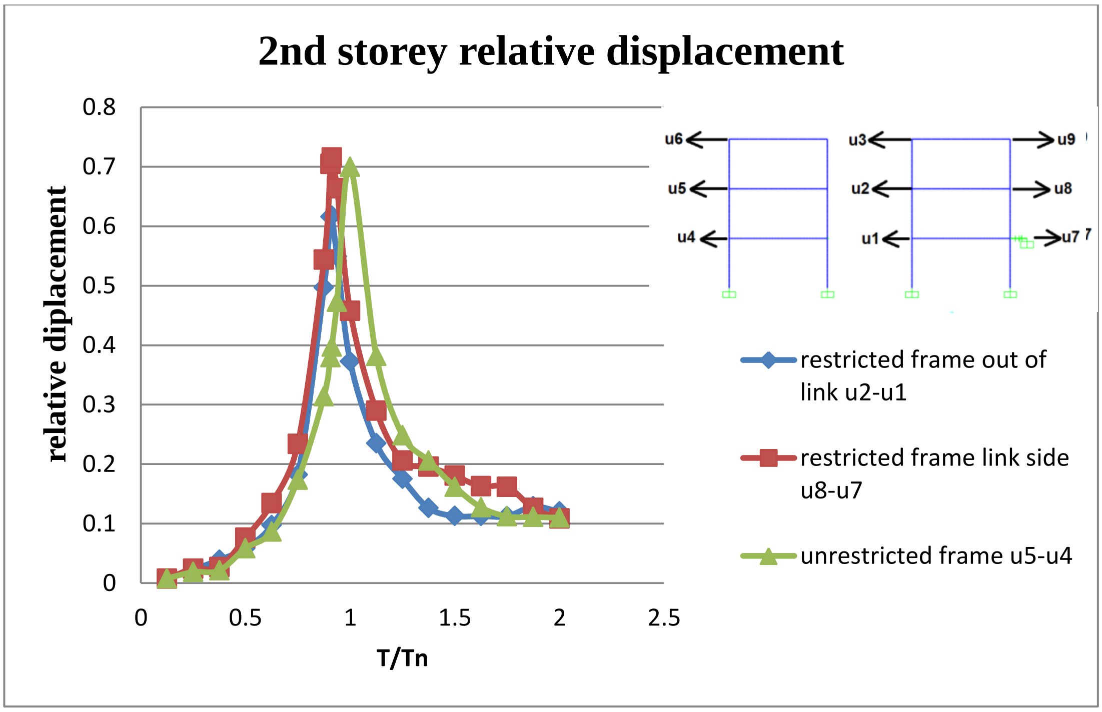


Figure (6. 56): 2nd storey relative displacement (m) results for the two frames.

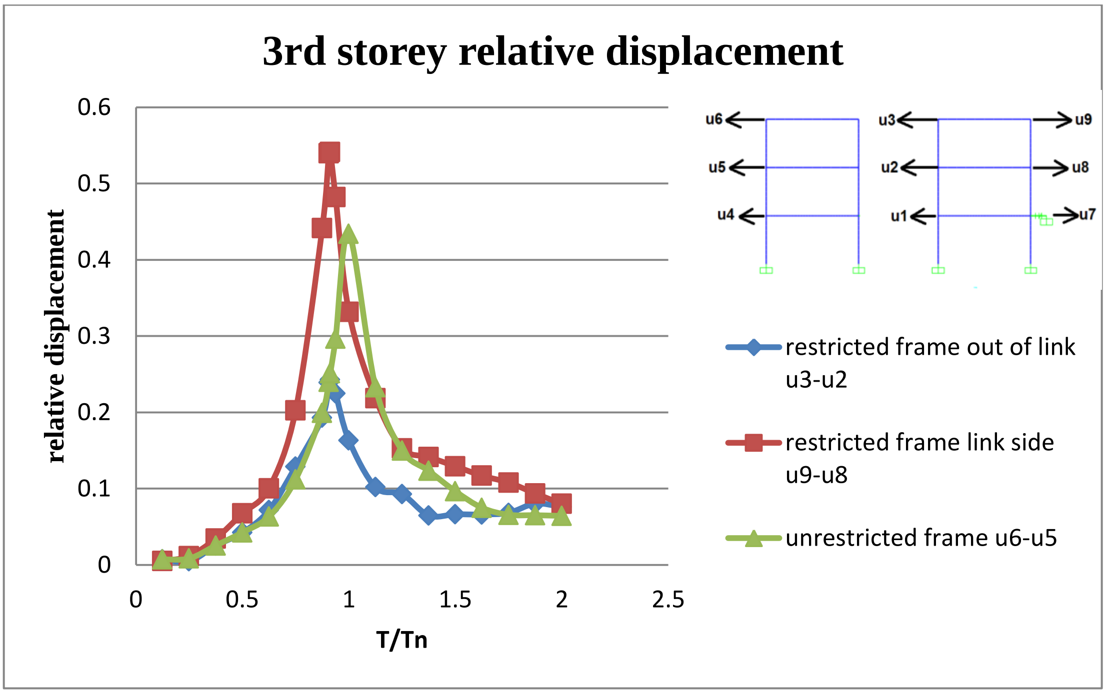


Figure (6. 57): 3rd storey relative displacement (m) results for the two frames.

Percentage of exceedance will be shown later in the next chapter, as a plot between (T/T_n) and (restricted SD/unrestricted SD).

6.5.2 The Link At 2nd Storey

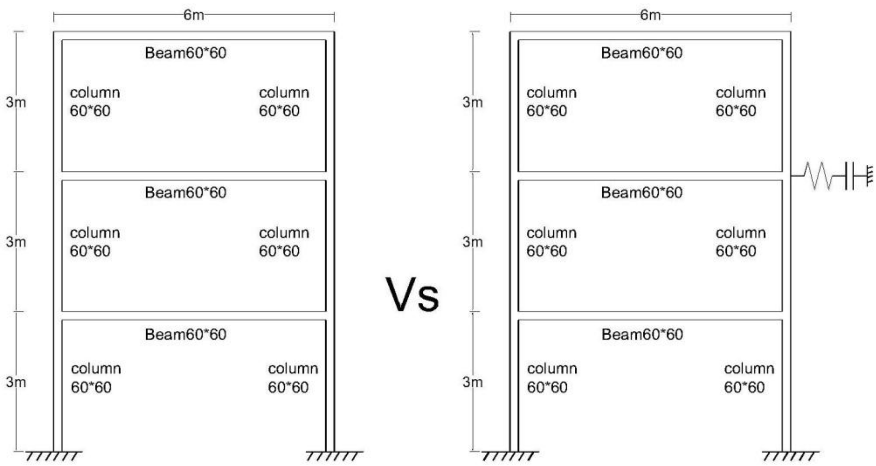


Figure (6. 58): 3DOF models, unrestricted vs. restricted with link at 2nd storey.

6.5.2.1 Response Spectrum Analysis

The two frames would be analyzed here for multi periodic sine wave has periods from 0.1sec to 2sec with same magnitude:

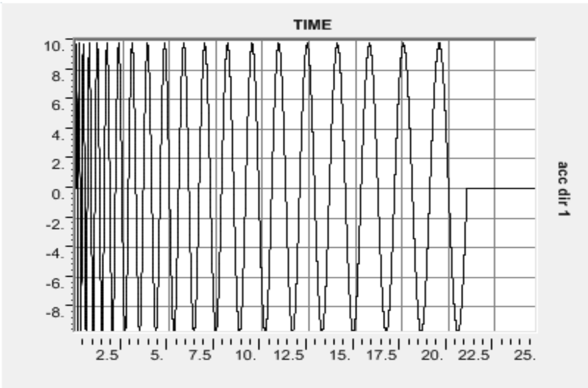


Figure (6. 59): multi periodic sine excitation.

The table below summarizes the results of displacement for the degrees of freedoms illustrated in the following figure for the two frames in two directions and the relative displacement between stories.

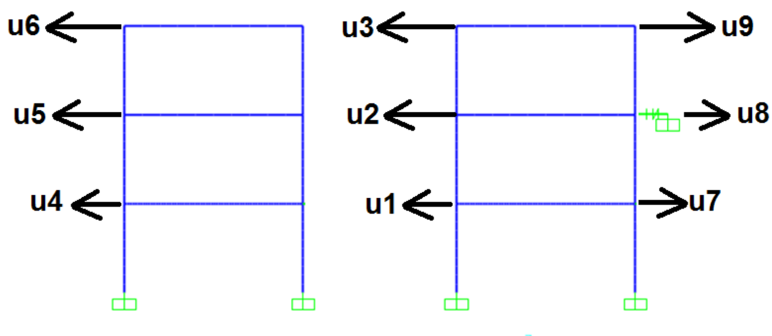


Figure (6. 60): degrees of freedom to be measured.

Table (6. 13): displacement (m) results for the two frames.

Wd	15	50	130	245	380	560	760	1000	1280	1600
T_n	**0.13862**	**0.20319**	**0.3029**	**0.4053**	**0.5001**	**0.60248**	**0.7002**	**0.8003**	**0.904**	**1.0095**
out of link										
u1	0.00809	0.01823	0.042	0.0867	0.1296	0.1969	0.2862	0.3981	0.5417	0.6801
u2	0.01296	0.02975	0.0666	0.1378	0.2041	0.3273	0.473	0.6452	0.8628	1.088
u3	0.01512	0.03416	0.0763	0.1608	0.2377	0.37	0.5362	0.7406	1.001	1.274
u4	0.00829	0.02117	0.0519	0.1091	0.1779	0.272	0.3811	0.5099	0.6636	0.8316
u5	0.01534	0.03914	0.0966	0.2036	0.3324	0.5101	0.7148	0.9586	1.249	1.568
u6	0.01943	0.04967	0.1236	0.2607	0.4262	0.6555	0.9186	1.234	1.608	2.021
Δ1L	0.00809	0.01823	0.042	0.0867	0.1296	0.1969	0.2862	0.3981	0.5417	0.6801
Δ2L	0.00487	0.01152	0.0246	0.0511	0.0745	0.1304	0.1868	0.2471	0.3211	0.4079
Δ3L	0.00216	0.00441	0.0097	0.023	0.0336	0.0427	0.0632	0.0954	0.1382	0.186
Δ1F	0.00829	0.02117	0.0519	0.1091	0.1779	0.272	0.3811	0.5099	0.6636	0.8316
Δ2F	0.00705	0.01797	0.0447	0.0945	0.1545	0.2381	0.3337	0.4487	0.5854	0.7364
Δ3F	0.00409	0.01053	0.027	0.0571	0.0938	0.1454	0.2038	0.2754	0.359	0.453
in link side										
u7	0.00336	0.00717	0.0175	0.0323	0.0515	0.08454	0.1217	0.1685	0.2231	0.2831
u8	0.0034	0.00687	0.0169	0.0368	0.061	0.094	0.1279	0.1704	0.2258	0.2862
u9	0.00864	0.02354	0.0569	0.1228	0.1939	0.2988	0.4116	0.5488	0.7174	0.8952
Δ1L+	0.00336	0.00717	0.0175	0.0323	0.0515	0.08454	0.1217	0.1685	0.2231	0.2831
Δ2L+	4.2E-05	-0.0003	-0.0006	0.0044	0.0094	0.00946	0.0062	0.0019	0.0027	0.0031
Δ3L+	0.00524	0.01667	0.04	0.086	0.133	0.2048	0.2837	0.3784	0.4916	0.609

The following figures will illustrate the behavior of each frame and compare the relative displacement for the two frames.

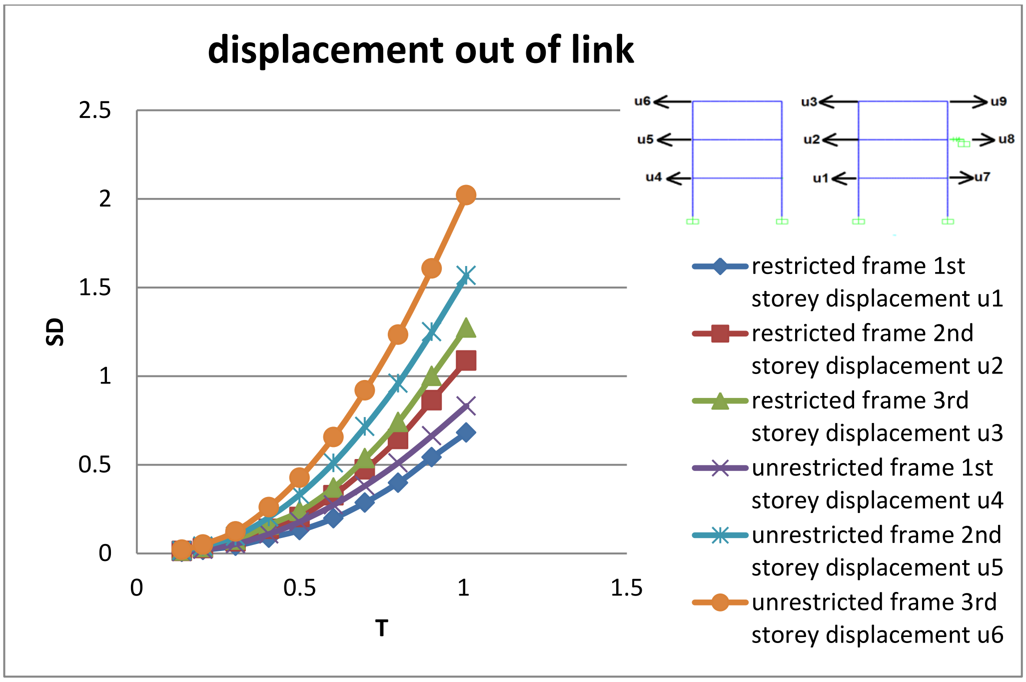


Figure (6. 61): displacement (m) results for the two frames out of link.

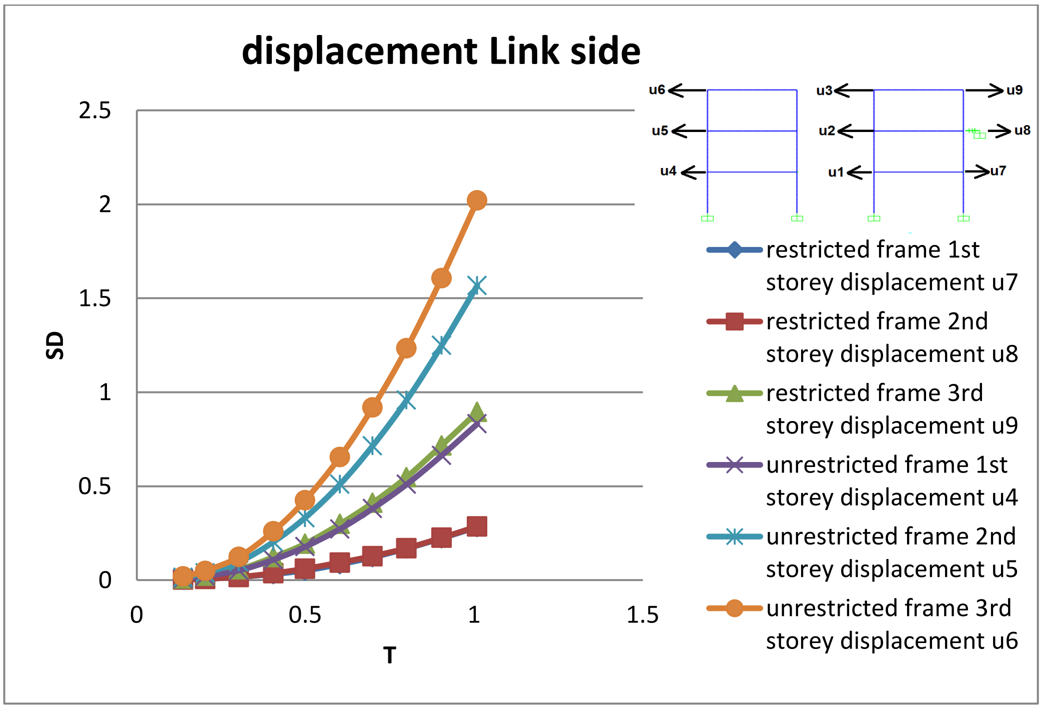


Figure (6. 62): displacement (m) results for the two frames in link side.

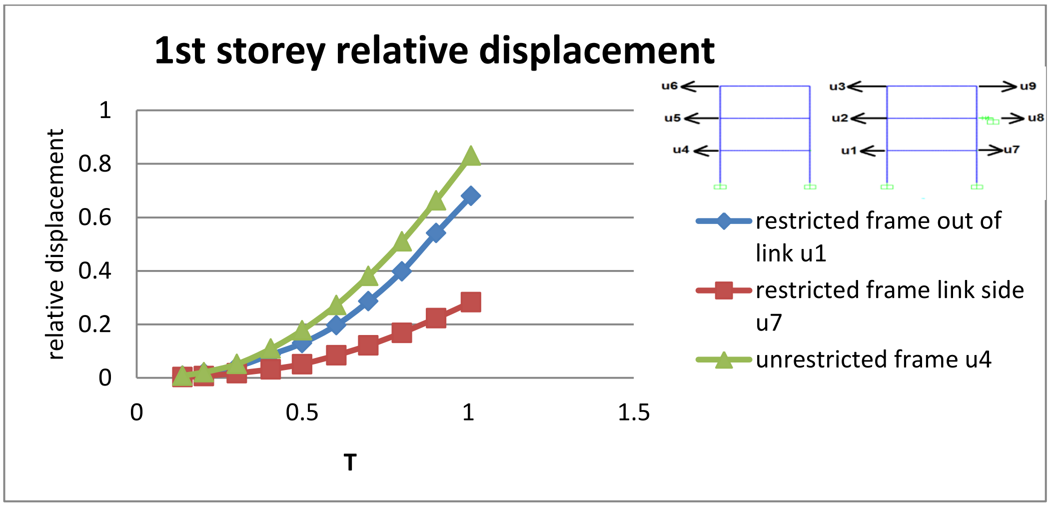


Figure (6. 63): 1st storey relative displacement (m) results for the two frames.

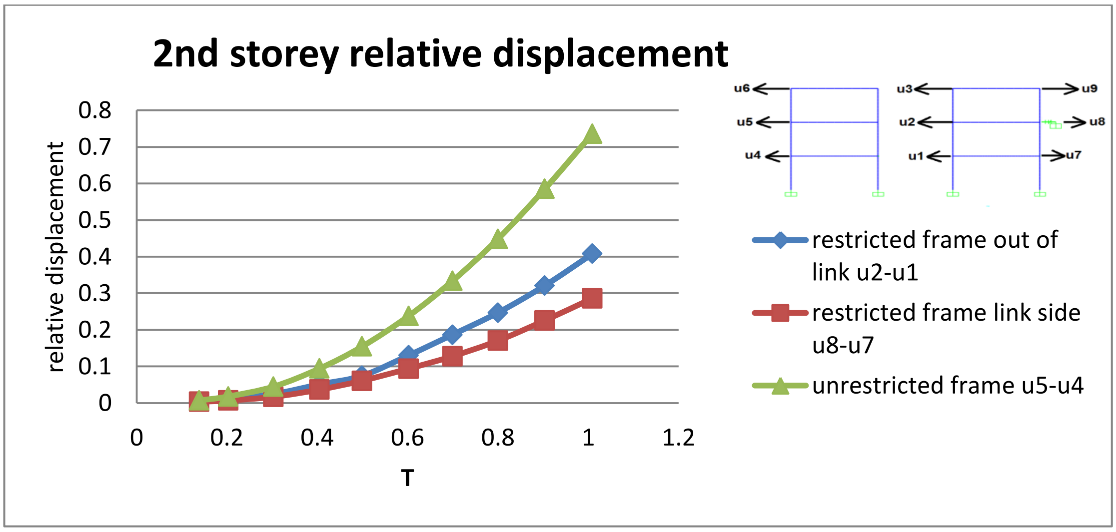


Figure (6. 64): 2nd storey relative displacement (m) results for the two frames.

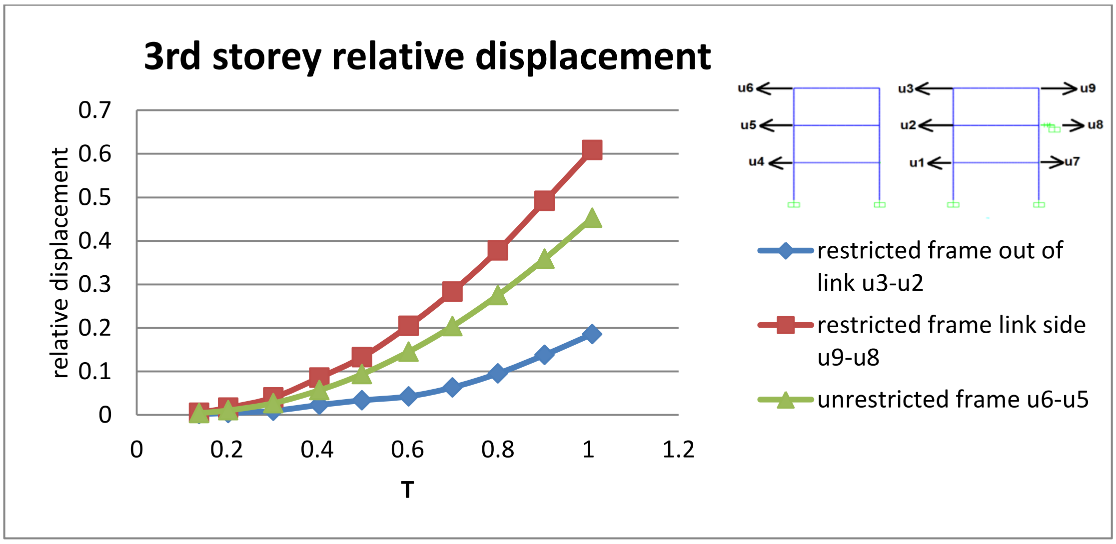


Figure (6. 65): 3rd storey relative displacement (m) results for the two frames.

6.5.2.2 Frequency Response Analysis

For the purpose of this analysis a frame of free natural period =0.8 sec and damping ratio 5% will compared to the same models of frame connected to link at 2nd storey with stiffness equal 500000 N/mm.

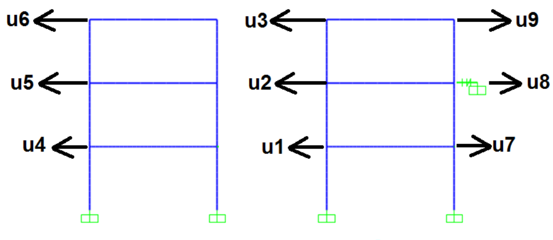


Figure (6. 66): degrees of freedom to be measured.

The table below summarizes the results of displacement for the two frames, and the following figures will illustrate the behavior of each frame.

Table (6. 14): displacement (m) results for the two frames.

