EFFECT OF NON-STRUCTURAL ELEMENTS ON STRUCTURE AND DAMAGE ASSESSMENT

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Report By

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DECLARATION

We hereby declare that this project report entitled "**EFFECT OF NON-STRUCTURAL ELEMENTS ON STRUCTURE AND DAMAGE ASSESSMENT**" submitted to the "NUST Institute of Civil Engineering", is a record of an original work done by us under the guidance of Supervisor "Dr. Sarmad Shakeel" and that no part has been plagiarized without citations. Also, this project work is submitted in the partial fulfillment of the requirements for the degree of Bachelor of Civil Engineering

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DEDICATION

We would like to dedicate our work to our teachers and

parents, without whom this could not have been possible.

ACKNOWLEDGEMENTS

Praise be to Allah SWT for giving us the knowledge and strength to complete this research**.**
We would like to wholeheartedly thank our advisor **Dr. Sarmad Shakeel** for helping us throughout the final year project. Without his keen guidance and support, it would not have been possible for us to meet the scope of the project in time.

Abstract

The facades are usually considered as architectural elements and are not accounted for in the structural design, even though they could significantly influence the overall performance of the structure. This study aims to examine the impact of NSE (Sliding and fixed Façade) on the general structural behavior and evaluate the resulting damage. A 2D, 5-storey, 3 Bay MRF was modelled on OPENSEES, and the behaviour of the façade was modelled through a non-linear spring by using a Pinching4 material. Static and dynamic non-linear analyses were conducted on, with and without façade and the results were compared. Then, damage estimation was done by comparing the results with fragility curves. The results showed that the presence of Façade increased stiffness of the building and thus, decreased drifts and time periods. From damage estimation, we can also state that sliding façade behaves good as compared to fixed façade and is prone to less damage.

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1. Introduction

1.1 Background

As non-structural elements contribute a major portion in overall cost of the structure, so, if we want to make more resilient and strong structures, understanding the impact of non-structural factors on the overall performance and damage assessment of buildings becomes important. The interaction between non-structural elements and structure is an extremely important topic in the field of civil engineering.

Non-structural elements include partitions, facades, ceilings, electrical and mechanical systems, and interiors. Despite these elements not directly supporting structural loads, they play a central role in determining the overall behaviour and response of the building during extreme events such as earthquakes, storms, wind, and other dangerous situations.

The aim of the study is to show the effect of non-structural factors on the structural behavior of buildings by doing damage assessment. After studying the interactions between the façade and the main structure, we will be able to understand the overall behaviour of the building under different loading conditions.

The importance of considering non-structural elements is based on the fact that structural performance is getting affected and safety and well-being of occupants is at stake. For example, during a seismic event, non-structural elements can amplify dynamic forces, alter the distribution of mass and stiffness, and can even become a potential source of danger themselves. In addition, after a disaster, damage assessment of non-structural elements becomes important for safety of people and rehabilitation of a structure.

The goals of this study include:

- Review of the existing literature on the behavior of non-structural elements, their vulnerability, and their effect on structural integrity. Dynamic response analysis of buildings with different non-structural configurations through numerical simulation.
- Develop a damage assessment methodology that incorporates non-structural factors assessment to accurately quantify overall structural damage and post-event safety conditions.

The study will provide valuable contribution into the design, evaluation and retrofit of buildings, making the structures stronger and more resilient structures. By considering the influence of façade on overall performance, engineers and designers can take targeted measures to minimize damage and ensure user safety. Through this research, a better understanding can be developed about how much non-structural elements (façade) get damaged when different static and dynamic analysis are run on the model.

1.2 Introduction to LGS Façade and Connection

Mild steel is made of cold formed steel or cold rolled steel. The construction process using LGS sections is similar to the construction of wood. These parts are usually zinc plated to prevent corrosion. The LGS part is used in both standard and nonstandard frameworks. The reason why it is called mild steel is its thickness. It is available in thicknesses of 1-3 mm for products and 0.6-1 mm for non-standard products. There are two main types of connections for LGS façades:

Sliding Connection: In a sliding connection, the bottom end of the LGS façade is firmly fixed to the building, just like in the sliding connection. However, the top end of the façade is allowed to move or slide horizontally.

Fixed Connection: In a fixed connection, both the top and bottom ends of the LGS façade are firmly fixed in place. In this type of face connection, there is no room for any lateral movement.

1.3 LGS vs Masonry Infills

LGS Facade has many advantages over masonry infills making it a superior choice for engineers:

Light Weight and High Strength: Compared to building materials such as concrete or masonry, LGS steel is lightweight and still provides exceptional strength and durability. This lightness simplifies transportation, handling and field installation.

Design flexibility: LGS curtain wall gives architects and designers more flexibility in terms of shape, form and beauty. Steel frames can be easily designed and customized to create unique designs, curves and patterns for a unique architectural statement.

Construction Speed: The prefabricated structure of the LGS system allows faster construction compared to traditional methods. Steel framing components are usually produced on site in a controlled factory environment and then transported to the job site for assembly. This shortens construction time and reduces labour costs.

Sustainability: LGS facades are considered environmentally friendly due to recycling metal and reducing waste during construction. In addition, steel is a non-combustible material and provides fire resistance to LGS façades and increases overall building safety

Materials: Mild steel has excellent properties with high strength-to-weight ratio, corrosion resistance and durability. LGS facades can withstand various environmental loads such as wind, earthquake and thermal expansion and provide long-term durability.