T_n	T	T/T_n	u1	u2	u3	u4	u5	u6	u7	u8	u9
0.8	0.1	0.125	0.0113	0.0194	0.0266	0.0113	0.0193	0.0266	0.0069	0.00593	0.0129
0.8	0.2	0.25	0.0299	0.0485	0.057	0.0299	0.0486	0.0571	0.0208	0.02259	0.0292
0.8	0.3	0.375	0.0671	0.092	0.1606	0.0645	0.086	0.1115	0.0627	0.02172	0.078
0.8	0.4	0.5	0.0619	0.1208	0.1701	0.062	0.1207	0.1634	0.028	0.04206	0.1346
0.8	0.5	0.625	0.1825	0.2906	0.3843	0.0766	0.1629	0.2266	0.0824	0.09945	0.256
0.8	0.6	0.75	0.5642	0.8974	1.025	0.1767	0.3504	0.4626	0.163	0.2123	0.7799
0.8	0.62	0.78	0.71	1.146	1.327	0.1989	0.3905	0.5128	0.2256	0.2793	0.9922
0.8	0.63	0.781	0.7092	1.145	1.326	0.1998	0.3921	0.5148	0.2268	0.2795	0.9913
0.8	0.65	0.813	0.5707	0.9245	1.08	0.2303	0.4544	0.5993	0.208	0.2344	0.8009
0.8	0.7	0.875	0.3826	0.6158	0.7225	0.332	0.6457	0.8453	0.1594	0.1662	0.5279
0.8	0.8	1	0.244	0.3981	0.47	0.7914	1.505	1.947	0.1346	0.1209	0.3411
0.8	0.9	1.125	0.1921	0.3038	0.3628	0.438	0.82	1.053	0.101	0.09356	0.2886
0.8	1	1.25	0.1353	0.2404	0.3066	0.2916	0.5404	0.6903	0.0963	0.08921	0.2764
0.8	1.1	1.375	0.1337	0.246	0.3124	0.2437	0.4495	0.5729	0.097	0.08355	0.2585
0.8	1.2	1.5	0.1598	0.2906	0.3439	0.1946	0.3565	0.4532	0.0842	0.09164	0.2792
0.8	1.3	1.625	0.1623	0.3037	0.3607	0.1567	0.2842	0.3592	0.0767	0.07059	0.2149
0.8	1.4	1.75	0.1408	0.2532	0.3158	0.1379	0.25	0.3158	0.068	0.06377	0.1919
0.8	1.5	1.875	0.1374	0.2487	0.3138	0.1374	0.2486	0.3138	0.0602	0.05689	0.1693
0.8	1.6	2	0.1364	0.2464	0.3108	0.1364	0.2464	0.3108	0.0524	0.05601	0.1475

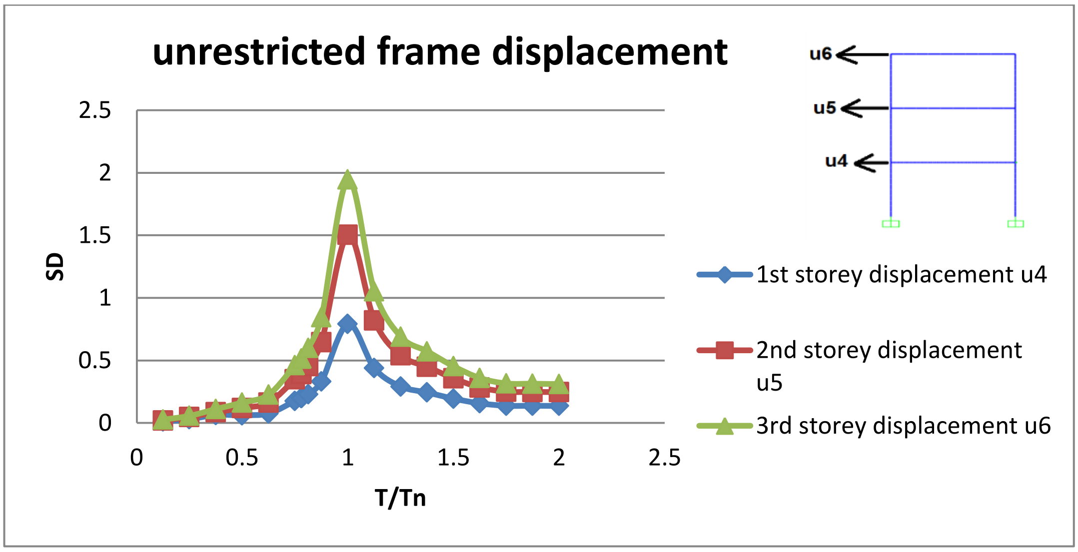


Figure (6. 67): displacement (m) results for unrestricted frame.

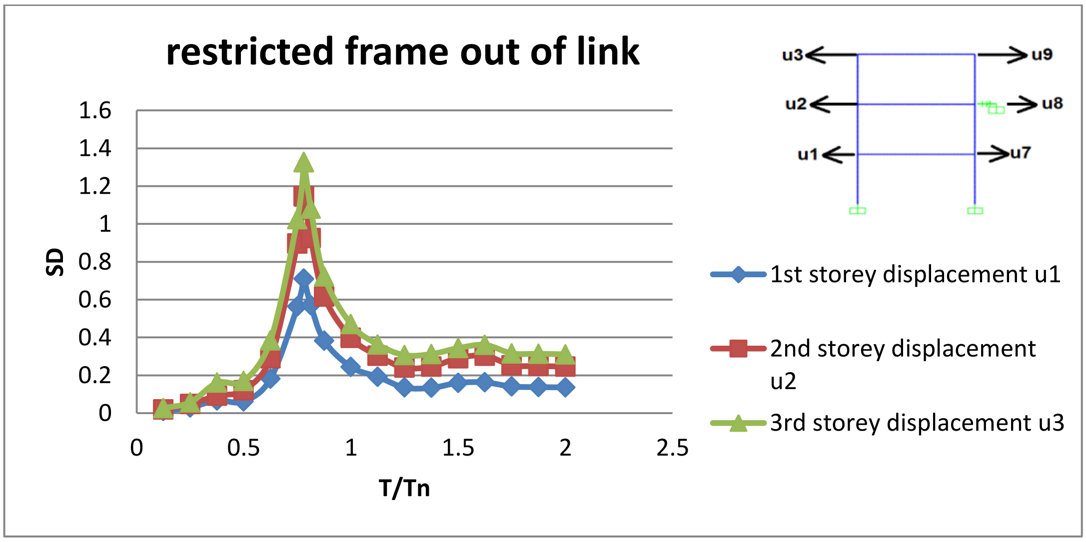


Figure (6. 68): displacement (m) results for restricted frame out of link.

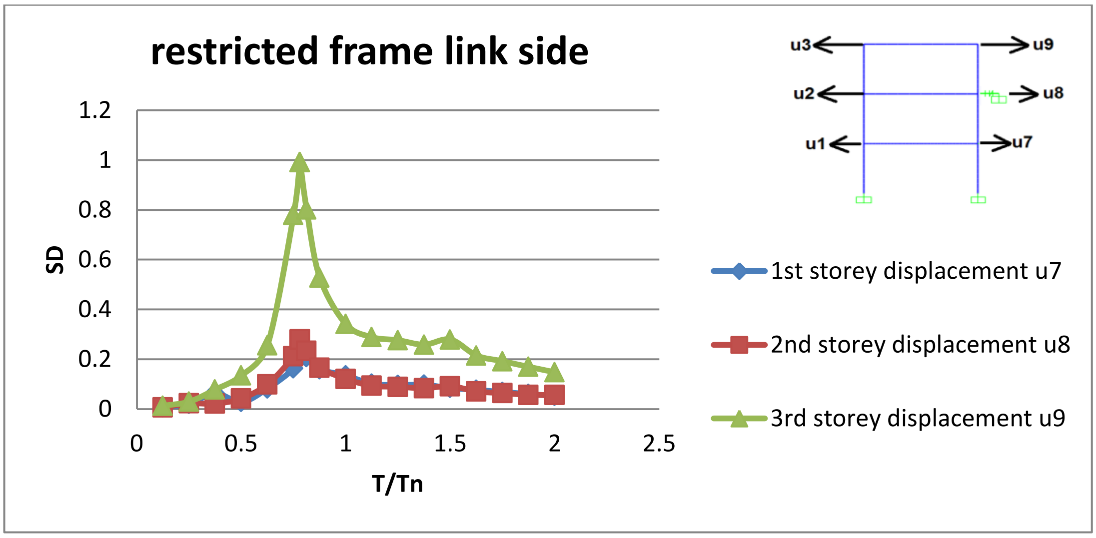


Figure (6. 69): displacement (m) results for the restricted frame in link side.

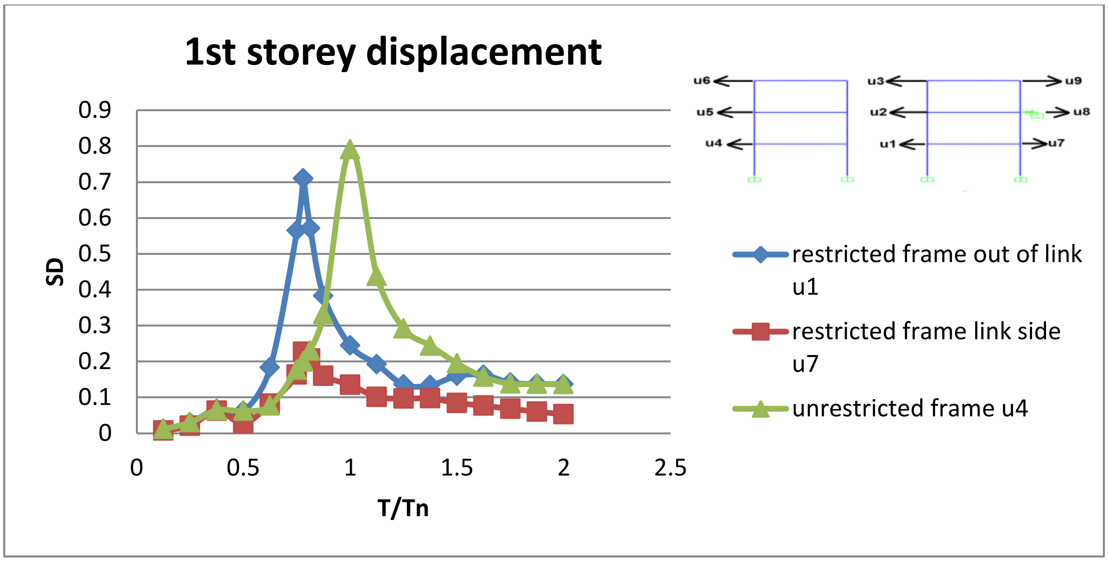


Figure (6. 70): 1st storey displacement (m) results for the two frames.

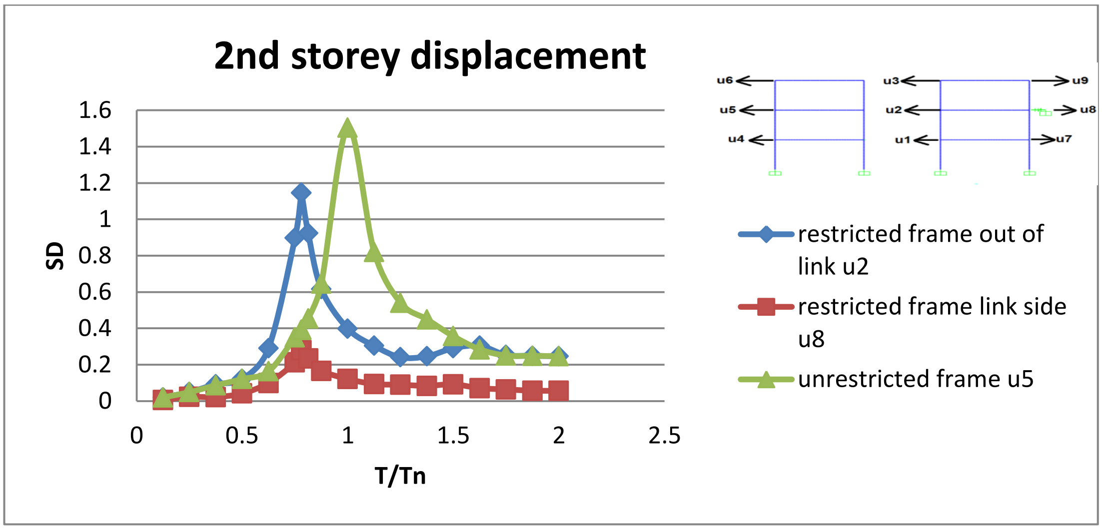


Figure (6. 71): 2nd storey displacement (m) results for the two frames.

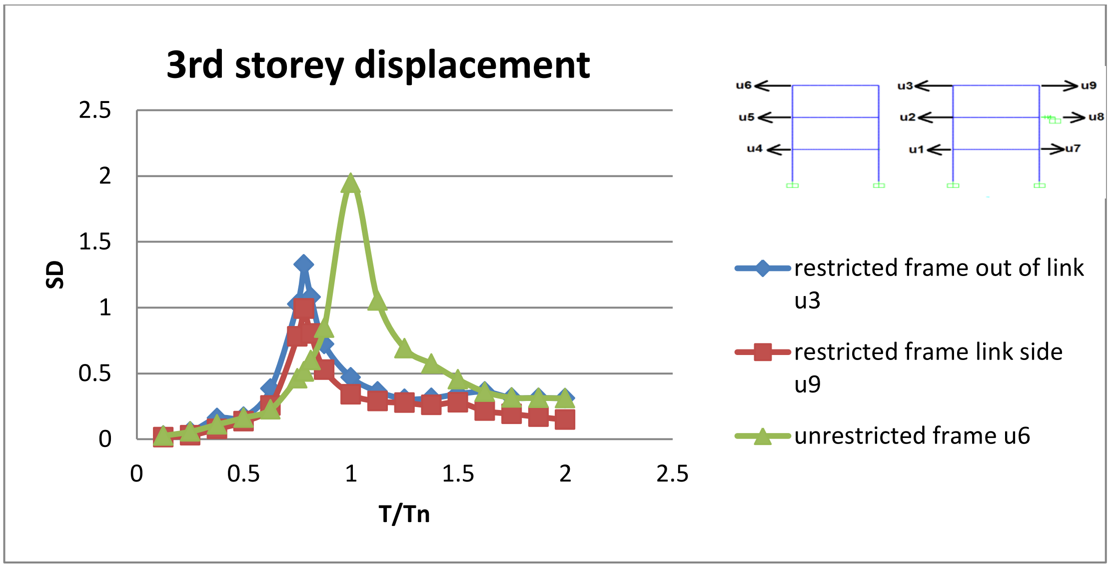


Figure (6. 72): 3rd storey displacement (m) results for the two frames.

The table and figures below summarize the computed results of relative displacement for the two frames.

Table (6. 15) relative displacement (m) results for the two frames.

T/Tn	**u1**	**u2-u1**	**u3-u2**	**u4**	**u5-u4**	**u6-u5**	**u7**	**u8-u7**	**u9-u8**
0.125	0.0113	0.00808	0.00719	0.0113	0.00803	0.00725	0.00688	-0.0009	0.00698
0.25	0.02987	0.01862	0.00854	0.02991	0.01867	0.00847	0.02075	0.00184	0.00663
0.375	0.06711	0.02493	0.06856	0.06454	0.02147	0.02549	0.06266	-0.0409	0.05625
0.5	0.06193	0.05887	0.0493	0.06203	0.05867	0.0427	0.02797	0.01409	0.09254
0.625	0.1825	0.1081	0.0937	0.07659	0.08631	0.0637	0.08242	0.01703	0.15655
0.75	0.5642	0.3332	0.1276	0.1767	0.1737	0.1122	0.163	0.0493	0.5676
0.78	0.71	0.436	0.181	0.1989	0.1916	0.1223	0.2256	0.0537	0.7129
0.78125	0.7092	0.4358	0.181	0.1998	0.1923	0.1227	0.2268	0.0527	0.7118
0.8125	0.5707	0.3538	0.1555	0.2303	0.2241	0.1449	0.208	0.0264	0.5665
0.875	0.3826	0.2332	0.1067	0.332	0.3137	0.1996	0.1594	0.0068	0.3617
1	0.244	0.1541	0.0719	0.7914	0.7136	0.442	0.1346	-0.0137	0.2202
1.125	0.1921	0.1117	0.059	0.438	0.382	0.233	0.101	-0.0074	0.19504
1.25	0.1353	0.1051	0.0662	0.2916	0.2488	0.1499	0.09631	-0.0071	0.18719
1.375	0.1337	0.1123	0.0664	0.2437	0.2058	0.1234	0.09703	-0.0135	0.17495
1.5	0.1598	0.1308	0.0533	0.1946	0.1619	0.0967	0.0842	0.00744	0.18756
1.625	0.1623	0.1414	0.057	0.1567	0.1275	0.075	0.07665	-0.0061	0.14431
1.75	0.1408	0.1124	0.0626	0.1379	0.1121	0.0658	0.06798	-0.0042	0.12813
1.875	0.1374	0.1113	0.0651	0.1374	0.1112	0.0652	0.06016	-0.0033	0.11241
2	0.1364	0.11	0.0644	0.1364	0.11	0.0644	0.0524	0.00361	0.09149

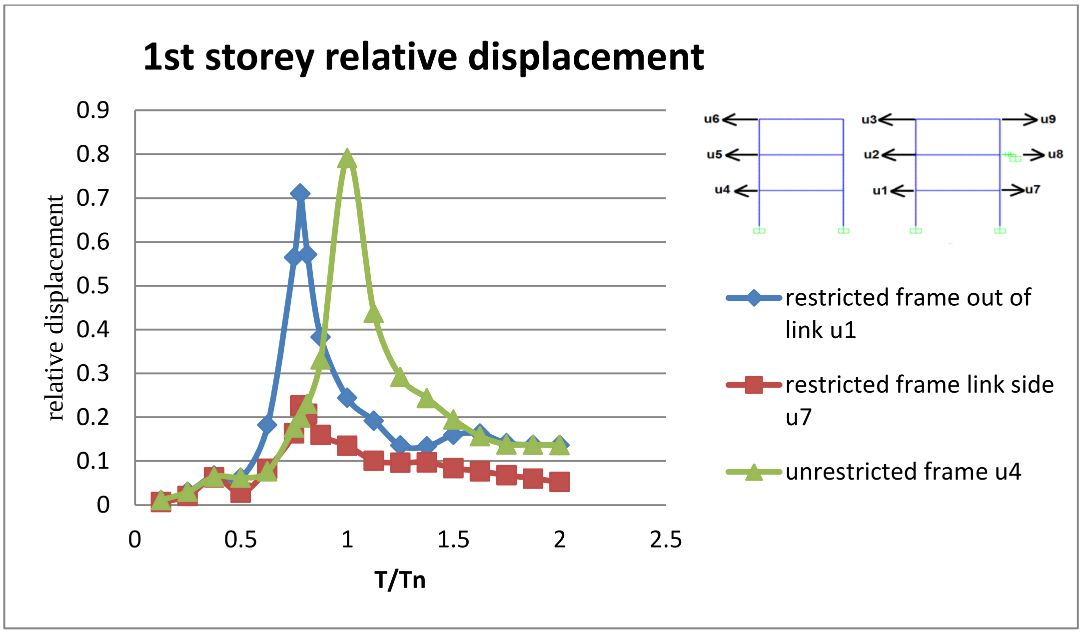


Figure (6. 73): 1st storey relative displacement (m) results for the two frames.

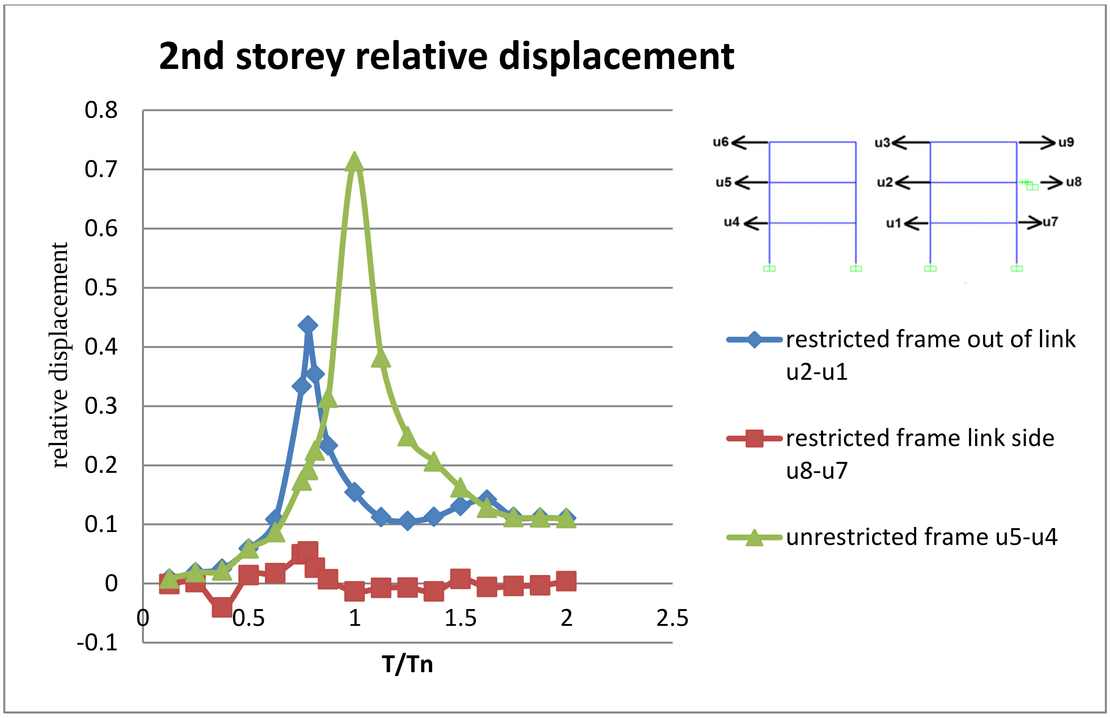


Figure (6. 74): 2nd storey relative displacement (m) results for the two frames.

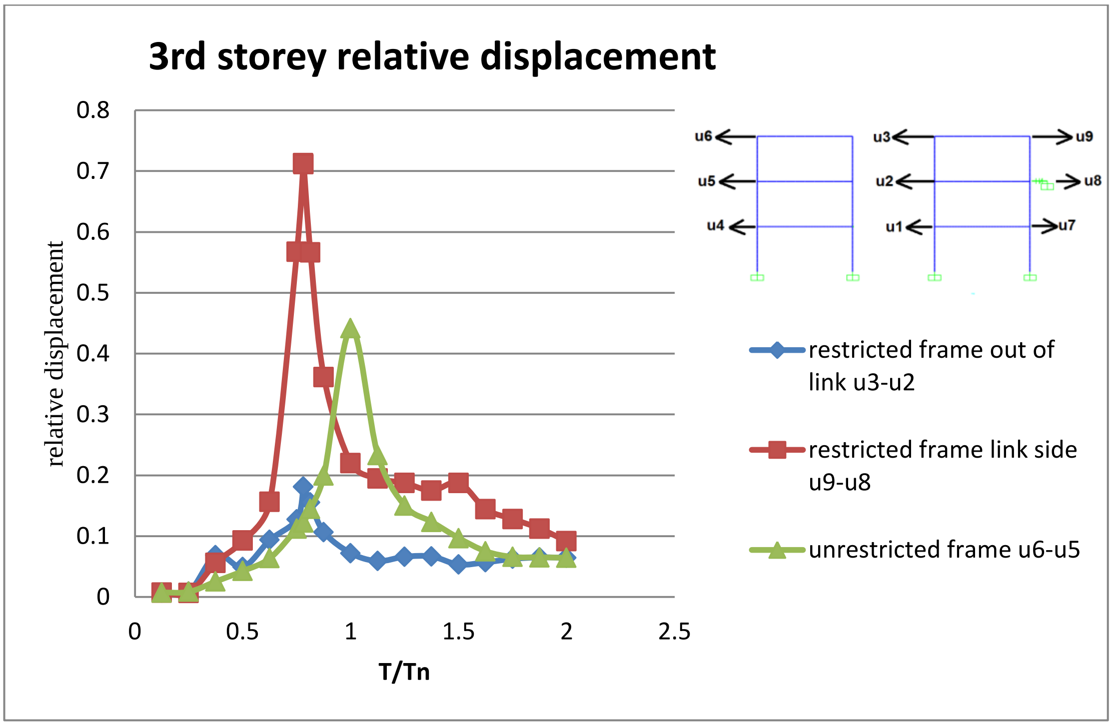


Figure (6. 75): 3rd storey relative displacement (m) results for the two frames.

Percentage of exceedance will be shown later in the next chapter, as a plot between (T/T_n) and (restricted SD/unrestricted SD).