1.4 Performance based design and Damage Assessment

Performance-Based Design is an approach used to design any type of building of any complexity and any scale. Without a precise prescribed technique to achieve those objectives, a structure erected in this way must nonetheless adhere to certain observable or predictable performance requirements, such as seismic load or energy efficiency. The old, specified building codes, on the other hand, require certain construction methods, such as the size of the studs and the spacing between them in wooden frame construction.

With such a strategy, it is possible to create tools and methodologies to assess every stage of the building process, from business transactions to procurement to construction and outcome evaluation. By adopting performance-based design, architects, engineers, and other stakeholders can collaboratively design buildings that are not only functional, but also efficient, sustainable, and adapted to user-specific needs. This strategy encourages innovation and originality while maintaining necessary performance requirements.

Ultimately, performance-oriented design aims to create structures that not only meet minimum standards, but exceed them in terms of usability, practicality, safety and energy efficiency. The emphasis is on using a comprehensive design approach that considers the interactions between various systems and parts and creates optimized and well-performing structures.

Damage assessment is the process of determining how a loss, disaster, or tragedy has affected a building. This can be used to assess repair coverage, insurance coverage, eligibility, or structural restoration. Depending on the type, severity and scope of the damage, damage assessment may take up to 24 hours after the event.

Damage assessment can be done in a variety of situations, including those involving man made and natural disasters. To assess damage in particular areas, such as buildings, bridges, electrical systems, or transportation networks, the process frequently involves skilled professionals, such as engineers, inspectors, or specialists.

Stakeholders can allocate resources, make informed decisions about repairs and reconstruction, and improve safety by using the data gathered during the damage assessment. It facilitates the coordination of recovery activities, prioritizes the allocation of resources, and directs the creation of action plans.

Both approaches are used in this study.

2. Problem Statement

2.1 Problem Identification

Non-structural elements contribute to the overall performance of a building, but they also pose unique challenges. Common challenges associated with non-structural elements include:

Damage and Failure at the time of disaster: Non-structural elements are often susceptible to damage and failure during natural disasters such as earthquakes, hurricanes, and floods. These elements can delaminate, disintegrate, or scatter, posing a hazard to occupants and hindering evacuation efforts. Ensuring the robustness and resilience of non-load-bearing elements against extreme events is a major challenge.

Building performance and personal safety: Non-structural elements play an important role in the overall performance and safety of a building. Improper construction, installation and maintenance of these elements can affect fire resistance, acoustic performance, thermal insulation and indoor air quality. This can affect occupant comfort, health and safety, highlighting the need for proper integration and quality control.

Compatibility with Structural Movement: Buildings are subject to many types of motion and vibration due to factors such as thermal expansion, wind loads and seismic activity. Non-structural elements must be designed and installed to accommodate these movements without damaging or affecting performance. Cracking, delamination, or loss of functionality can occur if structural behaviour compatibility is not addressed.

Maintenance and Durability: Non-structural elements require regular maintenance to maintain their performance and aesthetics. However, accessing and maintaining these items can be difficult due to their location, height, or complexity. Inadequate maintenance can lead to wear and tear, shortened service life and increased repair costs.

Coordinating Aesthetics and design: Non-structural elements contribute significantly to the architectural design and visual appeal of a building. Achieving the desired aesthetics while considering functional requirements, constructability and cost efficiency can be challenging. Coordination between architects, engineers, contractors and suppliers is critical to ensure design intent is met without compromising performance or budget.

Code Compliance and Regulation: Building codes and regulations often contain specific requirements for non-structural elements related to aspects such as fire protection, accessibility and energy efficiency. Staying up to date with evolving regulations and ensuring compliance with relevant regulations can be a challenge, especially for complex or innovative building designs.

2.2 Research Gap: Identifying Knowledge Gaps in Effect of NSEs

As non-structural elements contribute to almost 80% of the total cost of the building, NSEs like façades are the first elements to get damaged in a disaster (especially earthquakes), so they need to be taken care of. As these NSEs like façades do not carry structural loads so they are not properly designed. Moreover, no proper studies are found specifically on the façade, this made a huge research gap over the years. Although these infilled façades are not carrying any structural load they are connected to the structural members through a fixed connection. Therefore, in case of earthquake loading these infilled façades and partition walls also undergo seismic loading. In cases where these infilled façades are not designed properly, they may undergo partial or complete failure.

Failure of these NSEs affects the functionality of the building. In addition to this, falling components of façades can also injure people standing below them and require and Façades require high repair or replacement costs. All of these concerns demand through studies of seismic response and failure mechanism of the infilled facades and partition walls.

In the past studies have been carried out to determine the seismic response of these infilled façades and partition walls but the number of these studies is small. In addition to that, most of these studies were based on the experimentation with individual façades and partition walls. The seismic response of these infilled facades and partition walls is still not fully understood. The major hindrance in this regard was the complexity of the project need to be carried out to study their effect. As construction of a whole building just for research purposes is not an easy task.

2.3 Damage Assessment and Building Performance

An effective way to evaluate building performance is to subject a building model with external walls to various real loads for damage assessment. This made it possible to estimate how much damage the facade would have suffered in a specific case. This process allows us to design structures that respond more effectively to the stresses of the real world, creating a safe and flexible space for the occupants.

Applying different loads to the building model, such as wind, seismic or other related loads, facades can be observed and analyzed. This helps us understand how the facade will perform in these situations and how much damage it can sustain. By carefully evaluating the results, you can identify opportunities for improvement and take steps to improve the performance of your facades. This approach provides valuable insight into the performance of facades and allows us to design structures with better facilities to withstand and reduce damage from accidents in reality.

3. Research Objectives

The Objectives of this study are to

1. Analyze the effect of LGS façade on overall response and performance of structure when subjected to seismic loads. For this purpose, we used different real earthquake loads to check the seismic response of the building with and without façade.

2. Analyze the effect of LGS façade on structure, when subjected to different static pushover loads.

3. As already explained, there are two types of façade connections sliding and fixed, so to check the response of the building incorporating façade with both type of connections and comparison of results.

4. According to the response of the building subjected to static and dynamic loads, calculation of the damage to the building for both types of connections.

5. Then, according to those responses for each type of loading, find out the repair and replacement costs for each type of connection, and comparison of result to conclude which type of connection is better.

4. Literature Review

4.1 LGS Infilled Walls as Façades and Partition Walls

Cold-formed Light Gauge Steel (LGS) walls are the most commonly used non-structural walls used as façades and partition walls [Jose´ I. Restrepo et al. 2011; Ali Sahin Tasligedik et al. 2014]. The non-structural elements are mostly not designed to carry vertical or lateral loads other than their self-weight. But these LGS walls are mostly connected to the load-carrying members at multiple locations. During the event of an earthquake, these LGS walls undergo excitation imposed by the structure. As a result, the damage in these elements is associated with inter-story drift drifts and deflections of the primary structure. [ASCE 7-10] recommends the wall connections should be designed with adequate deformability to account for the variation in displacement. However, the introduction of deformability designs in the industry is a very challenging task. The reason is not only because of their complex geometric configuration but also due to the lack of studies investigating the force-deformation behaviour of these LGS elements under seismic loading. Only a few studies are available in the literature which can be used for the calibration of models of LGS partition walls for seismic analysis. The most relevant work done in this regard was by [Restrepo and Bersofsky. 2014]. Similar work was done by Japanese researchers [Lee et al. 2006]. The main difference between the work of [Restrepo and Bersofsky. 2014] and [Lee et al. 2006] is that the latter didn't use the top tracks in their models.

4.2 Damages to LGS Infilled Façade and Partition Walls

Cases from the past earthquake have shown severe damage to these LGS walls. Damages such as cracking of gypsum boards, bending of studs, failure of connections between track and slab, and partial/complete failure of infilled façades and partition walls were reported during these earthquakes [Craig Jenkins et al. 2016]. Damage to these LGS walls is a major concern as failing partition walls and façades not only cause economic losses [Miranda et al. 2012; Baird et al. 2014] but the falling components from these partition walls and façades can also injure the people below. The majority of the earthquake damages are to the non-structural infilled façades and partition walls [Craig Jenkins et al. 2016]. The buildings may be structurally intact after an earthquake event but the damaged/failed infilled façades and partition walls can make their immediate occupancy impossible. The functionality of strategic buildings is very critical after an earthquake event making the seismic behaviour of non-structural components very crucial to be considered [De Stefano et al. 2012].

4.3 Damage Propagation in LGS Infilled Façade and Partition Walls

Therefore, to investigate the seismic behaviour of the LGS walls and to use these findings in engineering practice several studies have been undertaken. For example, [McMullin and Merrick. 2005] and [Rihal SS. 1987] investigated the damage propagation of LGS partition walls but they didn't consider any specimen mounted on the structural frame. [Lee et al. 2006] investigated the 12 seismic performances of LGS partition walls used in Japanese industry. They developed the full-scale models of partition walls mounted on the structural frame. Using the quasi-static loading they assessed the damage against the structural response like inter-story drift. [Restrepo and Bersofsky. 20007] studied the behaviour of 16 CFS partition wall specimens under quasi-static loading. [Lang, A. F., and Restrepo, J. I. 2007] were the first to present the seismic damage metrics for the partition walls subjected to quasi-static cyclic loading.

4.4 Quasi-Static Raking Testing of LGS partition Walls

[Restrepo and M. Bersofsky 2010] performed quasi-static raking testing on LGS partition walls with gypsum boards. Eight identical LGS partition wall specimens were tested with variables like stud thickness, stud spacing, spacing of self-tapping screws, and wallboard thickness. Based on the results they presented the in-plane seismic performance characteristics of the LGS partition walls. Also, they observed different limit states in partition walls. Based upon their observation these limit states were divided into three groups of Damage States. These damage sates were related to the interstory drift ratios. Damage State I occurred at 0.05-1% inter-story drift ratio required minor repairs if needed. Damage State occurred at 0.5-1.5% inter-story drift requiring repairs which may interrupt usual business, Damage State I occurred at 0.5-3% inter-story drift ratio requiring complete change of partition wall. [Davies et al. 2011] studied the fragility of the walls under in-plane simulated seismic loading. They used numerical models to study the in-plane seismic response of full-scale LGS partition walls. Experimental data was used to develop a numerical model using the System for Earthquake Engineering Simulation (OPENSEES) modeling platform. To capture the non-hysteretic response of the LGS partition walls, a lumped material was developed using the pinching 4 material available in OPENSEES.