6.5.3 The Link At 3rd Storey

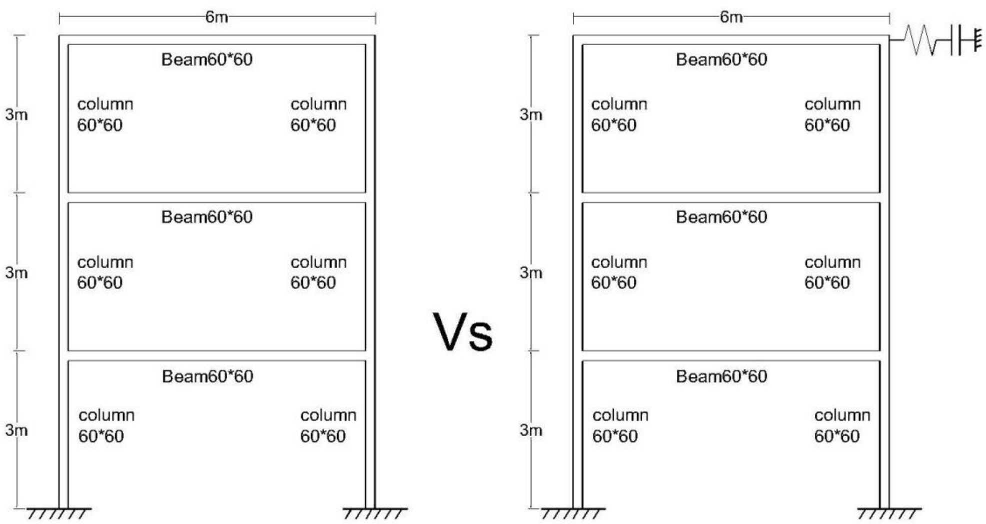


Figure (6. 76): 3DOF models, unrestricted vs. restricted with link at 3rd storey.

6.5.3.1 Response Spectrum Analysis

The two frames would be analyzed here for multi periodic sine wave has periods from 0.1sec to 2sec with same magnitude:

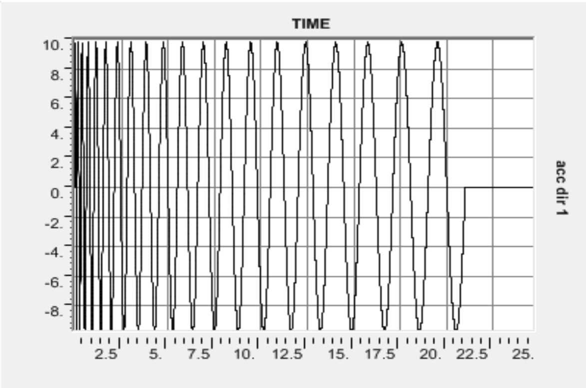


Figure (6. 77): multi periodic sine excitation.

The table below summarizes the results of displacement for the degrees of freedoms illustrated in the following figure for the two frames in two directions and the relative displacement between stories.

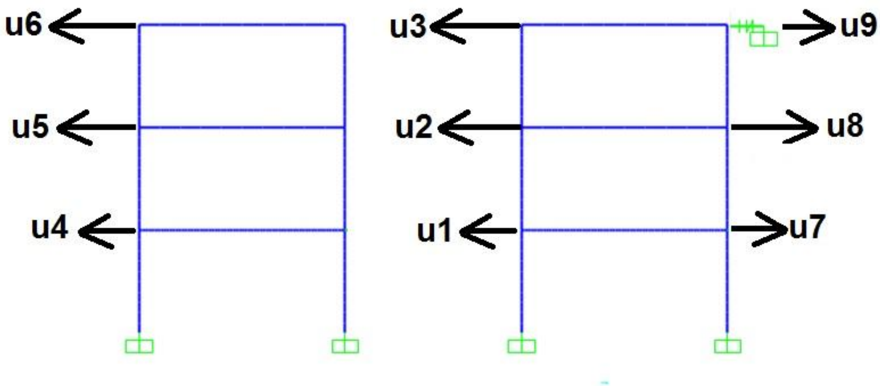


Figure (6. 78): degrees of freedoms to be measured.

Table (6. 16): displacement (m) results for the two frames.

Wd	15	50	130	245	380	560	760	1000	1280	1600
Tn	**0.13862**	**0.2032**	**0.3029**	**0.4053**	**0.5001**	**0.6025**	**0.7001**	**0.8003**	**0.904**	**1.0095**
out of link										
u1	0.00689	0.0145	0.0329	0.06287	0.09743	0.1534	0.2155	0.2867	0.3672	0.482
u2	0.01006	0.0222	0.0465	0.08572	0.1422	0.2383	0.3565	0.4631	0.5667	0.7019
u3	0.01401	0.03	0.0673	0.1385	0.214	0.3455	0.5129	0.6859	0.8692	1.08
u4	0.00829	0.0212	0.0519	0.1091	0.1779	0.272	0.3811	0.5099	0.6636	0.8316
u5	0.01534	0.0391	0.0966	0.2036	0.3324	0.5101	0.7148	0.9586	1.249	1.568
u6	0.01943	0.0497	0.1236	0.2607	0.4262	0.6555	0.9186	1.234	1.608	2.021
Δ1L	0.00689	0.0145	0.0329	0.06287	0.09743	0.1534	0.2155	0.2867	0.3672	0.482
Δ2L	0.00317	0.0076	0.0137	0.02285	0.04477	0.0849	0.141	0.1764	0.1995	0.2199
Δ3L	0.00395	0.0078	0.0208	0.05278	0.0718	0.1072	0.1564	0.2228	0.3025	0.3781
Δ1F	0.00829	0.0212	0.0519	0.1091	0.1779	0.272	0.3811	0.5099	0.6636	0.8316
Δ2F	0.00705	0.018	0.0447	0.0945	0.1545	0.2381	0.3337	0.4487	0.5854	0.7364
Δ3F	0.00409	0.0105	0.027	0.0571	0.0938	0.1454	0.2038	0.2754	0.359	0.453
in link side										
u7	0.00369	0.013	0.0309	0.05977	0.08001	0.1285	0.1981	0.2703	0.3206	0.4095
u8	0.00527	0.0165	0.0407	0.07907	0.1133	0.1695	0.2388	0.3326	0.4277	0.5612
u9	0.00377	0.0078	0.0182	0.03745	0.06985	0.1082	0.1468	0.1851	0.2248	0.3128
Δ1L+	0.00369	0.013	0.0309	0.05977	0.08001	0.1285	0.1981	0.2703	0.3206	0.4095
Δ2L+	0.00159	0.0035	0.0098	0.0193	0.03329	0.041	0.0407	0.0623	0.1071	0.1517
Δ3L+	-0.0015	-0.0087	-0.0225	-0.0416	-0.0435	-0.061	-0.092	-0.148	-0.2029	-0.248

The following figures will illustrate the behavior of each frame and compare the relative displacement for the two frames.

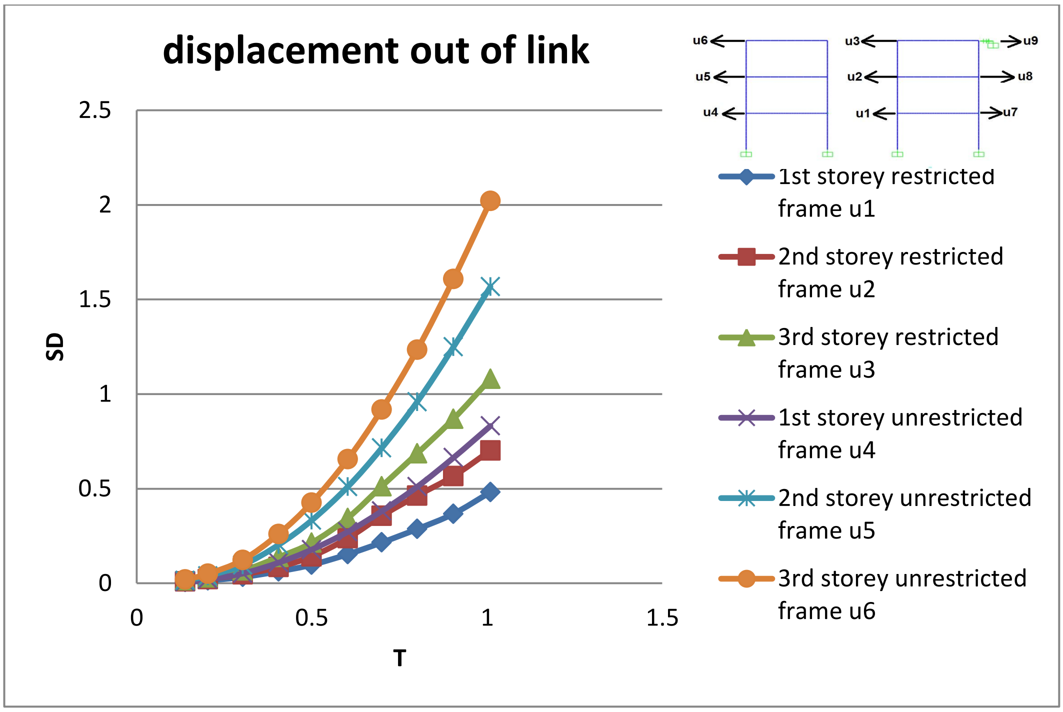


Figure (6. 79): displacement (m) results of the two frames out of link.

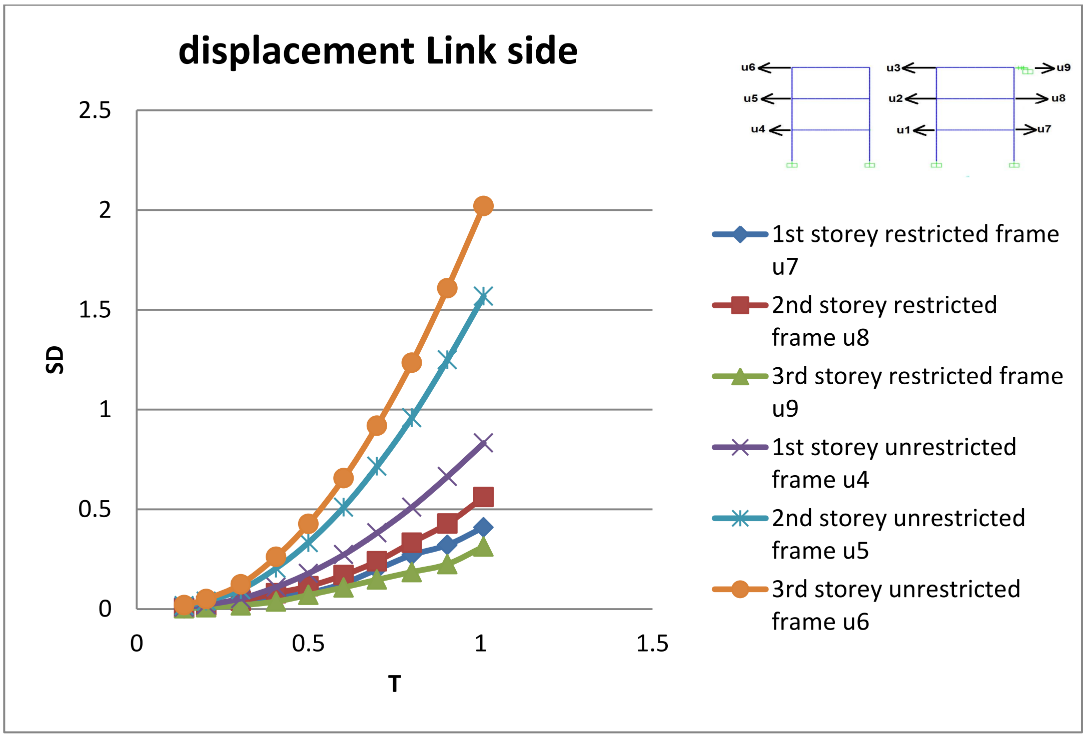


Figure (6. 80): displacement (m) results of the two frames in link side.

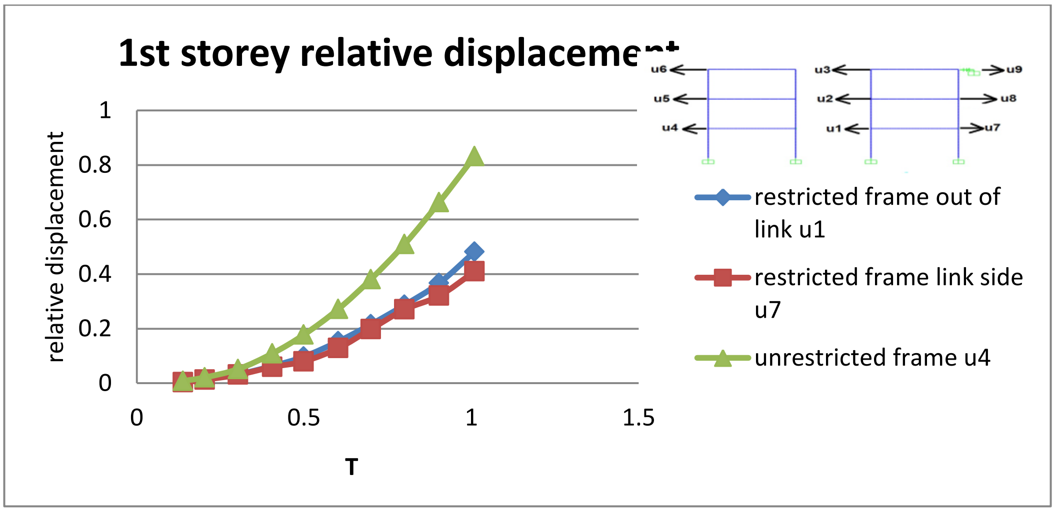


Figure (6. 81): 1st storey relative displacement (m) results for the two frames.

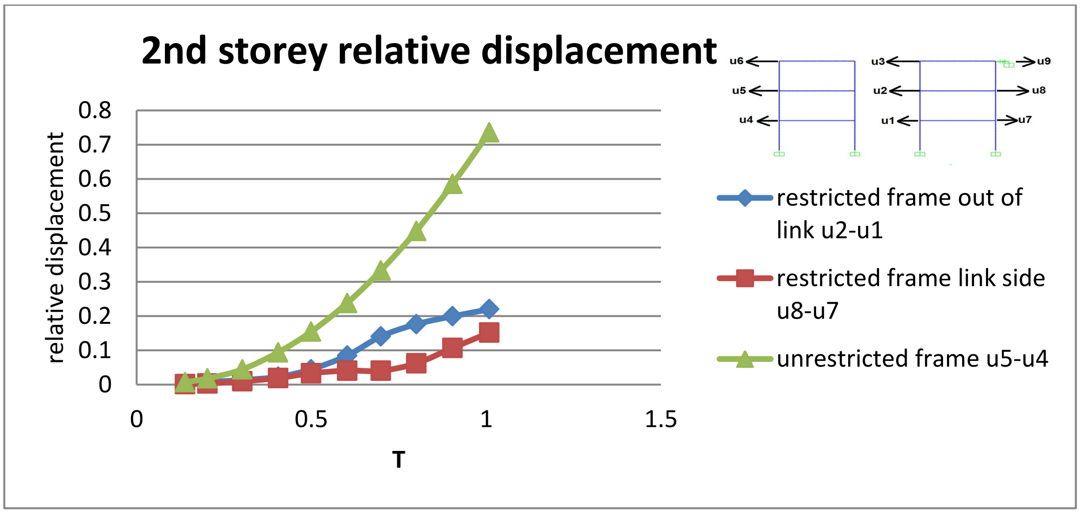


Figure (6. 82): 2nd storey relative displacement (m) results for the two frames.

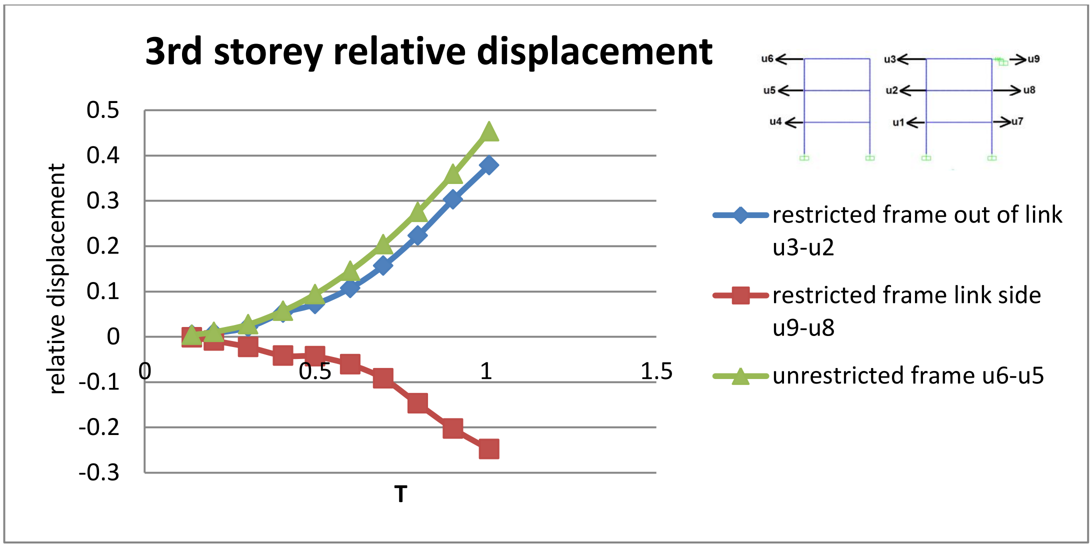


Figure (6. 83): 3rd storey relative displacement (m) results for the two frames.

6.5.3.2 Frequency Response Analysis

For the purpose of this analysis a frame of free natural period =0.8 sec and damping ratio 5% will compared to the same models of frame connected to link at 3rd storey with stiffness equal 500000 N/mm.

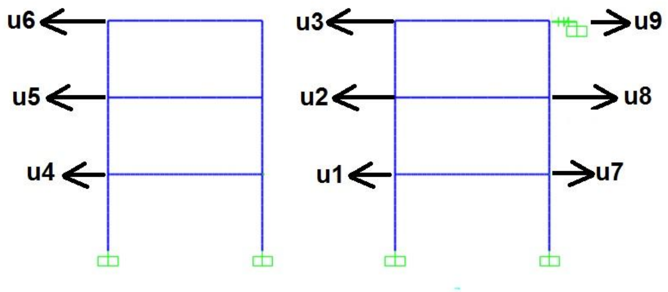


Figure (6. 84): degrees of freedom to be measured.

The table below summarizes the results of displacement for the two frames, and the following figures will illustrate the behavior of each frame.

Table (6. 17): displacement (m) results for the two frames.

Tn	T	T/Tn	u1	u2	u3	u4	u5	u6	u7	u8	u9
0.8	0.1	**0.125**	0.0113	0.0193	0.0266	0.0113	0.0193	0.0266	0.0102	0.0103	0.0052
0.8	0.2	**0.25**	0.0299	0.0486	0.057	0.0299	0.0486	0.0571	0.0211	0.0347	0.0113
0.8	0.3	**0.375**	0.1569	0.2405	0.3617	0.0646	0.086	0.1115	0.1466	0.1787	0.0594
0.8	0.4	**0.5**	0.127	0.1765	0.2148	0.062	0.1207	0.1634	0.0332	0.0779	0.0665
0.8	0.5	**0.625**	0.3422	0.4204	0.4913	0.0766	0.1629	0.2266	0.1269	0.1652	0.1946
0.8	0.6	**0.75**	0.4222	0.612	1.017	0.1767	0.3504	0.4626	0.3995	0.5018	0.1889
0.8	0.607	**0.7581**	0.4279	0.628	1.033	0.1832	0.3612	0.4767	0.4017	0.516	0.1984
0.8	0.65	**0.8125**	0.3189	0.4824	0.7589	0.2302	0.4546	0.5992	0.2827	0.3944	0.1867
0.8	0.7	**0.875**	0.2424	0.3666	0.5548	0.332	0.6457	0.8453	0.2025	0.2901	0.1623
0.8	0.8	**1**	0.2025	0.305	0.3714	0.7914	1.505	1.947	0.1626	0.2064	0.1368
0.8	0.9	**1.125**	0.161	0.2383	0.2975	0.438	0.82	1.053	0.1524	0.2031	0.112
0.8	1	**1.25**	0.1405	0.2403	0.3066	0.2916	0.5404	0.6903	0.1469	0.1934	0.1129
0.8	1.1	**1.375**	0.1623	0.2512	0.3176	0.2437	0.4495	0.5729	0.1528	0.1817	0.0999
0.8	1.2	**1.5**	0.1537	0.2776	0.3768	0.1946	0.3565	0.4532	0.1272	0.1652	0.0919
0.8	1.3	**1.625**	0.1376	0.2502	0.3164	0.1567	0.2842	0.3592	0.1153	0.1495	0.0834
0.8	1.4	**1.75**	0.1379	0.25	0.3157	0.1379	0.25	0.3158	0.1029	0.1336	0.0749
0.8	1.5	**1.875**	0.1374	0.2486	0.3137	0.1374	0.2486	0.3138	0.0921	0.1181	0.0666
0.8	1.6	**2**	0.1445	0.2546	0.3233	0.1364	0.2464	0.3108	0.0907	0.118	0.0628

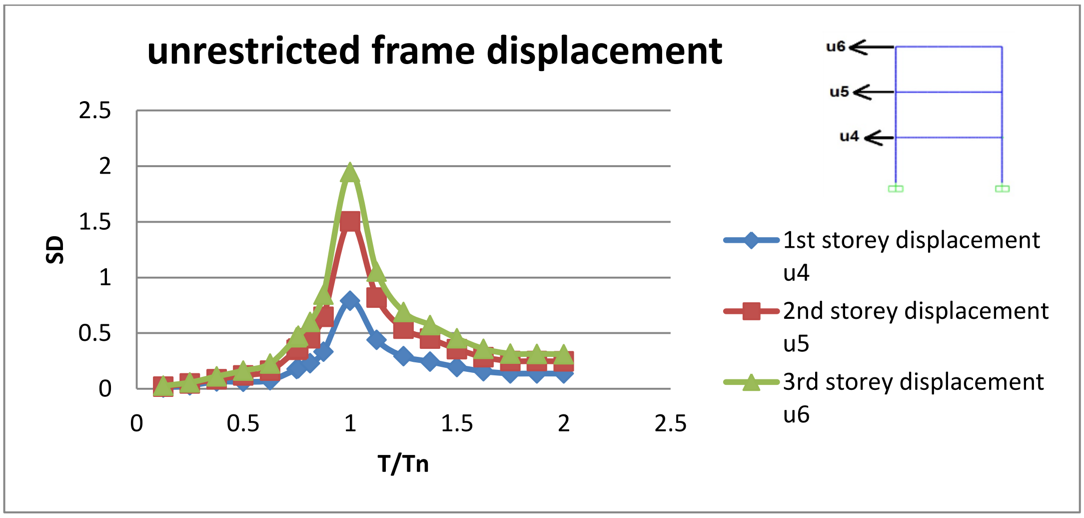


Figure (6. 85): displacement (m) results for unrestricted frame.

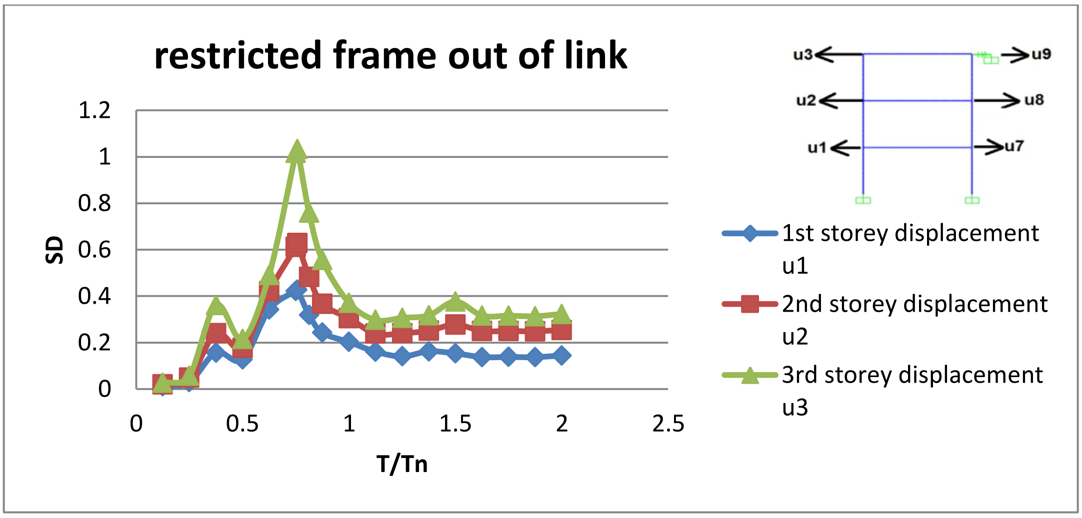


Figure (6. 86): displacement (m) results for restricted frame out of link.