4.5 Full Scale Building Testing

While competent level studies have been performed by many researchers. Only a few tests have been performed on full-scale building levels. Full-scale building tests are very important as they allow to study of the interaction between the walls and primary structure. Also, they provide an opportunity to study the interaction between these walls and other non-structural components. A few examples of system-level tests performed are [Restrepo and Lang 2011; Sasaki et al. 2012]. Also, most of the studies were done for the interior partition walls and only a few studies are available that investigate the exterior LGS façade performance against the seismic loadings. Some examples of these studies are [Nakata et al. 2012; Schafer 2013]. [Gennaro et al. 2013] performed the shake table testing of plaster board-based LGS partition walls to evaluate their performance against the seismic loading. A full-scale story model LGS partition was developed and tested in both horizontal directions with eleven different shaking intensities to investigate seismic damages and various inter-story drift demands. [R. Retamales et al. 2013] carried out an experimental program to study the seismic response and fragilities of gypsum board-based LGS partition walls. For this study, in-plane quasi-static and dynamic testing was performed on 36 LGS partition wall specimens with 16 different configurations. Effects of variables 13 like stud connection type, framing thickness, and partial wall height were studied through these 16 configurations. [X. Wang et al. 2015] performed the shake table testing of the full-scale five-story building. The study was done to see the performance of LGS partition walls and façades against seismic loading and to see their interaction with the primary structure and structural elements. The experiment studied the relationship between the drift demands of building with the damaged state of partition walls.

4.6 LGS Partition Walls Surround by RCC Structural Members

In the study [Tatiana Pali et al. 2018] the seismic behaviour of partition walls surrounded by RCC structural members from all sides, was observed. For comparison the partition walls connected to beams or floor from top and bottom, having connections with transverse façades at their ends were also tested. The study aimed to study the seismic behaviour of partition walls while incorporating the effect of the surrounding structural members as well as non-structural elements like outdoor façades. During the study effects of constructive parameters like fixed or sliding connections and sheathing panel types were investigated to find the effect of lateral response in secant stiffness in the case of quasi-static reversed cyclic loading. It was found that the specimens with sliding connections showed same level of damages as in with fixed connections, at a higher inter-story drift ratio. Making sliding connections more advantageous in case of seismic vulnerability.

4.7 Assessment of Local Behaviour and Response to Quasi-Static Loading

[Tatiana Pali and Sarmad Shakeel. 2019] presented their study based on the seismic performance evaluation of LGS non-structural components. The study was backed by experiments on several LGS framed non-structural components like indoor partition walls, façades, and suspended continuously suspended ceilings having gypsum or cement infills. The tests were performed systematically starting from the ancillary level, then element level tests, and finally assembly-level tests. These test series assed the local behaviour, response to quasi-static loading, and dynamic behaviour of these LGS non-structural components. Main findings of this study showed that stud spacing greatly influence the characteristics of partition walls like stiffness, strength and fundamental vibrational frequency. Even though experiments were performed by researchers and computational models are being developed there is still a need for more studies for defining the seismic objectives necessary for LGS walls in future seismic design codes. To tackle this need [Sarmad Shakeel et al. 2019] compared the seismic fragility evaluation of LGS partition walls with those already present in the literature. This was done to study damage progression which was further used for the identification of damage type and for studying their association with different damage states.

4.8 Seismic Structural Collapse Analysis Using FLPH Elements

[Filipe L.A. Ribeiro, Luis A.C. Neves, Andre R. Barbosa] The main objective of this paper was to present a unified implementation algorithm of the ModIMK deterioration models for use in CPH and FLPH models. For the CPH model, new implementations were provided for updating the unloading stiffness and the post-yield hardening ratio, as well as the computation of the committed member displacements and the updated spring displacements. For the FLPH models, an extended calibration procedure was proposed, which updates the flexural stiffness of the interior sections of the member to provide objective and consistent element responses when empirically calibrated moment rotations rules are employed for cyclic analysis.

4.9 Summary

The present studies are mainly based on experimental work and extensive shake table testing. Similarly, most of the previous studies discuss results that were derived from the shake table testing of individual LGS façades and partition walls. Few studies are available where the seismic response of the LGS partition walls and façades was studied on a story level and most of those experimental 15 setups were having RCC members around the partition walls and façades and even fewer studies are present where the seismic performance of LGS non-structural components is evaluated on a full-scale building level. The reason behind this is the fact that performing these experiments on a building level is challenging and complex. In recent years' computational models are being developed of individual models, on platforms like OPENSEES. This study aims to use the previous models and experimental data as reference and construct the building as well as infilled façades model. These façades and building models will then be used to study the effect and contribution of the LGS\façades to the seismic performance of the whole building.

5. Methodology

5.1 Description of Model

5.1.1 Description of Façades

Due to their exceptional material qualities, including strength and stiffness, lightweight steel facades are frequently used as non-structural building components. Other non-load-bearing external building components, like curtain walls and claddings, do not compare to facade walls in any significant way. Contrary to curtain walls and claddings, which are ordinarily not infilled within the building frame, façade walls are either fully or partially infilled within the structural framework. In addition, they are supported by a bottom structural component like a floor slab or beam.

Typically, metal connectors attached to the building face support curtain walls and claddings, allowing them to be independent of the building's structural elements. The structural system of a building's façade walls, on the other hand, is integrated into it and adds to the building's overall stability and load-bearing capacity. They are essential in transferring loads from the façade to the supporting elements and maintaining the building's structural integrity.

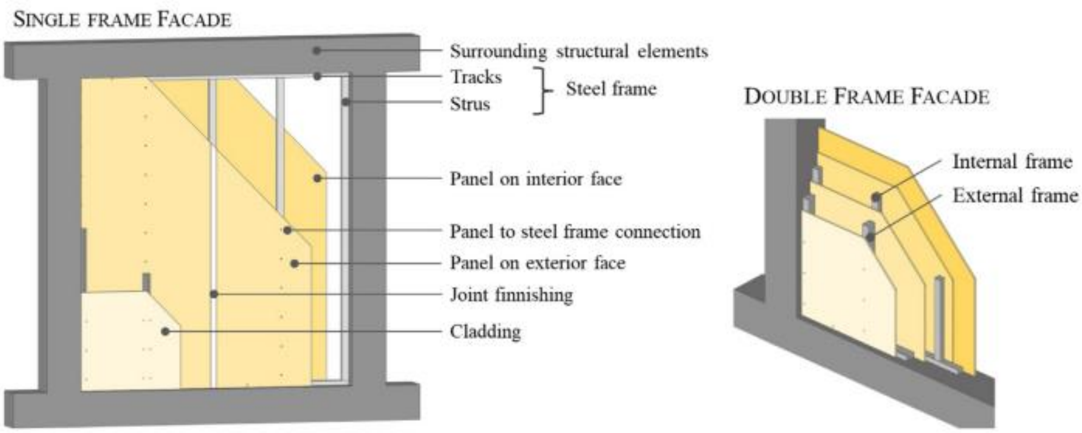


Fig 5.1: 3D illustration of the constructional details of lightweight steel drywall façades

LWS Facades have two functions: they enhance the building's architectural beauty and are essential in preventing financial losses during seismic events. With this study, we want to find out how these facades affect how well the overall structure functions and how much damage they sustain. The facades chosen for this study are 2400 mm in length by 2700 mm in height, and they have varying thicknesses between 75 mm and 211 mm. In our model, the W-06 and W-20 facade types are specifically used.

	Type of top connection	Material	Studs	Bracket/Studs	External frame interior face	External frame exterior face
W-06	Sliding	Steel box profiles	C 75 × 50 × 0.6	U	1 × GKB	1 × Aqua panel
W-20	Fixed	Steel box profiles	C 75 × 50 × 0.6	U	1 × GKB	1 × Aqua panel

Table 5.1: Description of Facades

In this study, the internal frames were sheathed solely on the exterior face using two layers of sheathing panels. On the other hand, the external frames were sheathed on both the exterior and interior faces, employing a single layer of sheathing panels. The sheathing panels chosen for interior face were standardized gypsum boards, specifically 12.5 mm thick, known as GKB boards. These particular sheathing panels, GKB boards, were utilized for both the W-06 and W-20 facades in research. For the exterior faces of W-06 and W-20, a specific type of sheathing panel called the Aqua panel outdoor boards are used. W-06 and W-20 are made in a similar fashion and the only difference between them is the type of connection at the top. The graphical representation of these two facades is shown below:

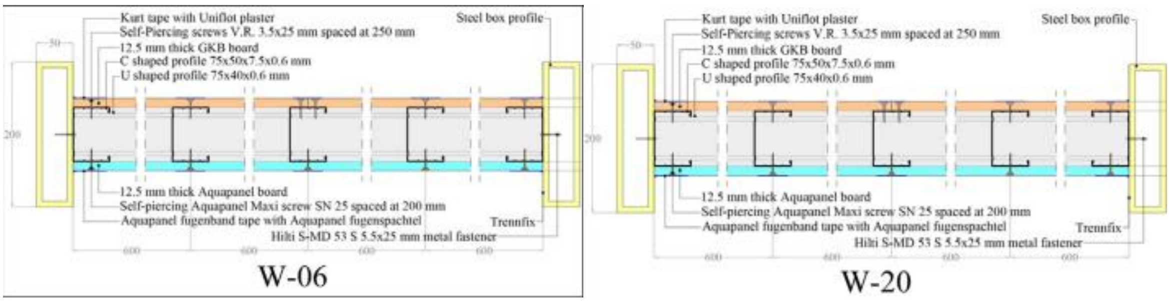
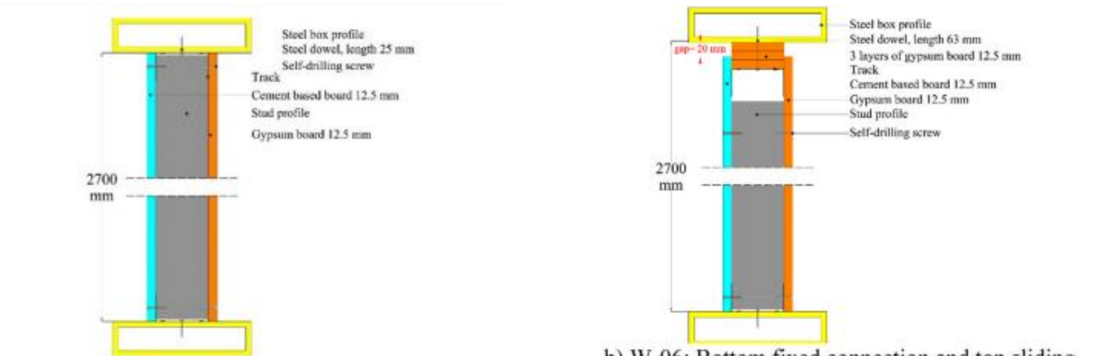


Fig 5.2: Horizontal Section of façade with sliding and fixed connection



a) W-20: Top and bottom fixed connection

b) W-06: Bottom fixed connection and top sliding connection

Fig 5.3: Vertical Section of façade with fixed and sliding connection.