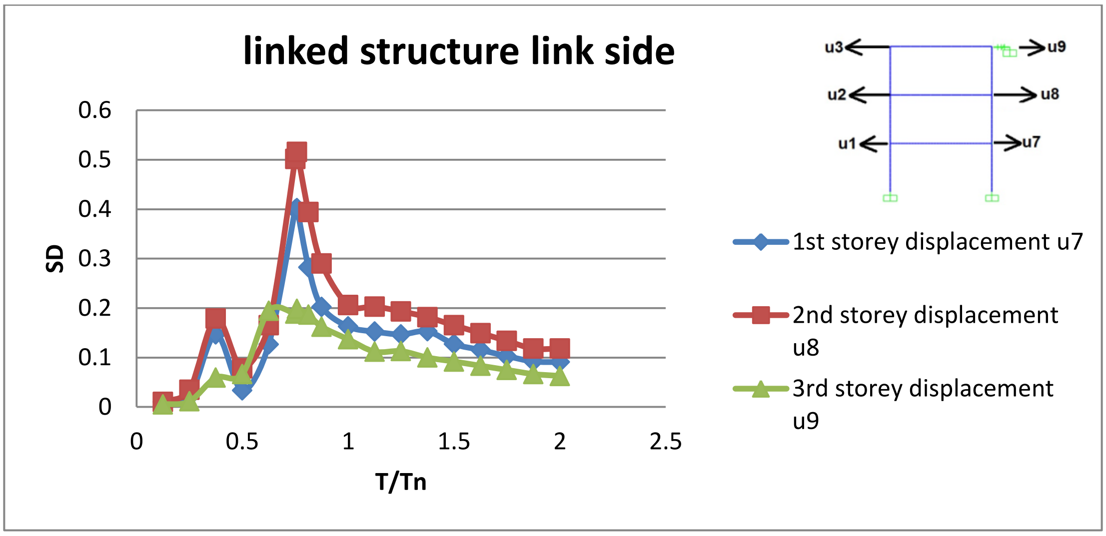


Figure (6. 87): displacement (m) results for restricted frame link side.

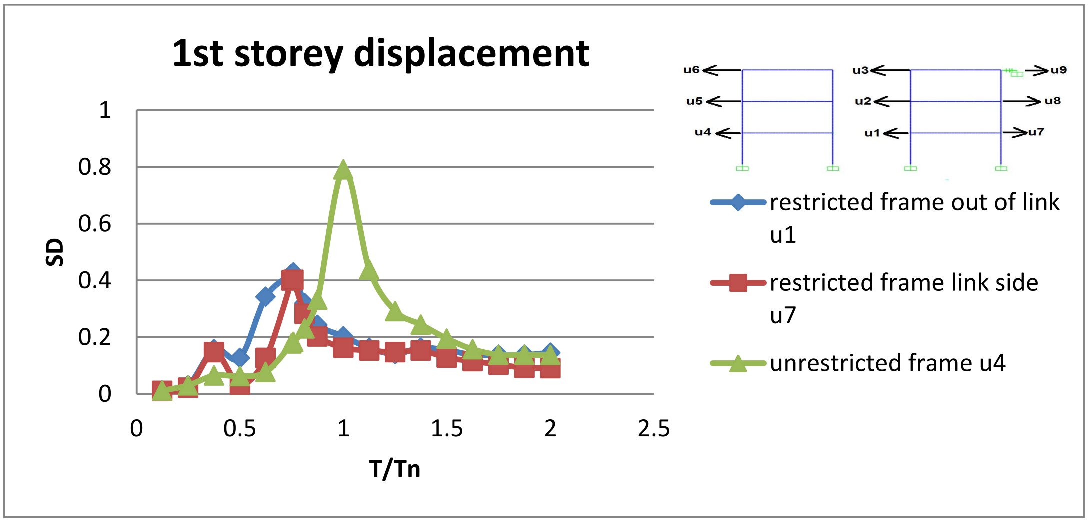


Figure (6. 88): 1st storey displacement (m) results for the two frames.

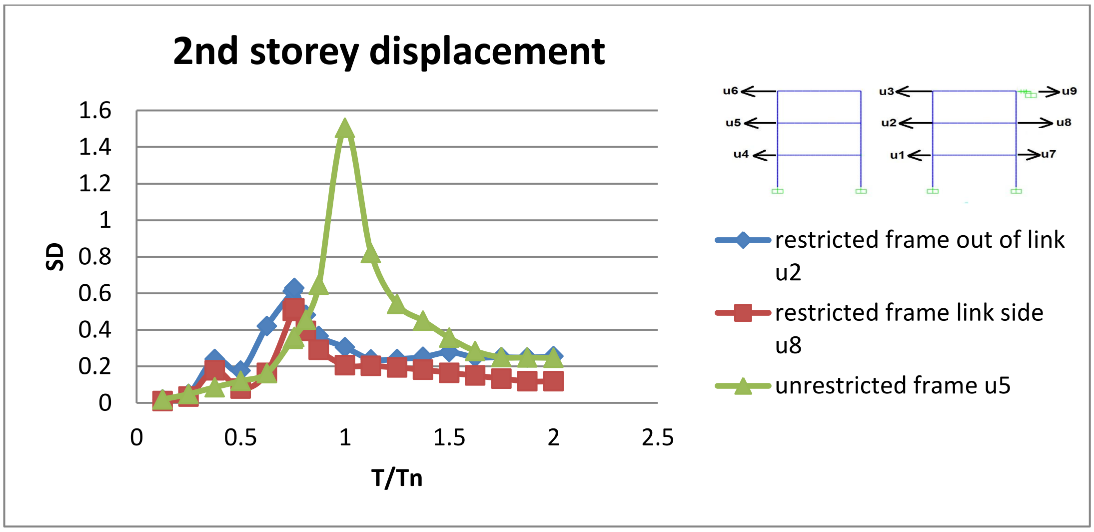


Figure (6. 89): 2nd storey displacement (m) results for the two frames.

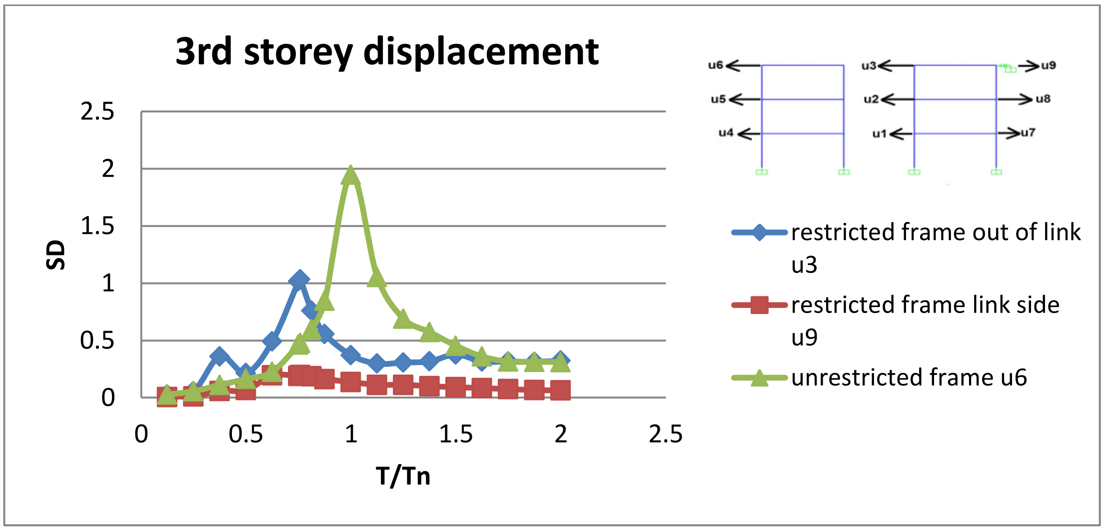


Figure (6. 90): 3rd storey displacement (m) results for the two frames.

The table and figures below summarize the computed results of relative displacement for the two frames.

Table (6. 18): relative displacement (m) results for the two frames.

T/Tn	u1	u2-u1	u3-u2	u4	u5-u4	u6-u5	u7	u8-u7	u9-u8
0.125	0.0113	0.00802	0.00723	0.0113	0.008	0.00725	0.01015	0.00012	-0.0051
0.25	0.02989	0.01867	0.00848	0.02991	0.0187	0.00848	0.02108	0.01366	-0.0234
0.375	0.1569	0.0836	0.1212	0.06456	0.0215	0.02549	0.1466	0.0321	-0.1193
0.5	0.127	0.0495	0.0383	0.06203	0.0587	0.0427	0.03321	0.04466	-0.0114
0.625	0.3422	0.0782	0.0709	0.07659	0.0863	0.0637	0.1269	0.0383	0.0294
0.75	0.4222	0.1898	0.405	0.1767	0.1737	0.1122	0.3995	0.1023	-0.3129
0.75813	0.4279	0.2001	0.405	0.1832	0.178	0.1155	0.4017	0.1143	-0.3176
0.8125	0.3189	0.1635	0.2765	0.2302	0.2244	0.1446	0.2827	0.1117	-0.2077
0.875	0.2424	0.1242	0.1882	0.332	0.3137	0.1996	0.2025	0.0876	-0.1278
1	0.2025	0.1025	0.0664	0.7914	0.7136	0.442	0.1626	0.0438	-0.0696
1.125	0.161	0.0773	0.0592	0.438	0.382	0.233	0.1524	0.0507	-0.0911
1.25	0.1405	0.0998	0.0663	0.2916	0.2488	0.1499	0.1469	0.0465	-0.0805
1.375	0.1623	0.0889	0.0664	0.2437	0.2058	0.1234	0.1528	0.0289	-0.0818
1.5	0.1537	0.1239	0.0992	0.1946	0.1619	0.0967	0.1272	0.038	-0.0733
1.625	0.1376	0.1126	0.0662	0.1567	0.1275	0.075	0.1153	0.0342	-0.0661
1.75	0.1379	0.1121	0.0657	0.1379	0.1121	0.0658	0.1029	0.0307	-0.0587
1.875	0.1374	0.1112	0.0651	0.1374	0.1112	0.0652	0.09205	0.02605	-0.0515
2	0.1445	0.1101	0.0687	0.1364	0.11	0.0644	0.09073	0.02727	-0.0552

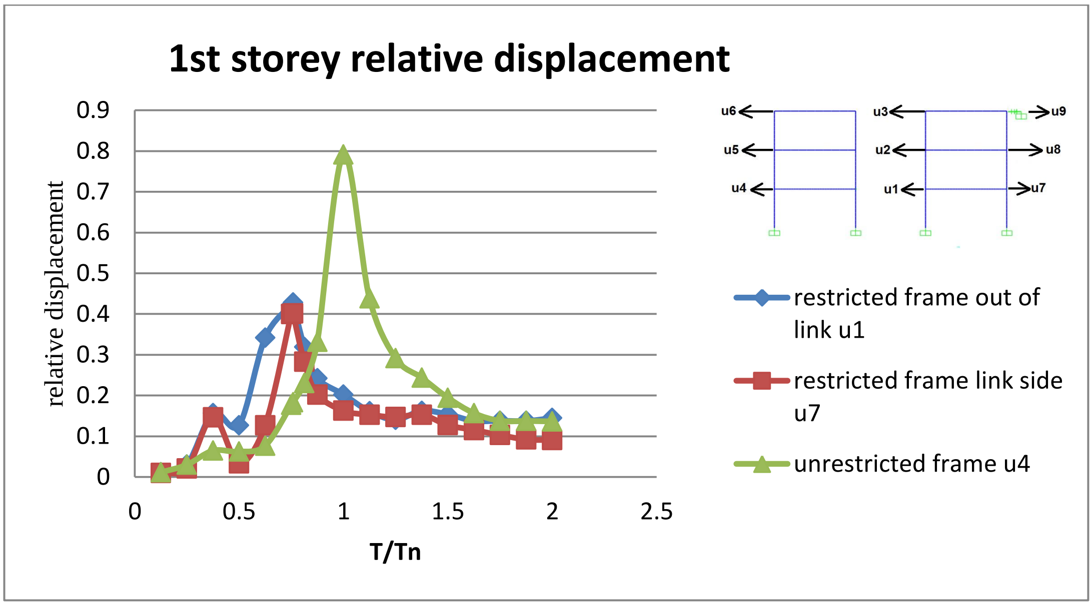


Figure (6. 91): 1st storey relative displacement (m) results for the two frames.

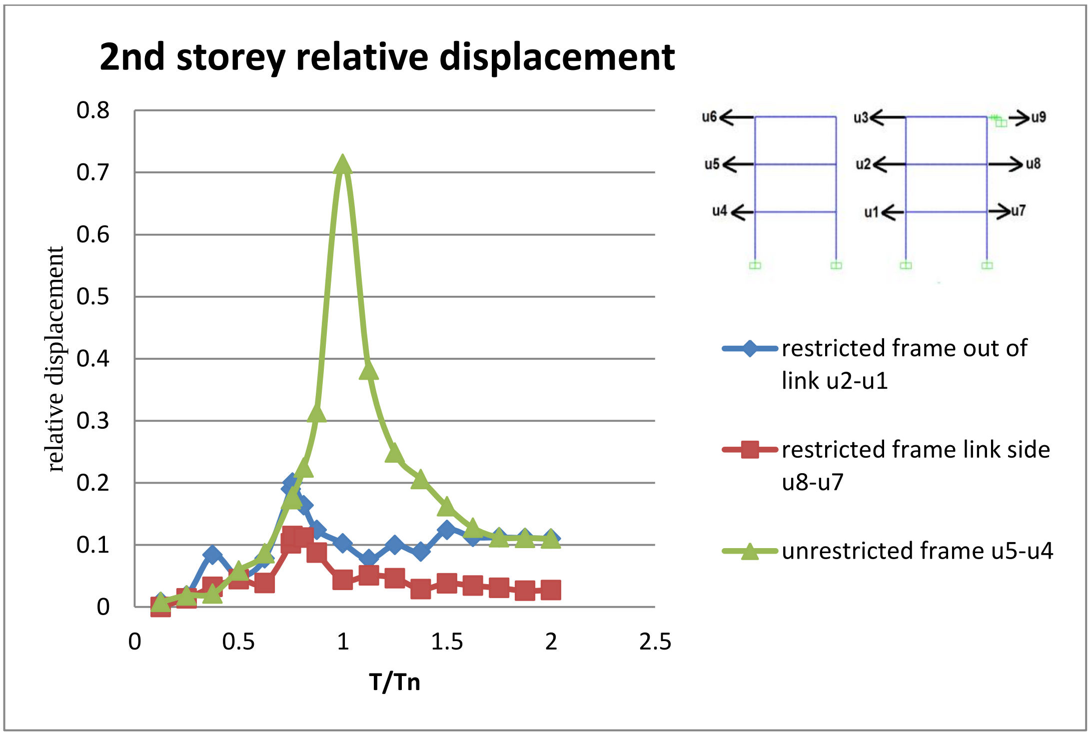


Figure (6. 92): 2nd storey relative displacement (m) results for the two frames.

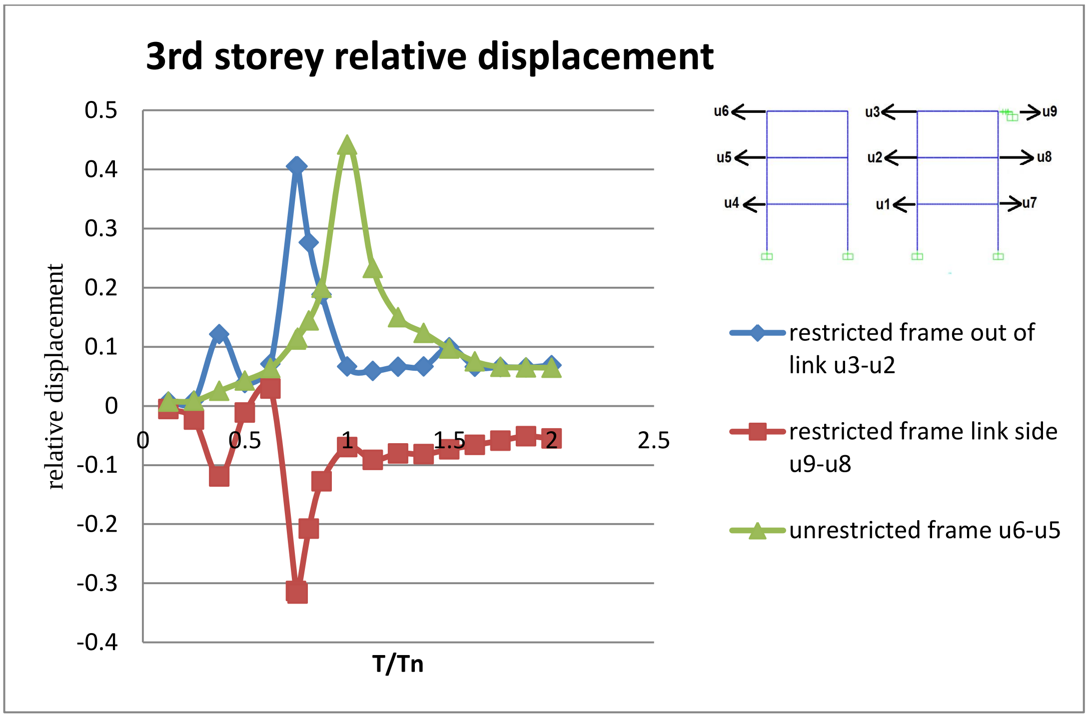


Figure (6. 93): 3rd storey relative displacement (m) results for the two frames.

Percentage of exceedance will be shown later in the next chapter, as a plot between (T/T_n) and (restricted SD/unrestricted SD).

6.6 Commentaries

6.6.1 Response spectrum analysis:

Ideal harmonic multi periodic excitations were applied to all possible configurations of link location for SDOF, 2DOF and 3DOF models.

The displacement and relative displacement results was shown in graphs, From these graphs, one can directly read the maximum relative displacement of both models. It was shown that for certain natural periods the linked structure may have a larger response for a certain ground motion.

6.6.2 Frequency response analysis:

For the purpose of this analysis a frame of determined unrestricted natural period and damping ratio 5% compared to the same models of frame connected to link with (link stiffness / structure stiffness) =15.

The displacement graphs show that unrestricted structure has its maximum displacement exactly at excitation period equals the natural period "T/T_n =1". For restricted models, the structure resonates at lower period.

From these graphs, the change in natural period was illustrated and the maximum response of restricted and unrestricted models could be compared.

The next chapter could illustrate the percentage of exceedance in relative displacement.

Chapter Seven
Normalization of Results

7.1 General

In statistics and applications of statistics, normalization can have a range of meanings. In the simplest cases, normalization of ratings means adjusting values measured on different scales to a common scale, often prior to averaging. In more complicated cases, normalization may refer to more sophisticated adjustments where the intention is to bring the entire probability distributions of adjusted values into alignment.

In another usage in statistics, normalization refers to the creation of shifted and scaled versions of statistics, where the intention is that these normalized values allow the comparison of corresponding normalized values for different datasets in a way that eliminates the effects of certain gross influences, as in an anomaly time series. Some types of normalization involve only a rescaling, to arrive at values relative to some size variable. In terms of levels of measurement, such ratios only make sense for ratio measurements (where ratios of measurements are meaningful), not interval measurements (where only distances are meaningful, but not ratios).

The point of normalization is to make variables comparable to each other, in this study the reference of results is the unrestricted frame analysis, so the plots of relative displacements of each case in the previous chapter that computed based on frequency response method will be normalized as a ratio to the results of unrestricted case of the same model.

7.2 Single degree of freedom model

The following table summarizes the results of displacement ratio for restricted frame with link stiffness /structure's stiffness =15 to the same frame without restrictions.

Table (7. 1): The ratio of restricted displacement to unrestricted displacement out of link.

T/Tn	unrestricted	restricted	ratio
0.1	0.02516	0.02856	1.135135
0.15	0.04053	0.1075	2.652356
0.2	0.05116	0.2288	4.472244
0.3	0.09938	0.4676	4.705172
0.4	0.1507	0.2872	1.905773
0.5	0.1997	0.3887	1.94642
0.6	0.3205	1.904	5.940718
0.65	0.3366	1.353	4.019608
0.7	0.4606	0.7733	1.678897
0.8	0.7252	0.5329	0.734832
0.85	0.9306	0.4928	0.529551
0.9	1.245	0.4187	0.336305
1	2.362	0.4191	0.177434
1.1	1.501	0.3754	0.2501

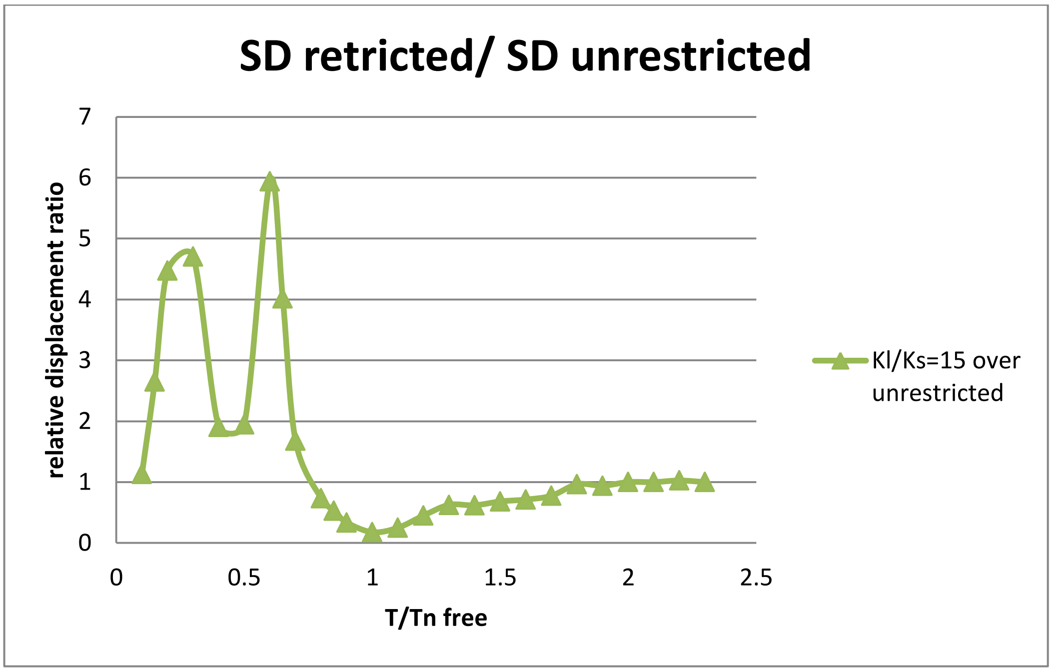


Figure (7. 1): The ratio of restricted displacement to unrestricted displacement out of link.

7.3 Two degrees of freedom model

7.3.1 The link at 1st storey

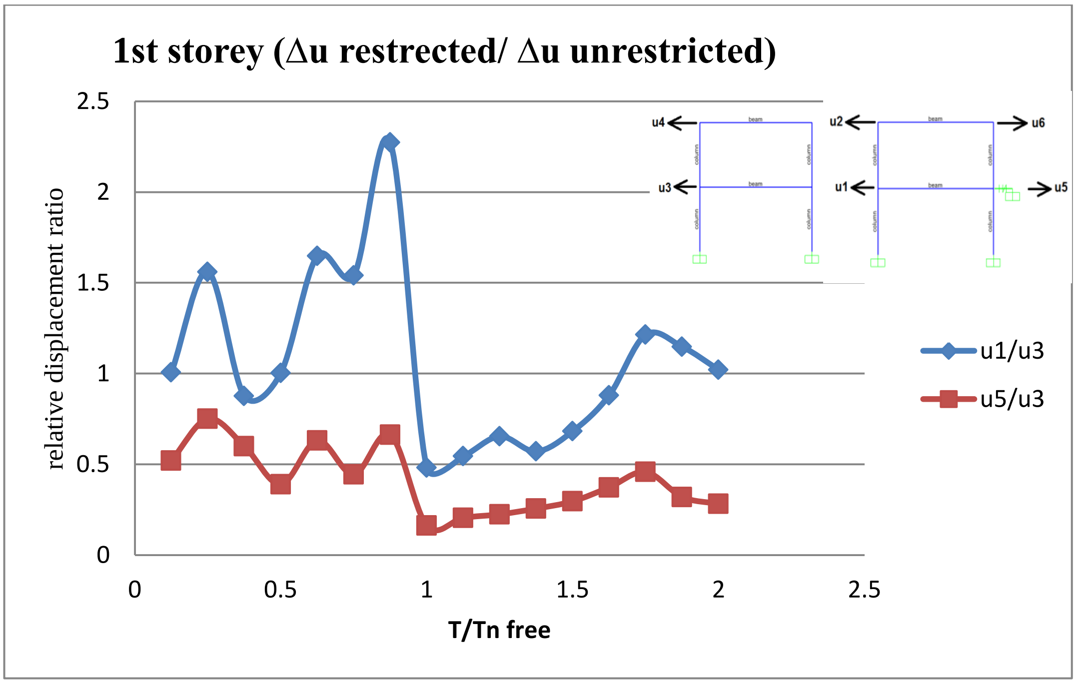


Figure (7. 2): 1st storey Relative displacement ratio.

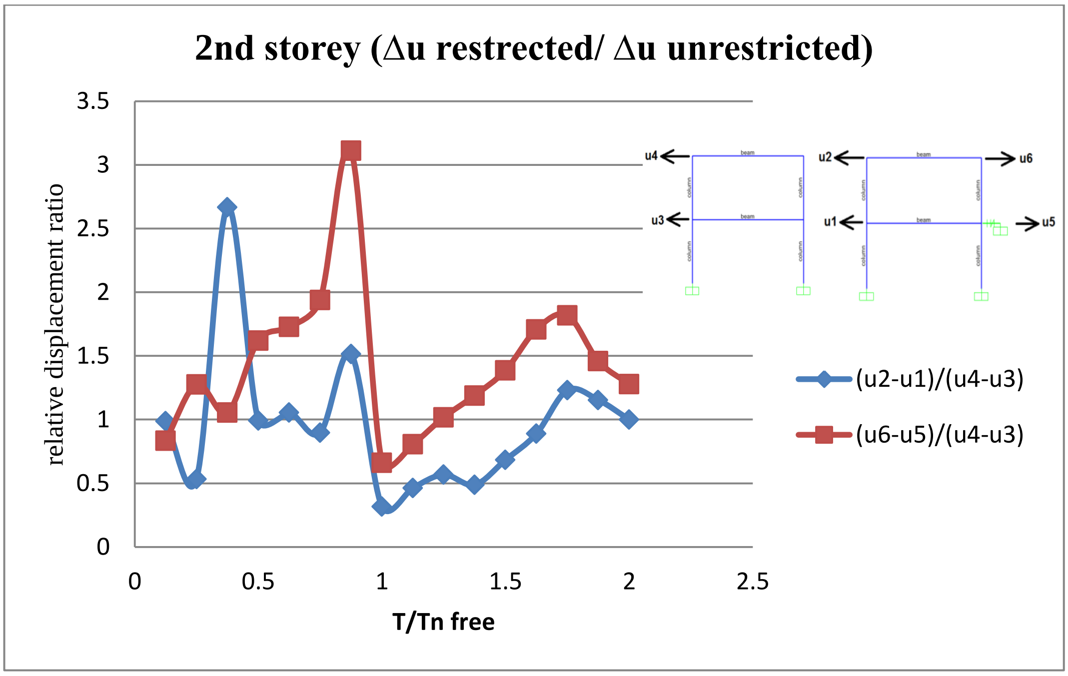


Figure (7. 3): 2nd storey Relative displacement ratio.

7.3.1 The link at 2nd storey

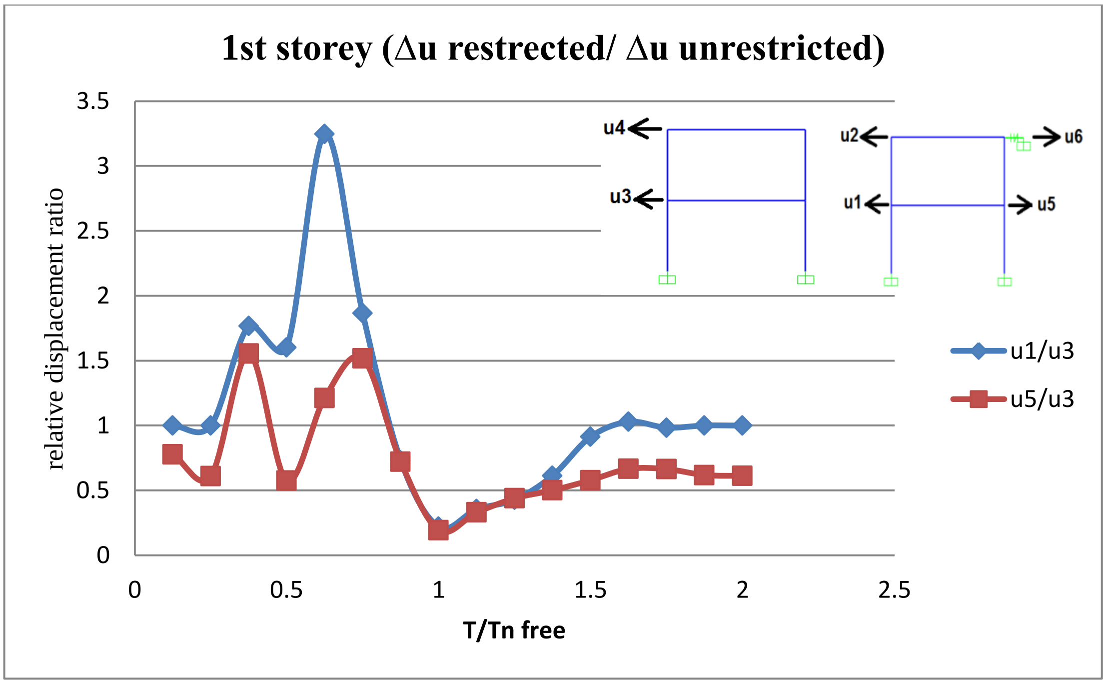


Figure (7. 4): 1st storey Relative displacement ratio.

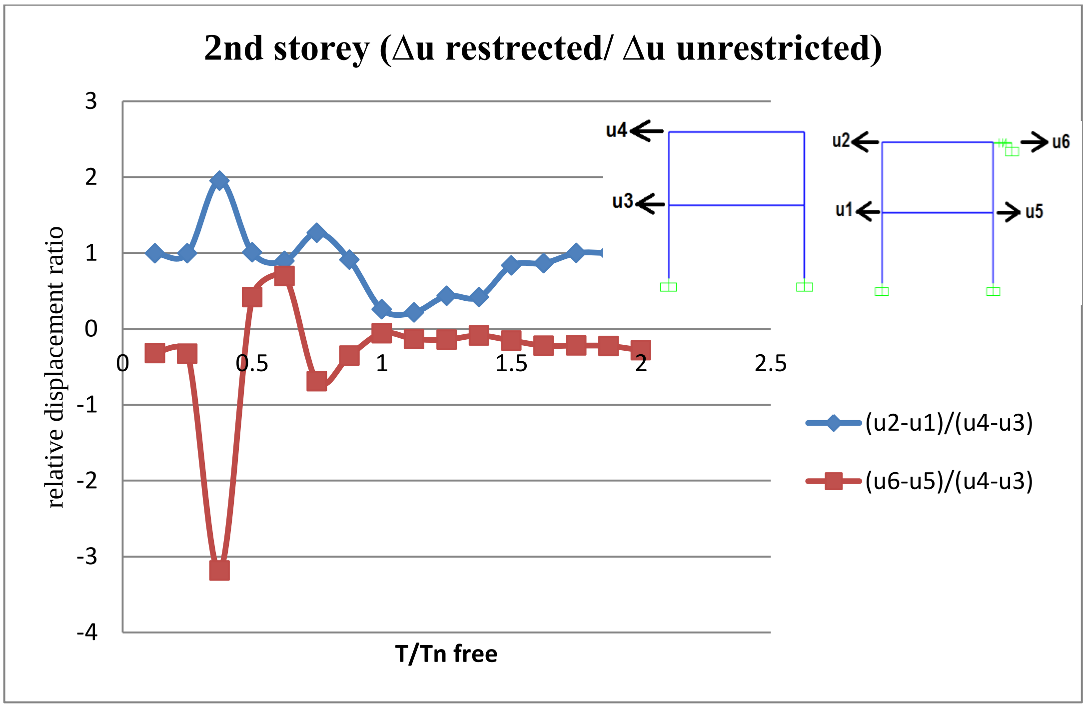


Figure (7. 5): 2nd storey Relative displacement ratio.

7.4 Three degrees of freedom model

7.4.1 The link at 1st storey

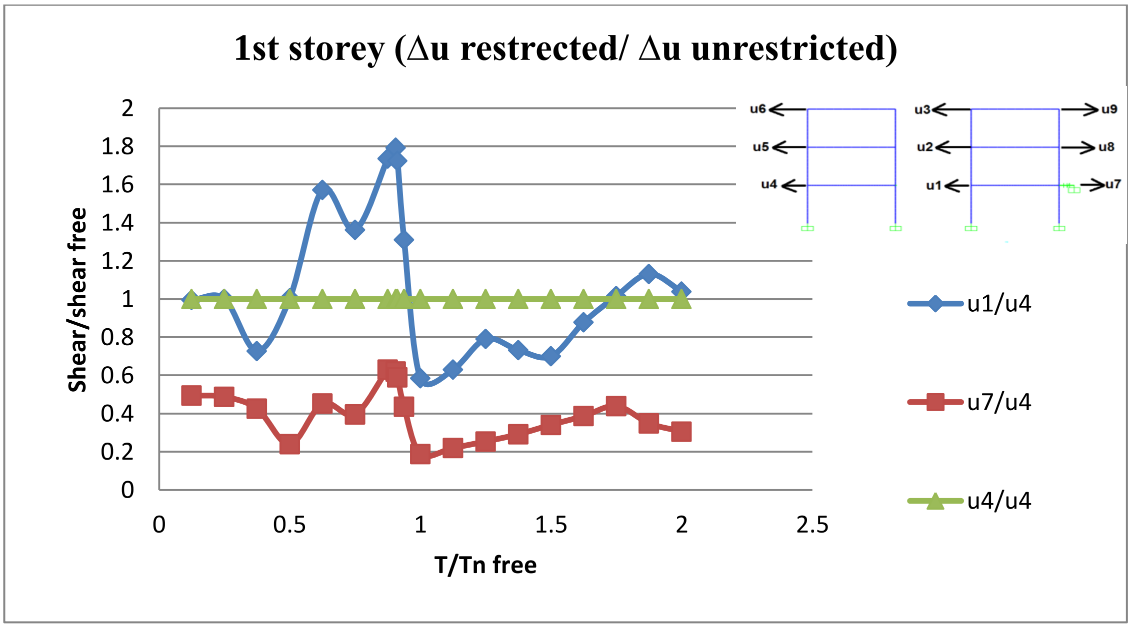


Figure (7. 6): 1st storey Relative displacement ratio.

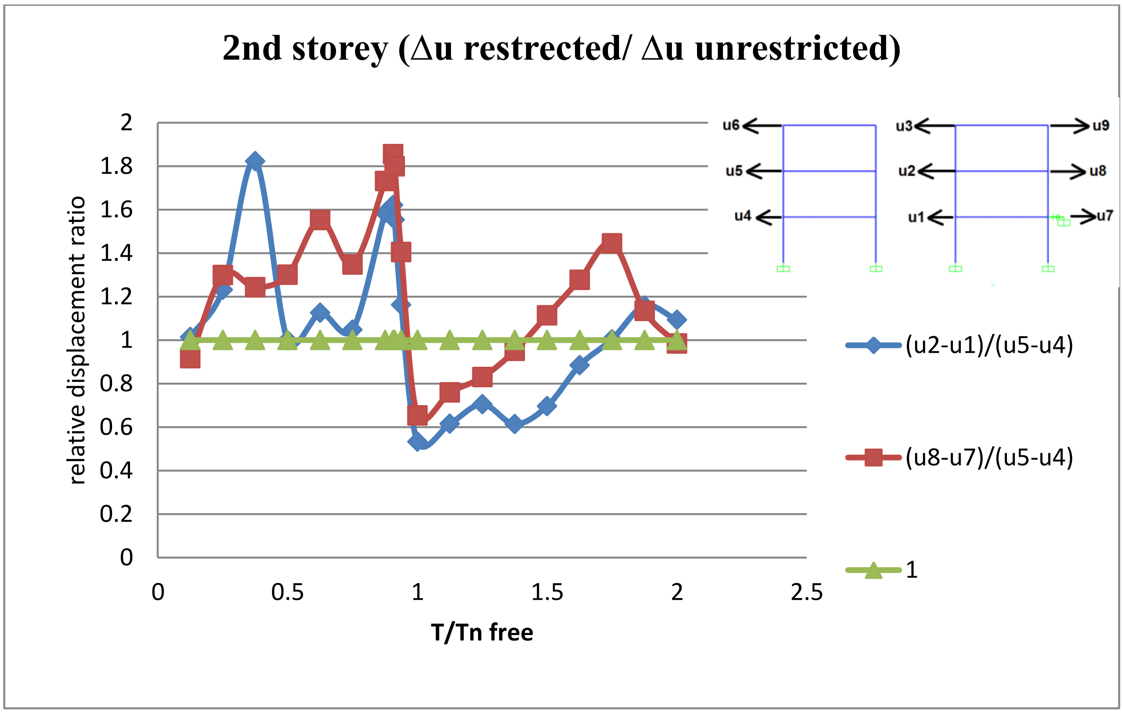


Figure (7. 7): 2nd storey Relative displacement ratio.

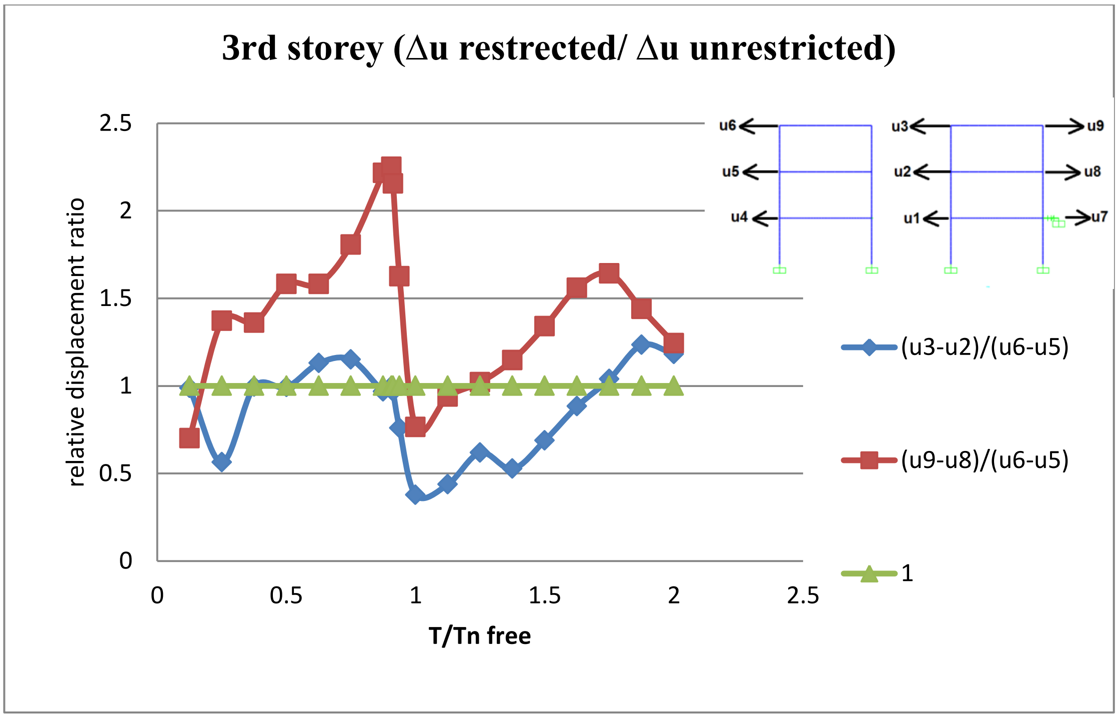


Figure (7. 8): 3rd storey Relative displacement ratio.

7.4.2 The link at 2nd storey

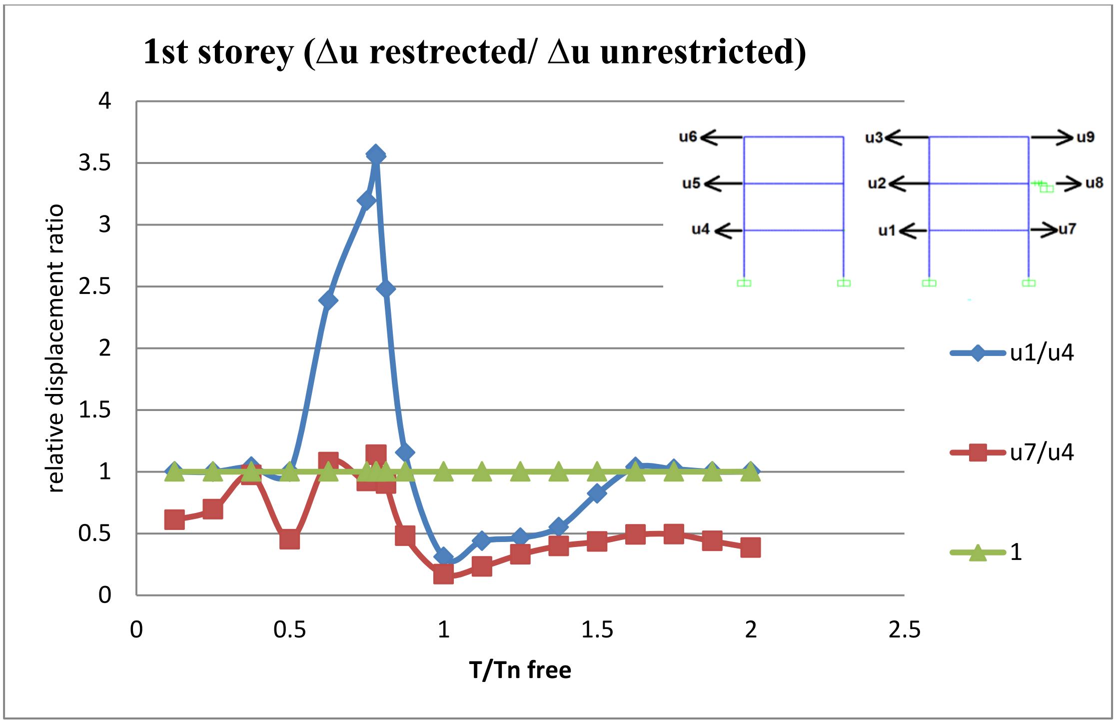


Figure (7. 9): 1st storey Relative displacement ratio.

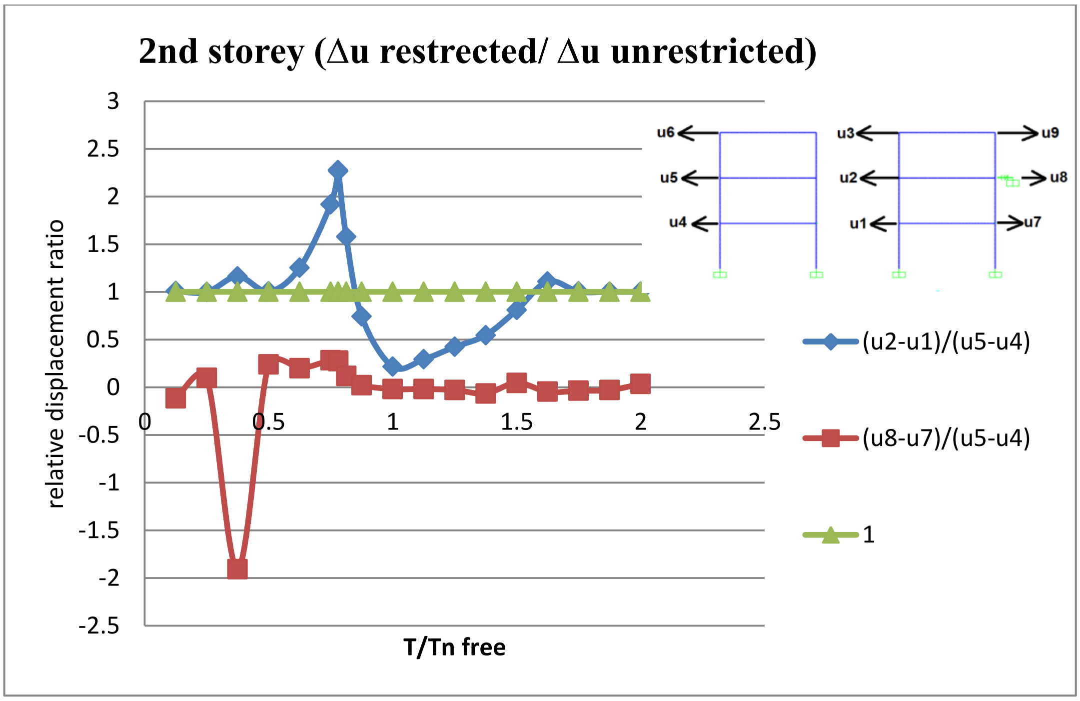


Figure (7. 10): 2nd storey Relative displacement ratio.

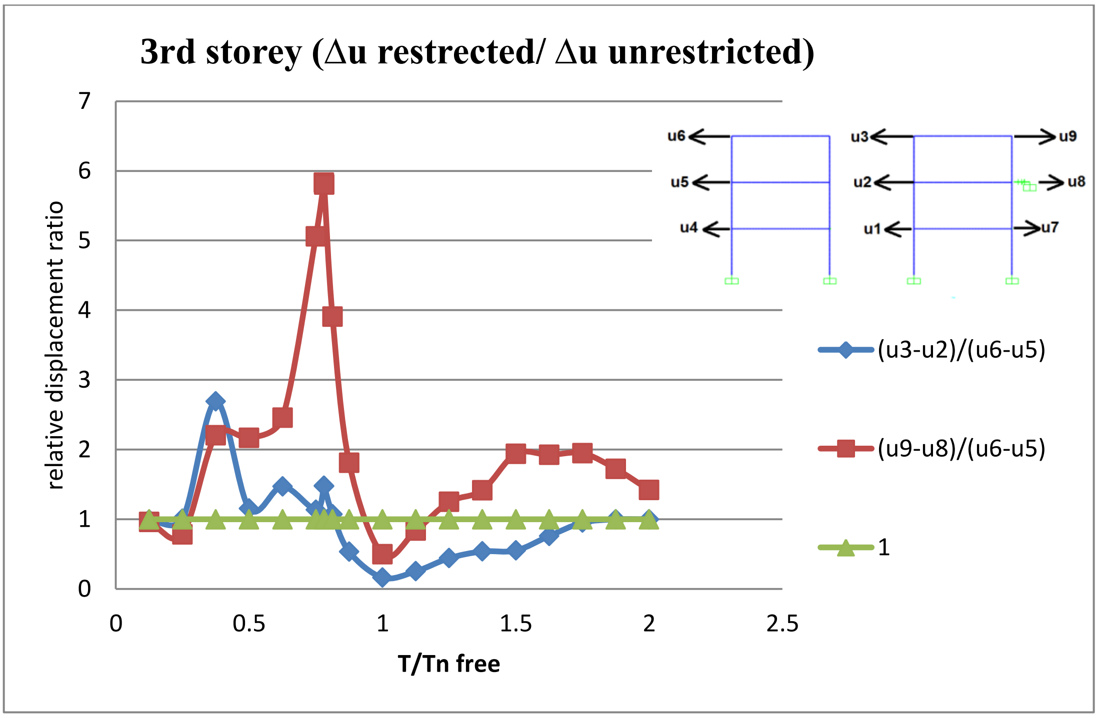


Figure (7. 11): 3rd storey Relative displacement ratio.