The construction parameter under study is the type of connection between façades (W-06 & W-20) and surrounding constructional elements. We checked in this project how fixed and sliding condition effected the structural performance and damage assessment.

5.1.2 Sliding and Fixed Connections

Two types of connections play a decisive role in the construction of facades: fixed connections and sliding connections. These connections have different properties that affect the performance of the facade during seismic events such as earthquakes. Let's start with fixed connections. In this type of connection, there are no degrees of freedom at the top and bottom, which means that the attachment is rigid and does not allow any movement. This stiffness ensures that the facade is firmly anchored, preventing any lateral movement in the event of a seismic event. The lack of movement helps maintain the building's structural integrity. However, in the case of an earthquake, the forces generated are transferred to the facade, causing the facade to completely overload and even fail.

Sliding joints, on the other hand, offer a different approach. In this design, the lower part of the joint is securely fastened, but there is a 20 mm gap at the top. This arrangement allows some movement and flexibility in the upper part of the facade, allowing it to accommodate greater lateral deflection during seismic events. By providing this space, sliding joints ensure that the facade can move within a controlled range without compromising its overall stability. The inclusion of sliding joints becomes particularly important in scenarios involving damage assessment, especially after earthquakes. Sliding joints allow for efficient redistribution of forces and energy, effectively reducing the effects of seismic loads. This feature helps protect the facade system by reducing stress concentration and the potential for failure. By providing controlled movement, sliding joints promote a more flexible response during earthquakes, limiting the extent of damage.

5.1.3 Structural Definition

Residential buildings classified as Ordinary Moment Resisting Steel Framing (OMRF), meaning that steel is used as the main structural component. The structure is five stories high and has an area of 100 x 150 square feet. Floor slabs consist of structural steel decks and W-sections, materials often used in steel construction to create columns and beams within the structure. A column with a W24x131 section has a specific surface area of 38.5 square inches, while a beam with a W27x102 section has a specific surface area of 30.0 square inches. Columns and beams have specific moment of inertia (I) values that measure their resistance to bending. The moment of inertia (I) of the column is 4020.0 square inches to resist the forces acting on it, while the moment of inertia (I) of the beam is 3620.0 square inches. The structure varies in floor height, with the first floor being 12 feet high and the other floors being 10 feet high. To create the desired layout, position the columns so that they create a 20-foot-wide space along the x-axis and a 25-foot-long space along the y-axis. This connection effectively distributes the load and ensures the overall stability and strength of the building. To integrate facade elements into the construction model, OPENSEES uses spring and truss elements. The truss component connects to the structure and the actual facade force-displacement are assigned to spring to simulate and evaluate the effects of structure and injury assessment.

5.2 Introduction to OPENSEES

5.2.1 Basic Introduction to OPENSEES

OPENSEES is used to model and analyze structures under various loading scenarios. It focuses on earthquake engineering and structural dynamics, providing a comprehensive platform to study how structures respond to different loading situations. OPENSEES provides a customizable setup for performing complex numerical analyzes and simulations. OPENSEES allows you to analyze and model complex structural systems, apply different types of forces and simulate dynamic responses. It is capable of processing a variety of construction materials such as concrete, steel, wood, and composite materials, and is suitable for a variety of engineering fields. One of the outstanding features of OPENSEES is that it uses TCL (Tool Command Language), a scripting language that provides a user-friendly interface. TCL allows users to more easily interact with the software and automate simulations by allowing definition of analysis procedures and regulation of the simulation process. OPENSEES supports various types of analysis, including static, dynamic, nonlinear, and time history analysis. Users can specify boundary conditions and load scenarios, as well as the mass and properties of structural components such as beams, columns, and walls. The software uses the latest numerical algorithms to solve the equations of motion and accurately predict structural responses. The extensive library of predefined material models, elements and analysis methods in OPENSEES further enhances the adaptability of the software. Models can be visualized in 2D or 3D renderings.

5.2.2 Benefits of using OPENSEES

OPENSEES offers a comprehensive and flexible platform for the simulation and analysis of the behavior of structural and geotechnical systems. It allows users to look at different aspects of structural behavior by supporting different types of analysis such as static, dynamic, nonlinear and time history analysis. OPENSEES enables accurate modeling and analysis of a wide range of structures, as it is compatible with a variety of structural materials, including concrete, steel, wood and composites. Its user-friendly interface includes a simple TCL (Tool Command Language) scripting language for improved accessibility and efficient customization and automation. OPENSEES use a cutting numeric algorithm to solve the movement equation that will produce accurate results each time. The software retains time during the modeling process. The software is widely defined by material models, elements and analysis of technology libraries. In addition, the visualization functions of the OPENSEES are provided as a result of 2D and 3D reproduction, which helps to understand and distribute the results of the analysis. Promoting support, collaboration and knowledge exchange and active users helps develop software. In addition, various analyzes such as pushover and dynamic analysis require very little processing power, which means that this analysis takes less time compared to other GUI software that requires more graphics and processing power. OPENSEES, open-source software distributed under the GNU General Public License, is freely available and promotes innovation in structural engineering by promoting accessibility and teamwork.