7.4.3 The link at 3rd storey

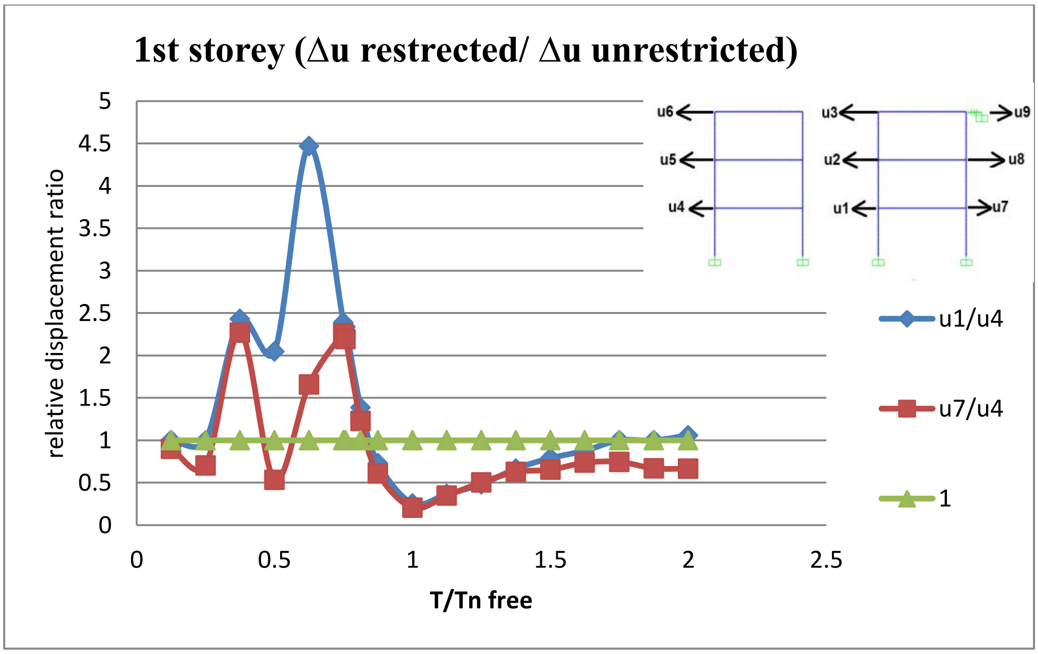


Figure (7. 12): 1st storey Relative displacement ratio.

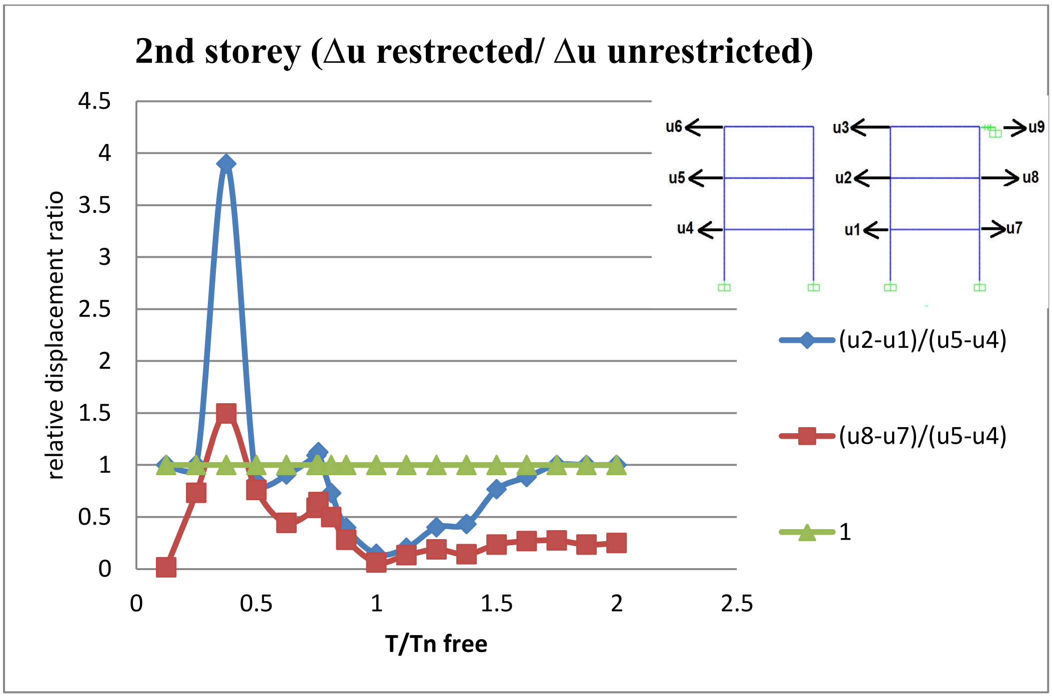


Figure (7. 13): 2nd storey Relative displacement ratio.

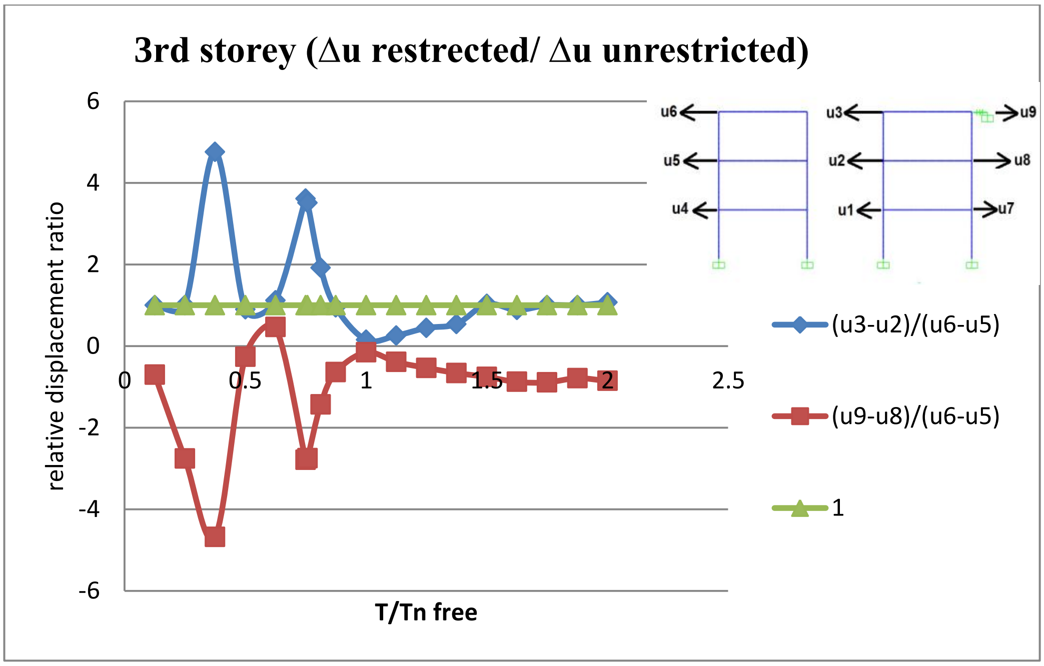


Figure (7. 14): 3rd storey Relative displacement ratio.

7.5 Commentaries

The above graphs show that when considering the natural period of unrestricted structure in analysis without take into consideration the effect of link: the results of displacements for the real structure which have a link may have greater values depending on excitation period.

That's means for any T/T_n the relative displacement ratio greater than one, the building is vulnerable to face much more shear than the unrestricted case, which are in some cases greater than any factor of safety could be taken in the design.

Chapter Eight
Sensitivity Study

8.1 General

Sensitivity analysis is the study to measure the impacts of fluctuations in parameters of a mathematical model or system on the outputs or performance of the system.

To this aim, one of the system parameters is changed by a certain percentage assuming all of the other parameters constant, the model is run and the percentage change of the pre-specified performance indicator is observed.

Sensitivity analysis can be applied to explore the robustness and accuracy of the model results under uncertain conditions, and to comprehend the relationships between input parameters and performance indicators of a system or model, by revealing the unexpected relationships. Monitoring the impacts of variations in model parameters is useful in terms of the identification of the inputs that cause significant uncertainty in the performance indicators. Therefore, these significant parameters can be focused to reduce the uncertainty and increase the robustness and reliability of the system.

In this chapter, two main analysis parameters will be checked to ensure the overall results: the minimum number of modes taken into consideration due the analysis and the sensitivity of structure response to excitation period intervals.

8.2 Minimum numbers of modes

For this analysis, a three degrees of freedom model to be analyzed using the first mode only, and compare the results of the same frame using two, three and four modes of vibration.

8.2.1 One mode analysis

OutputCase	StepType Text	StepNum Unitless	Period Sec	UX Unitless
MODALritz	Mode	1	0.795682	0.91948

Figure (8. 1): modal mass participation ratio.

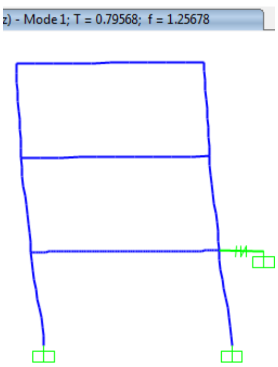


Figure (8. 2): mode shapes and periods.

Table (8. 1): one mode analysis results.

			1mode					
Tn	T	T/Tn	u1	u2	u3	u7	u8	u9
0.8	0.1	0.125	0.01167	0.0204	0.0253	0.00706	0.0123	0.0153
0.8	0.2	0.25	0.02666	0.04666	0.0578	0.01475	0.0258	0.032
0.8	0.3	0.375	0.04931	0.08618	0.1068	0.03884	0.0679	0.0841
0.8	0.4	0.5	0.09711	0.1697	0.2103	0.05323	0.093	0.1153
0.8	0.5	0.625	0.1803	0.3152	0.3906	0.1082	0.1891	0.2343
0.8	0.6	0.75	0.4586	0.8015	0.9933	0.2878	0.5029	0.6233
0.8	0.7	0.875	0.5352	0.9355	1.159	0.3459	0.6046	0.7492
0.8	0.725	0.9063	0.4342	0.7589	0.9404	0.2858	0.4996	0.6191
0.8	0.73	0.9125	0.4218	0.7372	0.9135	0.2775	0.4815	0.5967
0.8	0.75	0.9375	0.3671	0.6416	0.7951	0.2461	0.4302	0.5331
0.8	0.8	1	0.2899	0.5067	0.6279	0.1973	0.3449	0.4274
0.8	0.9	1.125	0.2219	0.3879	0.4806	0.1395	0.2439	0.3022
0.8	1	1.25	0.1605	0.2805	0.3477	0.1339	0.234	0.29

8.2.2 Two modes analysis

OutputCase	StepType Text	StepNum Unitless	Period Sec	UX Unitless
MODALritz	Mode	1	0.799412	0.90154
MODALritz	Mode	2	0.24708	0.09816

Figure (8. 3): modal mass participation ratio.

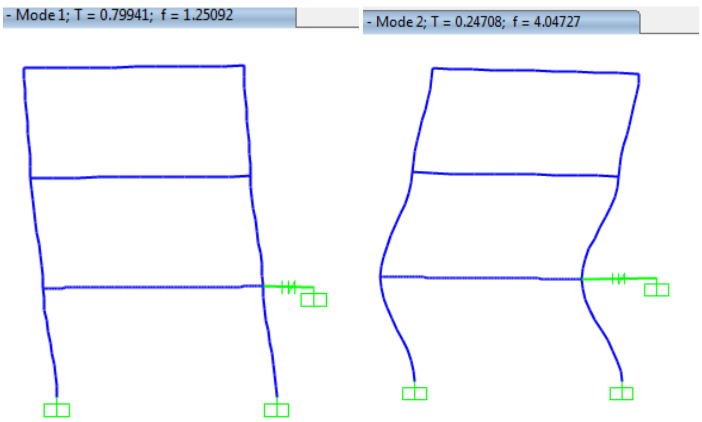


Figure (8. 4): mode shapes and periods.

Table (8. 2): Two modes analysis results.

Tn	T	T/Tn	u1	u2	u3	u7	u8	u9
0.8	0.1	0.125	0.01216	0.02011	0.02623	0.006837	0.01266	0.01928
0.8	0.2	0.25	0.08039	0.05567	0.05925	0.04255	0.01856	0.03624
0.8	0.3	0.375	0.04579	0.08484	0.1122	0.0229	0.05029	0.07409
0.8	0.4	0.5	0.05948	0.1228	0.1616	0.01985	0.1015	0.1507
0.8	0.5	0.625	0.1165	0.2311	0.2964	0.03642	0.1769	0.2602
0.8	0.6	0.75	0.2638	0.4423	0.5434	0.08221	0.3443	0.4922
0.8	0.7	0.875	0.6714	1.128	1.384	0.1953	0.8825	1.27
0.8	0.725	0.90625	0.8197	1.366	1.675	0.2426	1.067	1.54
0.8	0.73	0.9125	0.8123	1.351	1.654	0.242	1.054	1.523
0.8	0.75	0.9375	0.6995	1.15	1.406	0.2174	0.8968	1.299
0.8	0.8	1	0.4804	0.7622	0.9323	0.167	0.5996	0.8713
0.8	0.9	1.125	0.2799	0.485	0.6168	0.1254	0.4051	0.5667
0.8	1	1.25	0.22	0.4002	0.5067	0.08863	0.299	0.4169

8.2.3 Three modes analysis

OutputCase	StepType Text	StepNum Unitless	Period Sec	UX Unitless
MODALritz	Mode	1	0.800332	0.89686
MODALritz	Mode	2	0.278259	0.0981
MODALritz	Mode	3	0.119949	0.00474

Figure (8. 5): modal mass participation ratio.

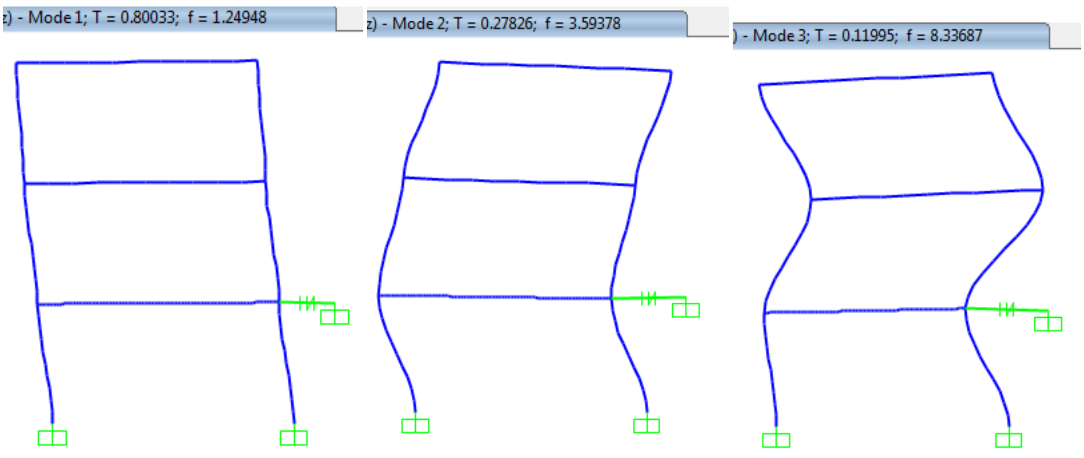


Figure (8. 6): mode shapes and periods.

Table (8. 3): Three modes analysis results.

Tn	T	T/Tn	u1	u2	u3	u7	u8	u9
0.8	0.1	0.125	0.01247	0.01908	0.02669	0.006694	0.01249	0.01788
0.8	0.2	0.25	0.03553	0.04938	0.05812	0.02042	0.03159	0.04503
0.8	0.3	0.375	0.04758	0.08397	0.113	0.02663	0.05739	0.08058
0.8	0.4	0.5	0.06284	0.12	0.1636	0.01908	0.09071	0.1589
0.8	0.5	0.625	0.1168	0.2237	0.2865	0.03382	0.1647	0.2706
0.8	0.6	0.75	0.2326	0.4376	0.5556	0.0621	0.3136	0.5082
0.8	0.7	0.875	0.6094	1.077	1.379	0.1825	0.8	1.257
0.8	0.725	0.90625	0.772	1.355	1.714	0.2327	1.011	1.57
0.8	0.73	0.9125	0.7776	1.36	1.713	0.2331	1.018	1.573
0.8	0.75	0.9375	0.6922	1.197	1.488	0.2024	0.9018	1.378
0.8	0.8	1	0.4695	0.8102	0.9987	0.1365	0.6126	0.9324
0.8	0.9	1.125	0.2883	0.5016	0.614	0.0919	0.3943	0.6008
0.8	1	1.25	0.239	0.4027	0.4909	0.07562	0.2883	0.4284

8.2.4 Four modes analysis

OutputCase	StepType Text	StepNum Unitless	Period Sec	UX Unitless
MODALritz	Mode	1	0.800332	0.89686
MODALritz	Mode	2	0.278614	0.09754
MODALritz	Mode	3	0.137367	0.00385
MODALritz	Mode	4	0.110973	0.00163

Figure (8. 7): modal mass participation ratio.

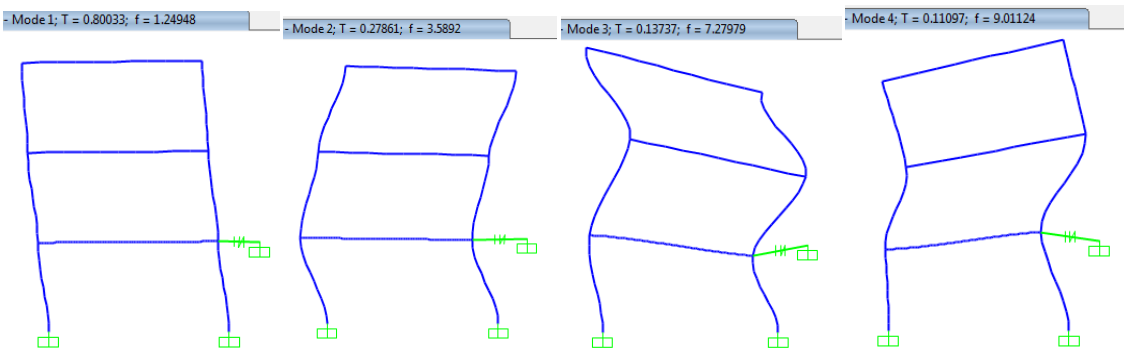


Figure (8. 8): mode shapes and periods.

Table (8. 4): Four modes analysis results.

Tn	T	T/Tn	u1	u2	u3	u7	u8	u9
0.8	0.1	0.125	0.01125	0.01939	0.02656	0.005595	0.01295	0.01804
0.8	0.2	0.25	0.02994	0.05293	0.0577	0.01458	0.03883	0.05043
0.8	0.3	0.375	0.04702	0.08604	0.1114	0.02749	0.0541	0.08878
0.8	0.4	0.5	0.06226	0.1209	0.1633	0.01483	0.09109	0.1586
0.8	0.5	0.625	0.1203	0.2175	0.2893	0.03462	0.1687	0.2693
0.8	0.6	0.75	0.2406	0.4226	0.5517	0.06993	0.3039	0.5065
0.8	0.7	0.875	0.5759	1.073	1.266	0.2089	0.7523	1.194
0.8	0.725	0.90625	0.7285	1.344	1.583	0.252	0.9562	1.496
0.8	0.73	0.9125	0.7366	1.354	1.597	0.2522	0.9674	1.509
0.8	0.75	0.9375	0.6721	1.221	1.446	0.2231	0.8868	1.369
0.8	0.8	1	0.4537	0.8264	0.9899	0.1458	0.6034	0.9352
0.8	0.9	1.125	0.276	0.5107	0.6126	0.09646	0.3864	0.6051
0.8	1	1.25	0.2307	0.406	0.4988	0.07407	0.2802	0.4332

8.2.5 Presentation of results

The following graphs shows the result of displacement of each floor as per number of modes taken into account in analysis

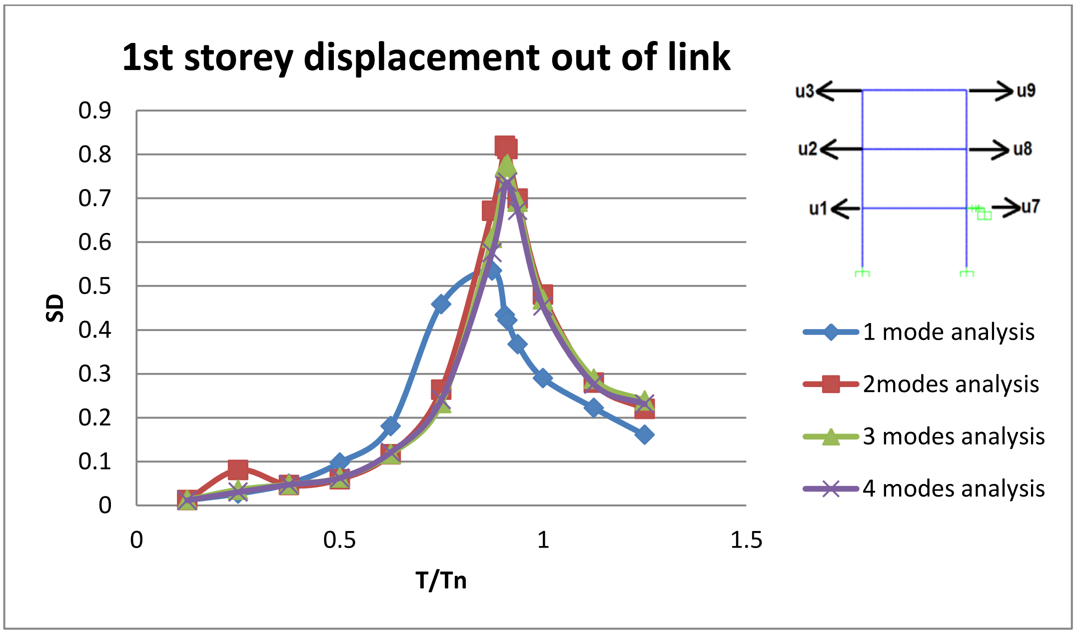


Figure (8. 9): 1st storey displacement out of link for the 4 models.

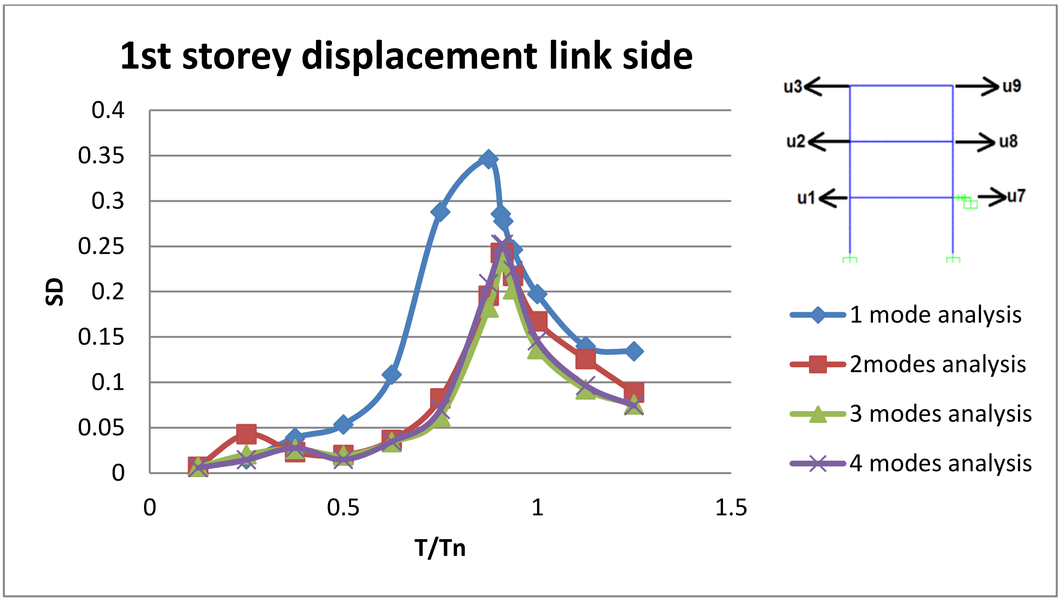


Figure (8. 10): 1st storey displacement in link side for the 4 models.