5.3 Modeling in OPENSEES

5.3.1 Commands Utilized for Modeling

From defining dimension & material properties to setting up degrees of freedom and elements, there is a command to be used with specific arguments. These commands are written in TCL language and called upon by specific OPENSEES libraries and used in a harmonious fashion. The commands that we used are defined below with brief explanation:

Model Basic Builder: It sets up dimension whether the model is going to be 2D or 3D and degrees of freedom in x, y and z axis. We defined our model as 2D model with three DOFs.

Node Command: It define nodes and assign masses to beam-column intersections of frame. Each node was defined separately and nodal masses that were based on combination of dead load and live load were assigned to each node. Since, it is model based on concentrated plasticity approach, separate nodes were defined for plastic hinges springs.

Equal Degree of Freedom Command: It assigns a slave node to a master node in such a way that slave node will translate or rotate in accordance with the master node. The first pier nodes were master nodes and subsequent nodes at the floor were slave nodes and have the same translation as that of the master node.

Fix Command: It assigns boundary conditions to nodes. The three nodes of floor 1 were fixed and the p-delta column was pinned. All the other nodes on floor 2 to roof were assigned as fixed nodes. Uniaxial Material command was used to define the material. Two types of material were used in our model: Elastic and Pinching 4. Pinching 4 material takes argument from already defined Force Deformation Values of façade.

Element Command: It is used to define members such as beam, column and truss elements. Element has to be succeeded by type of member that is to be defined such elastic-beam-column element or truss or zero-length element. Beams and Column were defined using Elastic-Beam-Column elements. Beams in p-delta bay and Truss elements of façade were defined using Truss elements. Non-linear springs were created using Zero-Length element.

Rotational Springs Command: This command is used to define springs that exhibit the non-linear behavior concentrated at the two ends of each member in terms of plastic hinges. So, each member in our model has two springs at each end. The springs are based upon modified IBARA KRAWINKLER DETERIORATION model which defines non-linear moment-rotation relations for them.

Leaning Column Spring Command: It is used to create springs for p-delta column line that is in the third bay of model. It is a specialized spring of zero stiffness that was used in our model to simulate the behavior of rotationally leaning columns. Axial and rotational deformations can occur in rotational leaning columns, which are structural members.

Region Command: This command creates region for elements. The benefit of using this command is that it creates a single group of elements, and it is easier to analyze them based on a single output. This command is used in our model for springs: both façade and non-linear plastic hinge.

Load Command: This command is used to apply loads either gravity or lateral. It applies the load to a specific node in x, y or z direction. Some snippets from the code are given below:

Model Basic Builder: model BasicBuilder -ndm 2 -ndf 3;	Fix: fix 11 1 1 1;
Node: node 12 $Pier1 $Floor2 -mass $NodalMass2 $Negligible $Negligible;	Equal Degree of Freedom: equalDOF 117 12122 1 2 3;
Element (Elastic-Beam-Column): element elasticBeamColumn 112 127 136 $Acol_12 $Es $Icol_12mod $PDeltaTransf;	Uniaxial Material (Elastic): uniaxialMaterial Elastic $TrussMatID $Es;
Element (Truss): element truss 632 32 42 $Arigid $TrussMatID;	Uniaxial Material (Pinching 4): uniaxialMaterial Pinching4 $matID [lindex $Fb 0] [lindex $Db 0] [lindex $Fb 1] [lindex $Db 1] [lindex $Fb 2] [lindex $Db 2] [lindex $Fb 3] [lindex $Db 3] [lindex $Fb 4] [lindex $Db 4] [lindex $Fb 5] [lindex $Db 5] [lindex $Fb 6] [lindex $Db 6] [lindex $Fb 7] [lindex $Db 7] [lindex $cyc 0][lindex $cyc 1] [lindex $cyc 2][lindex $cyc 3] [lindex $cyc 4][lindex $cyc 5] [lindex $gammaK 0][lindex $gammaK 1] [lindex $gammaK 2][lindex $gammaK 3] [lindex $gammaK 4][lindex $gammaD 0] [lindex $gammaD 1][lindex $gammaD 2] [lindex $gammaD 3][lindex $gammaD 4] [lindex $gammaF 0][lindex $gammaF 1] [lindex $gammaF 2][lindex $gammaF 3] [lindex $gammaF 4] $gammaE $dam;
Rotational Spring: rotSpring2DModIKModel 3111 11 117 $Ks_col_1 $b $b $Mycol_12 [expr - $Mycol_12] $LS $LK $LA $LD $cS $cK $cA $cD $th_pP $th_pN $th_pcP $th_pcN $ResP $ResN $th_uP $th_uN $DP $DN;	Region: region 4 -ele 912121 912232 912343 912454 912565;
Leaning Column Spring: rotLeaningCol 5412 42 426;	Load: load 12 0.0 $P_F2 0.0;

Table 5.2: Snippets of Code from the Model

5.3.2 Plastic Hinges, P-delta Bay, Façade Spring

In order to understand and simulate the complex behavior of individual structural components and apply it to the overall structure, several strategies are implemented in this model. These strategies are aimed at accurately representing the response of the structure under seismic forces. Our objective is to comprehensively capture the behavior of the entire structure as it undergoes deformations across different stages, including the elastic range, plastic range, and the post-peak range. To achieve this comprehensive response, we incorporate the use of plastic hinges and p-delta column in our model.