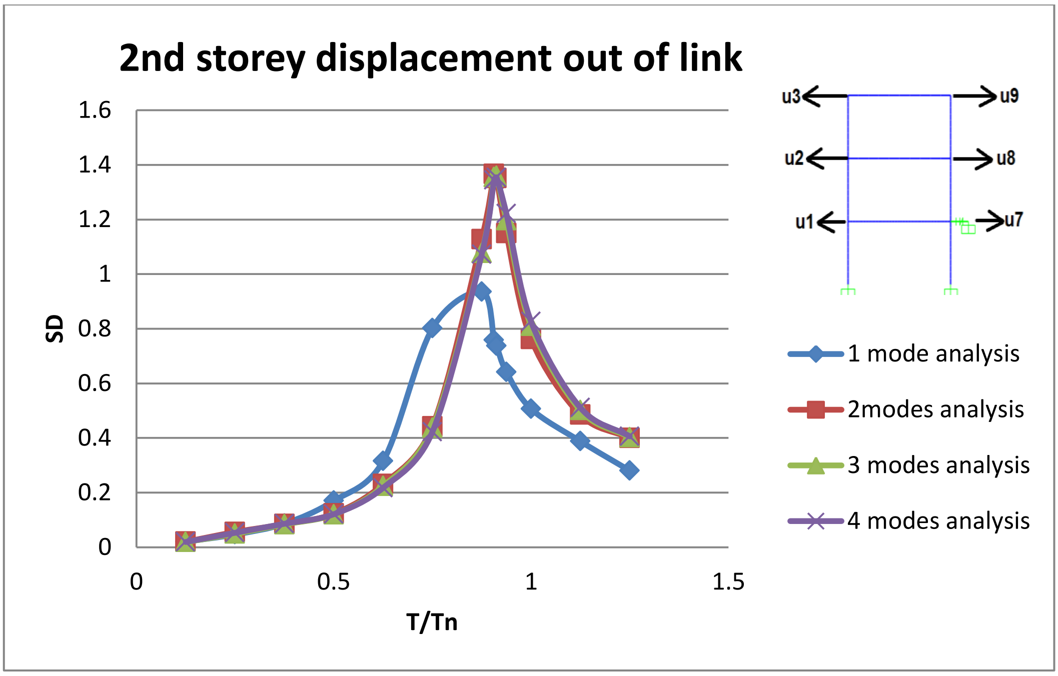


Figure (8. 11): 2nd storey displacement out of link for the 4 models.

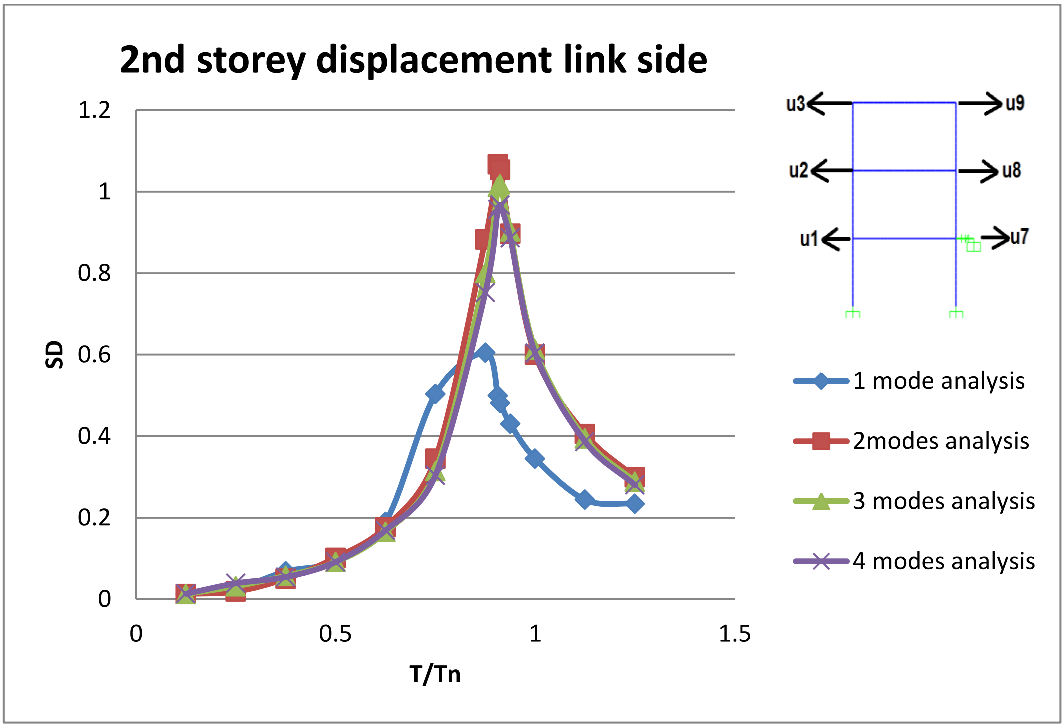


Figure (8. 12): 2nd storey displacement in link side for the 4 models.

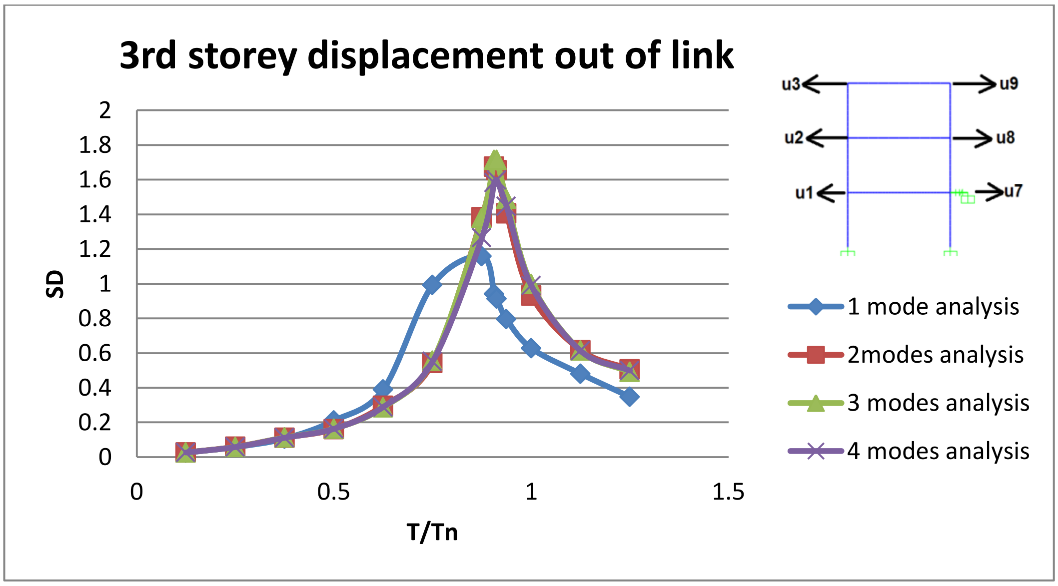


Figure (8. 13): 3rd storey displacement out of link for the 4 models.

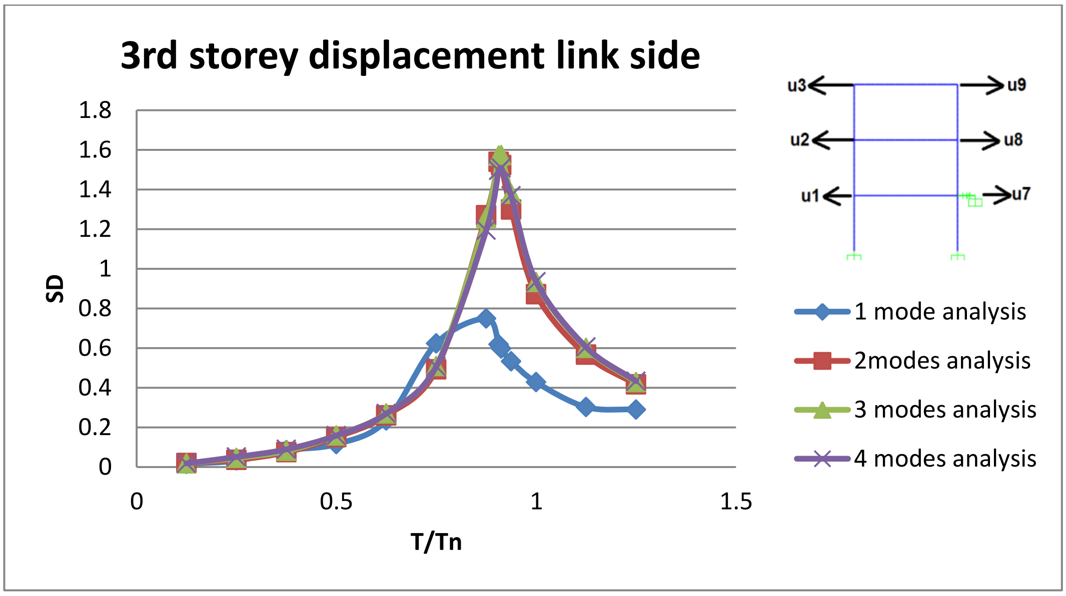


Figure (8. 14): 3rd storey displacement in link side for the 4 models.

8.2.6 Commentaries

The above graphs show that the results of one mode analysis is misleading even for the single degree of freedom structure, the participation of other modes is not major but the link behavior doesn't make sense in one mode analysis, so it's convenient to take the number of modes as the degrees of freedom and number of links.

8.3 Sensitivity of structural response to excitation period intervals size

For this analysis, five models with different link stiffness's values will be analyzed using excitation intervals of 0.1 sec, showing the results and make enhancement where it needed.

8.3.1 Excitation intervals of 0.1 sec

The following table shows the result of frequency response analysis of the four models by applying sine wave excitation with 0.1 sec intervals.

Table (8. 5): frequency response results with excitation intervals =0.1 sec.

			Kl	0	30891	308910	617820
Tn	ω	T	T/Tn	kl/k=0	kl/k=1	kl/k=10	kl/k=20
1	6.283499	0.1	0.1	0.02516	0.02516	0.02516	0.02856
1	6.283499	0.2	0.2	0.05116	0.05116	0.09979	0.2288
1	6.283499	0.3	0.3	0.09938	0.09938	0.1973	0.4676
1	6.283499	0.4	0.4	0.1507	0.1507	0.2625	0.2872
1	6.283499	0.5	0.5	0.1997	0.241	0.4523	0.3887
1	6.283499	0.6	0.6	0.3205	0.4003	1.173	1.904
1	6.283499	0.7	0.7	0.4606	0.7004	1.361	0.7733
1	6.283499	0.8	0.8	0.7252	1.544	0.7145	0.5329
1	6.283499	0.9	0.9	1.245	1.724	0.5398	0.4187
1	6.283499	1	1	2.362	0.9976	0.4734	0.4191
1	6.283499	1.1	1.1	1.501	0.7342	0.4122	0.3754
1	6.283499	1.2	1.2	1.047	0.6125	0.3873	0.4719
1	6.283499	1.3	1.3	0.8191	0.4735	0.6135	0.5128
1	6.283499	1.4	1.4	0.7053	0.4017	0.5363	0.4366
1	6.283499	1.5	1.5	0.6125	0.4052	0.4646	0.418
1	6.283499	1.6	1.6	0.5704	0.4068	0.4068	0.4068
1	6.283499	1.7	1.7	0.5247	0.5149	0.4069	0.4069
1	6.283499	1.8	1.8	0.477	0.4823	0.4057	0.46
1	6.283499	1.9	1.9	0.429	0.4403	0.459	0.4035
1	6.283499	2	2	0.4005	0.4005	0.4218	0.4005
1	6.283499	2.1	2.1	0.3968	0.3968	0.4085	0.3968
1	6.283499	2.2	2.2	0.3926	0.3926	0.3926	0.4028
1	6.283499	2.3	2.3	0.388	0.388	0.388	0.388

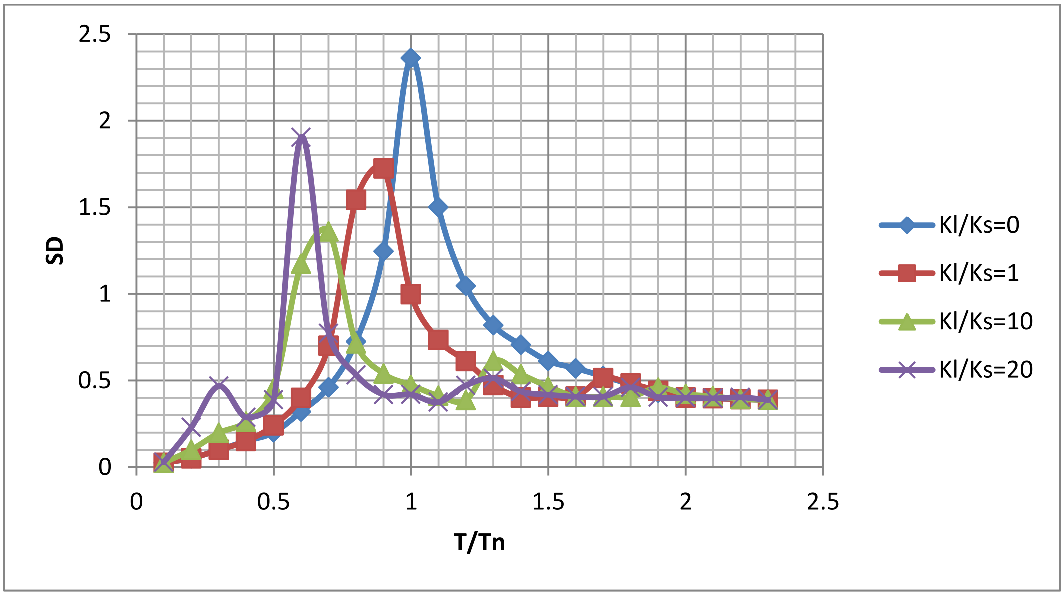


Figure (8. 15): frequency response curves with excitation intervals =0.1 sec.

It's clear as shown in the figure above that the results don't make sense where the in-between values of link stiffness have a lower maximum response than the smallest value, so a suggestion of taking sub intervals where maximum response expected.

8.3.2 Taking sub intervals where response magnified

Table (8. 6): frequency response results taking smaller intervals.

			Kl	0	30891	308910	617820
Tn	ω	T	T/Tn	kl/k=0	kl/k=1	kl/k=10	kl/k=20
1	6.283499	0.4	0.4	0.1507	0.1507	0.2625	0.2872
1	6.283499	0.5	0.5	0.1997	0.241	0.4523	0.3887
1	6.283499	0.6	0.6	0.3205	0.4003	1.173	1.904
1	**6.283499**	**0.65**	**0.65**	**0.3366**	**0.5218**	**2.214**	**1.353**
1	6.283499	0.7	0.7	0.4606	0.7004	1.361	0.7733
1	6.283499	0.8	0.8	0.7252	1.544	0.7145	0.5329
1	**6.283499**	**0.85**	**0.85**	**0.9306**	**2.344**	**0.5824**	**0.4928**
1	6.283499	0.9	0.9	1.245	1.724	0.5398	0.4187
1	6.283499	1	1	2.362	0.9976	0.4734	0.4191
1	6.283499	1.1	1.1	1.501	0.7342	0.4122	0.3754
1	6.283499	1.2	1.2	1.047	0.6125	0.3873	0.4719

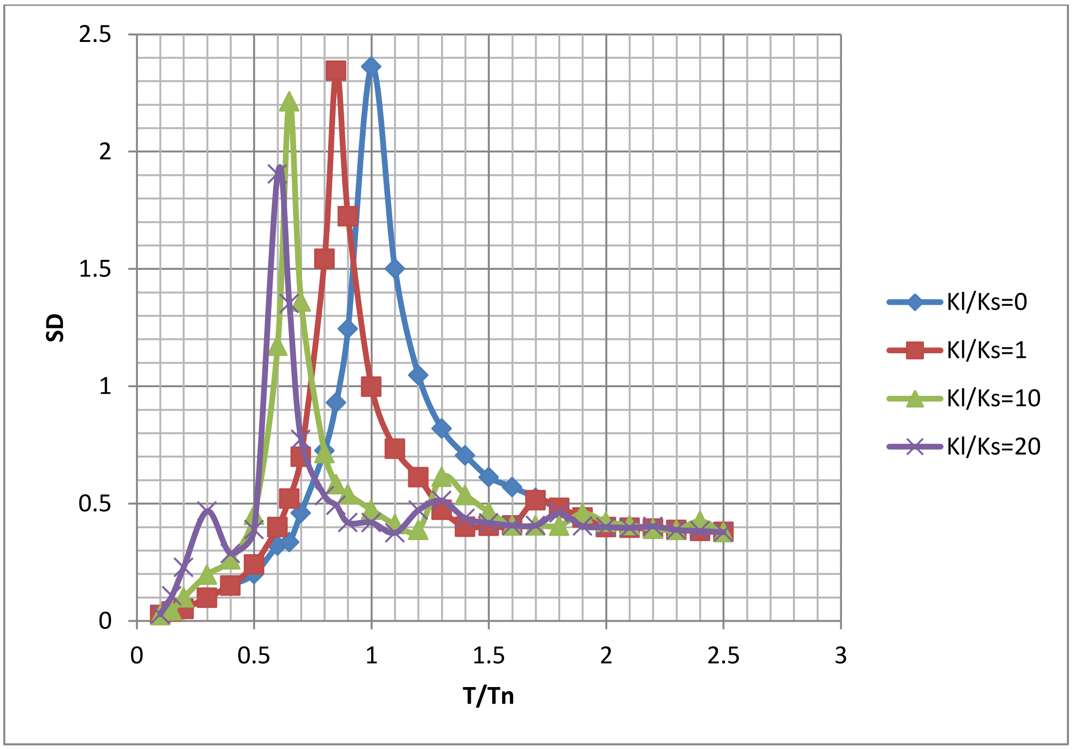


Figure (8. 16): frequency response curves taking smaller intervals.

The figure above shows the frequency response for the four models when taking new excitation periods into account in analysis.

8.3.3 Commentaries

The above graphs show that the maximum response is so sensitive to the period of excitation, so for frequency response analysis the intervals should be divided where the maximum response expected.

Chapter Nine

Parametric Study

9.1 General

Parametric study is a process based on algorithmic thinking that enables the expression of parameters and rules that, together, define, encode and clarify the relationship between design intent and design response

As moving forward in the design, one can assess the impact that changing certain parameters can have on the design. The parameters can include dimensional parameters. Parametric studies allow you to nominate parameters for evaluation, define the parameter range, specify the design constraints, and analyze the results of each parameter variation.

A parametric study requires the following:

- Design Objective is set to Parametric Dimensions
- Parameter ranges identified
- Various configurations generated

Once you determine that a configuration satisfies your design needs, you are able to promote that configuration back to the model. You are prompted whether to make changes.

For this study, we deal with simple frames which don't contain any additions except the link, so the effect of link stiffness will be analyzed in this chapter.

9.2 Effect of link-structure stiffness ratio to displacement

For the purpose of this analysis a SDOF frame of free natural period =1.0 sec and damping ratio 5% compared to the same models of frames connected to links with different stiffness values, as shown in table (8.2) below.

Table (9. 1): models parameters.

Tn	1.0	1.0	1.0	1.0
damping	0.05	0.05	0.05	0.05
ω	6.283499	6.283499	6.283499	6.283601
Klink	30891	308910	617820	3089100
Tn2	0.707071	0.301496	0.218207	0.099499
tnavg	0.853511	0.650723	0.609079	0.549716

Table (9. 2): analysis results for top displacements.

		Kl	0	30891	308910	3089100
Tn	ω	T/Tn	kl/k=0	kl/k=1	kl/k=10	kl/k=100
1	6.283499	0.1	0.02516	0.02516	0.02516	0.04019
1	6.283499	0.15	0.04053	0.04053	0.04448	0.08676
1	6.283499	0.2	0.05116	0.05116	0.09979	0.1086
1	6.283499	0.3	0.09938	0.09938	0.1973	0.2983
1	6.283499	0.4	0.1507	0.1507	0.2625	0.2638
1	6.283499	0.57	0.1997	0.241	0.4523	1.564
1	6.283499	0.6	0.3205	0.4003	1.173	1.102
1	6.283499	0.65	0.3366	0.5218	2.214	0.5374
1	6.283499	0.7	0.4606	0.7004	1.361	0.4182
1	6.283499	0.8	0.7252	1.544	0.7145	0.4139
1	6.283499	0.85	0.9306	2.344	0.5824	0.4053
1	6.283499	0.9	1.245	1.724	0.5398	0.4325
1	6.283499	1	2.362	0.9976	0.4734	0.5035
1	6.283499	1.1	1.501	0.7342	0.4122	0.4292
1	6.283499	1.2	1.047	0.6125	0.3873	0.4162

1	6.283499	1.3	0.8191	0.4735	0.6135	0.4052
1	6.283499	1.4	0.7053	0.4017	0.5363	0.4068
1	6.283499	1.5	0.6125	0.4052	0.4646	0.4404
1	6.283499	1.6	0.5704	0.4068	0.4068	0.4057
1	6.283499	1.7	0.5247	0.5149	0.4069	0.4035
1	6.283499	1.8	0.477	0.4823	0.4057	0.4005
1	6.283499	1.9	0.429	0.4403	0.459	0.4326
1	6.283499	2	0.4005	0.4005	0.4218	0.401
1	6.283499	2.1	0.3968	0.3968	0.4085	0.388
1	6.283499	2.2	0.3926	0.3926	0.3926	0.3831
1	6.283499	2.3	0.388	0.388	0.388	0.4094
1	6.283499	2.4	0.3831	0.3831	0.4253	
1	6.283499	2.5	0.3794	0.3794	0.378	

Fig (9.1) below shows the frequency response curves for different stiffness ratios between link and structure, it's clear that with larger stiffness ratio the period of structure decreases.

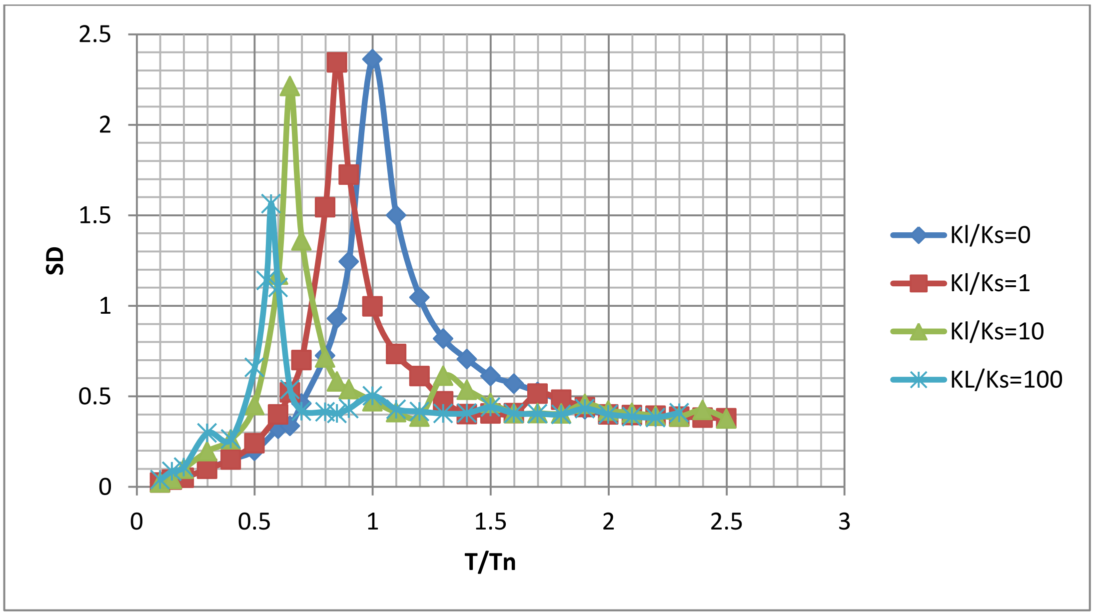


Figure (9. 1): frequency-response curves for different K/Kl ratios.

This graph shows that unrestricted structure "K_{link} / $K_{structure}$ =0" has its maximum displacement exactly at excitation period equals the natural period "T/T_n =1" which matches resonance theory perfectly.

For restricted models, the structure resonates at lower period between free natural period and combined link-structure period, and it is exactly at the average of both.

To ensure these results, seven linked models with different free natural periods were analyzed and the results of drift at the free natural period, combined period and the average of both plotted in fig (9.2).

Table (9. 3): top displacement results for linked models.

	T	SD free	SD Linked
tn1L	0.168	0.03449	0.05388
tn1avg	0.43	0.1293	0.778
tn1	0.7	1.103	0.2234
tn2L	0.19	0.04426	0.06554
tn2avg	0.49	0.2131	0.971
tn2	0.8	1.427	0.2953
tn3l	0.21	0.05437	0.08983
tn3avg	0.558	0.2776	1.382
tn3	0.9	1.79	0.378
tn4l	0.243	0.07203	0.1124
tn4avg	0.62	0.3411	1.569
tn4	1	2.094	0.48
tn5l	0.26	0.08299	0.1278
tn5avg	0.68	0.4111	1.964
tn5	1.1	2.644	0.571
tn6l	0.28	0.09664	0.1609
tn6avg	0.74	0.486	2.268
tn6	1.2	3.137	0.6816
tn7l	0.31	0.1176	0.1734
tn7avg	0.8	0.5671	2.582
tn7	1.3	3.663	0.8033

The results show that the maximum response of restricted structures occurs at an excitation period equals the average of combined link -structure period and the free natural period.

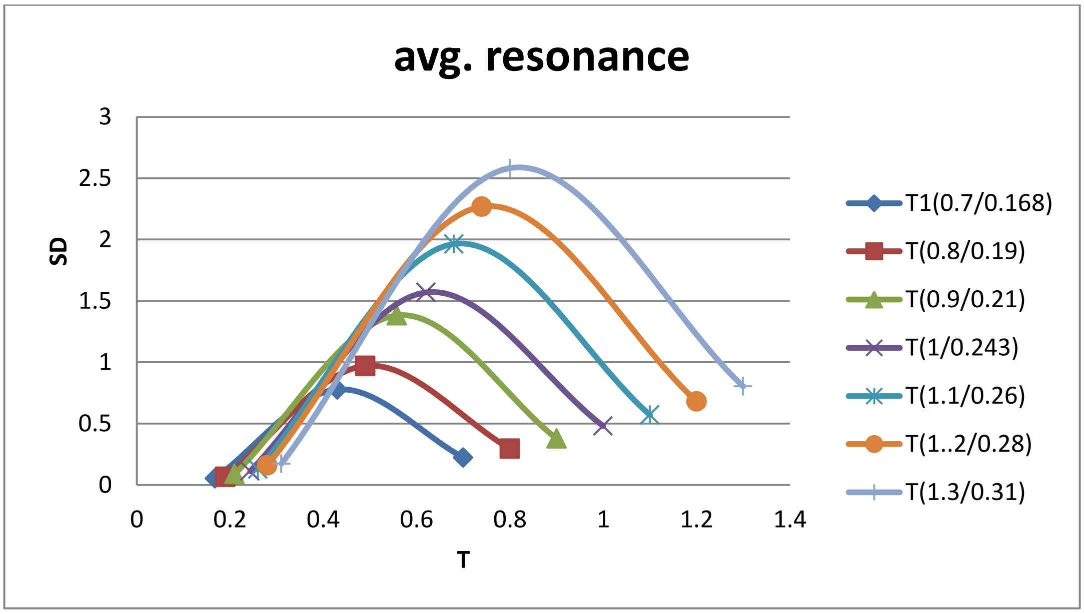


Figure (9. 2): maximum response for different models shows avg. peak response.

9.3 Effect of link-structure stiffness ratio to relative displacement

For the purpose of this analysis a 2DOF frame of free natural period =0.8 sec and damping ratio 5%, compared to the same models of frames connected to links at 1st storey with different stiffness values, as shown in table (8.4) below.

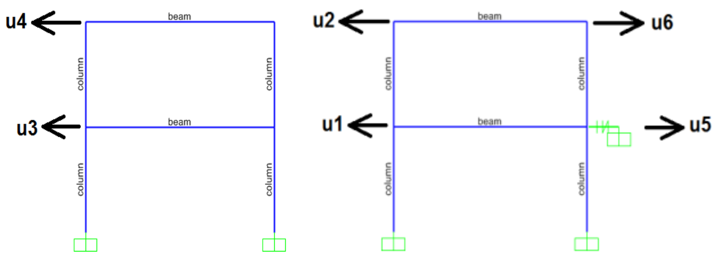


Figure (9. 3): Two degrees of freedoms model.

Table (9. 4): displacement results for different link stiffness values.

	unrestricted		Kl=30000		Kl=100000		kl=500000		Kl/Ks=1000000		Kl/Ks=10000000	
T/Tn	**u3**	**u4**	**u5**	**u6**	**u5**	**u6**	**u5**	**u6**	**u5**	**u6**	**u5**	**u6**
0.125	0.01442	0.02493	0.0128	0.02035	0.0117	0.0187	0.0075	0.01628	0.0051	0.0155	0.0021	0.0133
0.25	0.03819	0.05472	0.0288	0.04498	0.0243	0.0394	0.0287	0.04976	0.0178	0.0402	0.0042	0.0394
0.375	0.08316	0.1052	0.0815	0.08045	0.0768	0.0845	0.0499	0.07315	0.0345	0.0579	0.0055	0.0384
0.5	0.08873	0.1542	0.0565	0.151	0.0496	0.1489	0.0346	0.1407	0.0253	0.1351	0.0048	0.1211
0.625	0.1104	0.2081	0.1344	0.2684	0.1125	0.2614	0.0697	0.2384	0.0579	0.2249	0.0175	0.1965
0.75	0.2468	0.4388	0.222	0.4306	0.1982	0.4629	0.1097	0.4821	0.0923	0.4955	0.0256	0.4568
0.85	0.3909	0.6774	0.3897	0.7346	0.3592	0.8207	0.2465	1.102	0.2014	1.15	0.0317	0.7934
0.8563	0.4093	0.708	0.4056	0.7626	0.378	0.8598	0.2619	1.169	0.2034	1.162	0.0292	0.7818
0.875	0.4524	0.7811	0.4506	0.8416	0.4372	0.9843	0.3001	1.323	0.1901	1.088	0.0246	0.7165
0.95	0.7595	1.294	0.7992	1.472	0.7747	1.675	0.2141	0.8673	0.1187	0.6782	0.0159	0.4964
0.9938	1.05	1.777	0.9746	1.774	0.6346	1.347	0.1801	0.6918	0.101	0.5635	0.0157	0.4157
1	1.094	1.85	0.9618	1.748	0.6094	1.291	0.1783	0.6773	0.0985	0.5483	0.0154	0.4105
1.125	0.6288	1.048	0.5096	0.9097	0.3472	0.7141	0.1291	0.4675	0.071	0.3873	0.0135	0.2862
1.25	0.4182	0.6898	0.3618	0.6414	0.2529	0.524	0.0938	0.3704	0.0638	0.333	0.0126	0.2756
1.375	0.3433	0.5628	0.2692	0.4746	0.2098	0.4353	0.0879	0.3488	0.0626	0.3132	0.012	0.2592
1.5	0.2753	0.4484	0.2526	0.4414	0.1963	0.403	0.0815	0.3216	0.0592	0.2889	0.0113	0.2392
1.625	0.2153	0.3477	0.2315	0.4022	0.1794	0.3659	0.0803	0.3064	0.0649	0.2941	0.0123	0.2516
1.75	0.1928	0.3099	0.2079	0.3598	0.1607	0.3265	0.0883	0.3013	0.0541	0.2357	0.0112	0.1943
1.875	0.1891	0.3045	0.1834	0.3165	0.1454	0.2977	0.0605	0.229	0.044	0.2066	0.0084	0.1716
2	0.1878	0.3017	0.159	0.2738	0.1221	0.2478	0.0529	0.1986	0.0387	0.1796	0.0076	0.1493

The results of relative displacements shown in the table and figure below:

Table (9. 5): Relative Displacement Results for different link stiffness values.

KL	0	30000	100000	500000	1000000	10000000
T/Tn	**Δ1**	**Δ2**	**Δ3**	**Δ4**	**Δ5**	**Δ6**
0.125	0.01051	0.00753	0.00706	0.008769	0.010343	0.011196
0.25	0.01653	0.0162	0.01508	0.02109	0.0224	0.035138
0.375	0.02204	-0.00102	0.00768	0.02323	0.02333	0.032904
0.5	0.06547	0.09455	0.09934	0.10613	0.10981	0.11635
0.625	0.0977	0.134	0.1489	0.16874	0.16703	0.17905
0.75	0.192	0.2086	0.2647	0.3724	0.40323	0.43118
0.85	0.2865	0.3449	0.4615	0.8555	0.9486	0.7617
0.85625	0.2987	0.357	0.4818	0.9071	0.9586	0.75262
0.875	0.3287	0.391	0.5471	1.0229	0.8979	0.69192
0.95	0.5345	0.6728	0.9003	0.6532	0.5595	0.48055
0.99375	0.727	0.7994	0.7124	0.5117	0.4625	0.39999
1	0.756	0.7862	0.6816	0.499	0.44984	0.39507
1.125	0.4192	0.4001	0.3669	0.3384	0.31628	0.27274
1.25	0.2716	0.2796	0.2711	0.2766	0.26921	0.26299
1.375	0.2195	0.2054	0.2255	0.26095	0.25065	0.24718
1.5	0.1731	0.1888	0.2067	0.24009	0.2297	0.22792
1.625	0.1324	0.1707	0.1865	0.22613	0.22918	0.23929
1.75	0.1171	0.1519	0.1658	0.21301	0.1816	0.18308
1.875	0.1154	0.1331	0.1523	0.16848	0.16257	0.163222
2	0.1139	0.1148	0.1257	0.14566	0.14093	0.141665

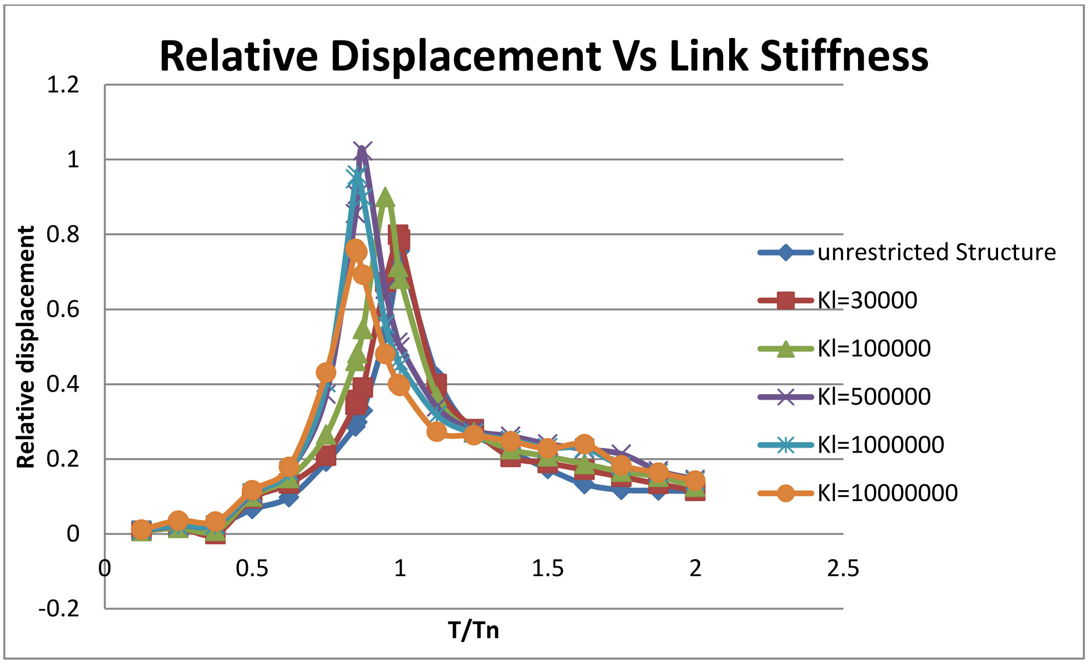


Figure (9. 4): Relative displacement plot for different link stiffness values.

To show the results clearly, a plot of results from T/Tn =0.5 to 1.5 illustrated in the figure below:

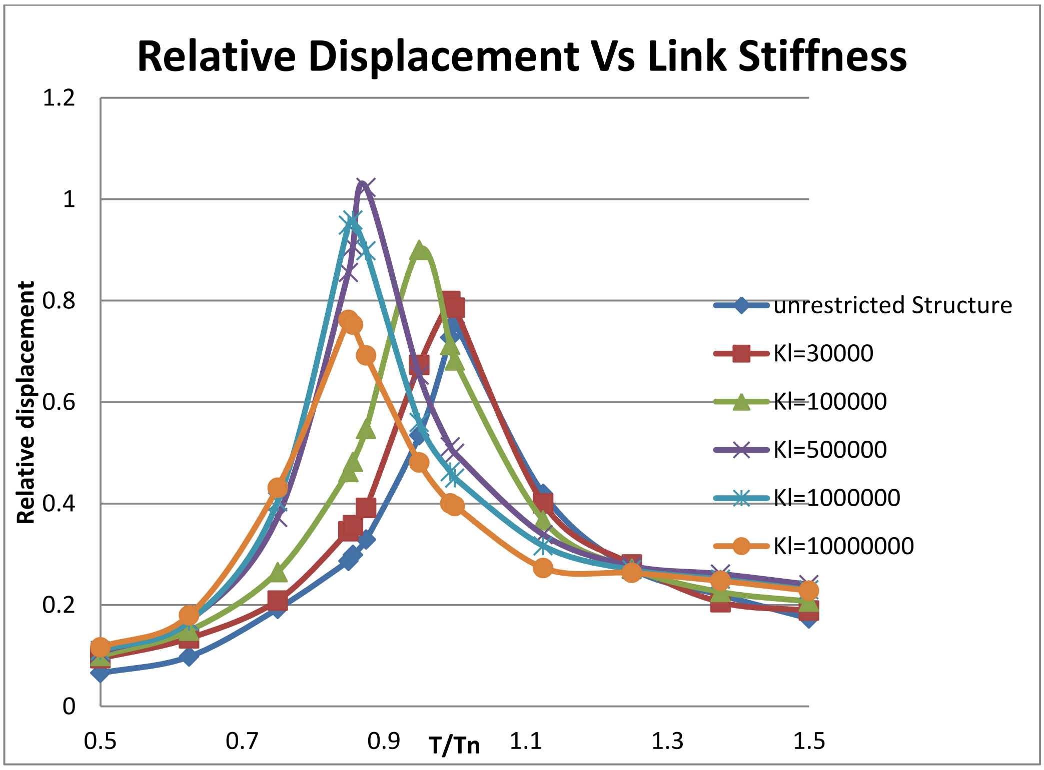


Figure (9. 5): Relative displacement plot for different link stiffness values.

9.4 Commentaries

The figure above shows the effect of link's stiffness value on the 2nd storey's relative displacement, it's clear that structure's relative displacement increases with adding a link of any stiffness, but it has the maximum relative displacement at certain value of link stiffness which coincide with its performance, not the lowest or greatest, exactly as the structure responds to the periods of excitation, and resonate at certain value equals its natural period.

Chapter Ten

Conclusions, Recommendations, and Future Work

10.1 Conclusions

In this study, the effect of links between frame structures and rock ground cut were studied. The modeling process was divided into three levels. Studying the behavior of one, two and three degrees of freedom models. Showing vulnerabilities, effect and ratios of considering or neglecting links in structural model. Then the main findings and results of the study will be summarized.

10.1.1 General Conclusions

The followings are the general conclusions of the research:

1. Making links between structure and ground cut have a major effect on the fundamental period and on the lateral stiffness of the structures. The case of always neglecting these links in the modeling phase is unrealistic design against earthquake load.

2. Links have different configuration depending on its location and type of connection to structure and ground, each configuration has its specific effect to the relative displacement for above and below stories.

3. The storey which has a link attached to ground should suffer much more axial shear in diaphragm system.

4. Considering the unrestricted natural period for computing the response of the structures having restriction in its vibration space is misleading and it could be catastrophic for many types of excitations, considering the random nature of excitation's periods and magnitudes.

10.1.2 Research Findings

As the results were introduced in this study as an expression of relative displacement, where it has a direct relation to the internal forces, the results will be summarized in main categories.

The configurations of all models studied earlier in this research could be rearranged as per link location as follows:

- The link at lower storey and have other stories above:

The above stories could have much more relative displacement when it moves in the direction of the link where it could reach 3 times unrestricted vibration in terms of frequency response at certain excitation period, and it could reach 1.3 times unrestricted vibration at its maximum response.

- the link at middle storey and have other stories below and above:

The above stories could have much more relative displacement when it moves in the direction of the link where it could reach 6 times unrestricted vibration in terms of frequency response at certain excitation period, and it could exceed 1.6 times unrestricted vibration at its maximum response.

The lower stories could have less relative displacement.

- The link at top storey and have other stories below:

The lower stories could have less relative displacement but the storey which have a link could have negative relative displacement, that means the slab-column connection of the storey just below the linked one could have a different shear sign above and below at an instant.

10.2 Recommendations

1. This research present the problem briefly, but any recommendation for solutions even they are clear, they should be tested considering all variables which had been included or excluded from this study.

2. The design of buildings have such configuration should be done twice, the first considering link, and then compare to the results of unrestricted case analysis, taking into consideration the probability of link damage during earthquake load cycles.

2. Probability of earthquake impacts on the Palestinian society will increase in the coming decades. Implies that random urbanization, build on unsuitable sites for construction, incomplete brilliant solutions to adapt with construction sites, prevailing construction styles, etc., Hence, the awareness and preparedness of engineers are an urgent necessity to reduce the loss of human lives and property damage.

3. It is recommended to avoid structural configurations which we haven't the complete vision about their behavior.

4. Designers are more interested in the structural response, whereas building owners only focus on the fiscally related matters. But comparing values goes complicated when become dynamic.

5. Seismic guidelines and provisions shall be stringently applied during the design and construction of building structures. Still, more statutory enforcements are necessary for seismic risk mitigation.

10.3 Future Work

1. The research mainly studied the quantitative effect of links on the 2D frames. It would be beneficial to investigate that effect on 3D buildings.

2. The study could be broadened to include much more variables related to structure, site, and connections to both.

3. Effect of side soil interaction with seismic load when dealing with soft rock.

4. How to deal with horizontal irregularities, vertical irregularities, and what is the participation of links to these irregularities when applying 3D analysis.

5. The effect of impact forces on the diaphragm system considering new slab systems used lately in Palestine.

6. For structures and building engineering firm, the magnification of response could be catastrophic. But it worth to study this effect beneficially in mechanical engineering applications.

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جامعة النجاح الوطنية

كلية الدراسات العليا

أثر الوصلات الانشائية بين المبنى والقطع الصخري على التصميم الزلزالي للمنشأ

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قدمت هذه الأطروحة إستكمالاً لمتطلبات الحصول على درجة الماجستير في هندسة الإنشاءات، بكلية الدراسات العليا، في جامعة النجاح الوطنية، نابلس– فلسطين.

2020

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الملخص

يشكل المهندس الانشائي وعالم الزلازل المفتاح الأساسي لضمان تصميم آمن للمنشآت لأحمال الزلازل، الا أن التطبيق الشائع هو أن يبدأ المهندس المعماري عملية التصميم للمبنى، وعليه يحدد العديد من الأمور التي لها علاقة بتكوين المنشأ والتي لها أثر رئيسي في أداء المنشأ واستجابته لأحمال الزلازل.

يمكن تعريف تكوين المبنى بحجم المنشأ وشكله في الأبعاد الثلاثة وشكل وتوزيع العناصر الانشائية، واتصاله مع المباني المجاورة أو الأرض الجانبية، وطبيعة ومواقع العناصر الانشائية وغير الانشائية التي تؤثر بالعادة على تصرف المنشأ تحت تأثير الاحمال المختلفة.

هذا البحث يشكل اضاءة على هذه المباني التي تحوي بعض هذه التكوينات التي تمنع المبنى من التصرف بحرية تحت تأثير الأحمال الجانبية.

المباني الخاضعة للدراسة تحوي وصلات انشائية مع الأرض المجاورة من جهة واحدة، تمنعها من الحركة باتجاه القطع الصخري فيما تبقى حرة الحركة في الاتجاه الاخر.

نظرا لتعقيد وتفرد كل نموذج من هذه المباني، فإن العديد من العوامل تؤثر على أداء المبنى تحت أحمال الزلازل ولذلك يتوجب دراسة هذه العوامل منفردة.

لتطوير رؤية واضحة لأداء هذه المباني تحت تأثير احمال الزلازل، تمت دراسة منشئات اطارية بسيطة تحوي وصلات تحد من حركتها الجانبية باتجاه واحد وتحليلها تحت تأثير أحمال مختلفة

ومقارنتها بنفس النماذج حرة الحركة، وذلك بواسطة برنامج SAP2000 كأداة رئيسية خلال هذه الدراسة ، بعد تأكيد النتائج بمقارنتها بالمعادلات النظرية للأحمال الموجية المتناسقة ، والتأكد من حسابات التردد الطبيعي للمنشأ بطريقة Rayleigh .

لغرض هذه الدراسة، تم اعتماد طريقتين للقياس تم تطبيقهما على كل الاحتمالات لمكان الوصلات بالنسبة للمنشأ الإطاري بنماذج بدرجة حرية واحدة واثنتين وثلاثة:

- طريقة طيف الاستجابة " response spectrum approach ": وعليه تم تطبيق حمل دوري جيبي متناسق يحوي ترددات من 0.1 ثانية – 2 ثانية بفرق درجات 0.1 ثانية وبنفس السعة على كل النماذج التي تحوي نفس التكوين بترددات طبيعية مختلفة تتراوح بين 0.1 ثانية – 2 ثانية.

وبناء عليه تم قياس الازاحة والازاحة النسبية واظهارها في رسومات بالنسبة لتردد الأحمال.

وعليه يتوضح أن النماذج المحدودة الحركة تبدي استجابة أكبر لبعض ترددات الأحمال الموجية.

- طريقة استجابة التردد " frequency-response approach": وعليه يتم تطبيق حمل موجي بتردد واحد من 0.1 ثانية – 1.6 ثانية منفردا على النماذج بزمن موجي طبيعي محدد للنموذج حر الحركة.

وعليه تظهر رسومات النتائج للإزاحة والازاحة النسبية التغير في الزمن الموجي الطبيعي للمنشئات محدودة الحركة وتقارن الاختلاف في الاستجابة القصوى بين النماذج الحرة والمحدودة.

نسبة الزيادة في الازاحة النسبية بين النماذج حرة الحركة ومحدودة الحركة موضحة في الرسومات القياسية بالنسبة لزمن التردد للأحمال والزمن الطبيعي للمنشأ.

تم اعتماد النماذج بمقارنتها مع الدراسات السابقة ومن ثم دراسة أثر تغير خصائص الوصلات وعلاقتها بأداء المنشأ محدود الحركة.

وعليه تم عمل رسومات نظرية بسيطة لتوقع أثر الوصلات الانشائية على النماذج المختلفة.

بعد إتمام هذه الدراسة، يتضح أن الوصلات الانشائية بين المبنى والأرض المجاورة لها أثر كبير على زمن التردد الطبيعي للمنشأ وعلى الجساءة الأفقية اللحظية للمباني الاطارية.

وعليه تشير نتائج هذه الدراسة أن تصميم المبنى لنمط توزيع قوى القص بناء على استجابة المبنى حر الحركة يمكن أن تكون غير واقية للتقليل من حدوث أو اتساع الضرر في المبنى الإطاري الذي يتعرض لقدر أكبر من التشوه الغير مرن نتيجة الإعاقة في مدى الازاحة الأفقي.