Concentrated plastic hinges (CPHs) are utilized in structural analysis to account for nonlinear behavior at the member level [S. Mazzoni et al., 2009]. They represent the assumption that plasticity is concentrated at the ends of the member, rather than being distributed along its length. This is in contrast to the distributed plasticity approach, where nonlinearity is spread over a finite length. In OPENSEES, the IBARA KRAWINKLER model is often employed to characterize the behavior of CPHs. This model provides a quantification of the bilinear hysteretic response, which describes the relationship between moment and rotation for structural components. The IBARA KRAWINKLER model is derived through empirical calibration and relies on two main curves: the monotonic curve and the hysteretic curve. The backbone monotonic curve is shown below:

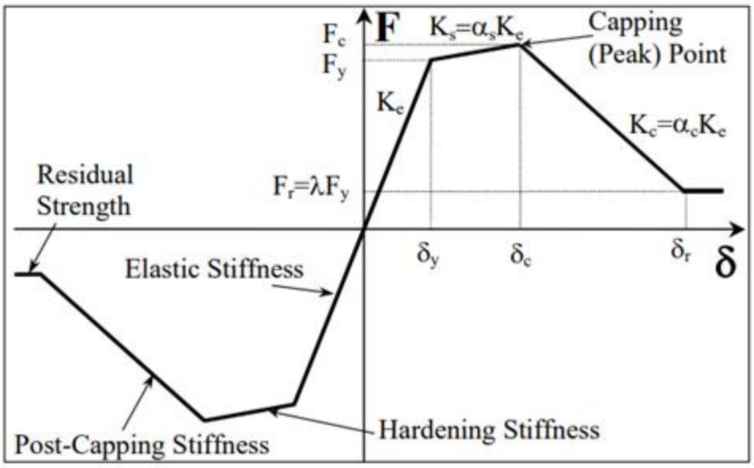


Fig 5.4: Monotonic Curve for Hinges – IK 1

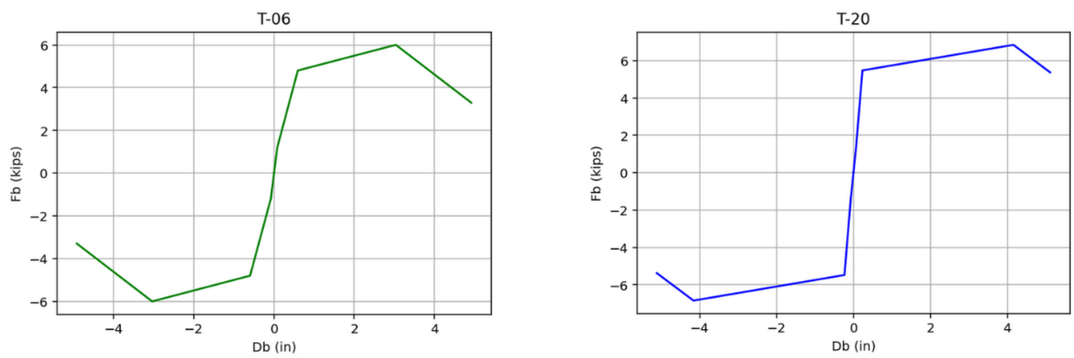
The monotonic curve represents the response of the component under increasing loads until failure. It captures the yielding, strain hardening, and ultimate strength of the material in elastic, hardening, post-capping and residual strength ranges. The hysteretic curve, on the other hand, accounts for the cyclic ehaviour of the component, considering the effects of cyclic loading, energy dissipation, and stiffness degradation. By using the IK model, moment-rotation relationships can be generated allowing for accurate representation of non-linearities in the model. The parameters for this model are taken from OPENSEES website and used in the code to model Concentrated Plastic Hinges [2]. These parameters are given below:

McMy 1.05;	ratio of capping to yield moment
LS 1000.0;	basic strength deterioration
LK 1000.0;	unloading stiffness deterioration
LA 1000.0;	accelerated reloading stiffness deterioration
LD 1000.0;	post-capping strength deterioration
cS 1.0;	exponent for basic strength deterioration
cK 1.0;	exponent for unloading stiffness deterioration
cA 1.0;	exponent for accelerated reloading stiffness deterioration
cD 1.0;	exponent for post-capping strength deterioration
th_pP 0.025;	plastic rot capacity for pos loading
th_pN 0.025;	plastic rot capacity for neg loading
th_pcP 0.3;	post-capping rot capacity for pos loading
th_pcN 0.3;	post-capping rot capacity for neg loading
ResP 0.4;	residual strength ratio for pos loading
ResN 0.4;	residual strength ratio for neg loading
th_uP 0.4;	ultimate rot capacity for pos loading
th_uN 0.4;	ultimate rot capacity for neg loading
DP 1.0;	rate of cyclic deterioration for pos loading
DN 1.0;	rate of cyclic deterioration for neg loading

Table 5.3: Rotational Spring Properties based on Ibarra Krawinkler Model

P-Delta Bay is an important inclusion to a structural model to consider the effects of deflections caused by lateral loads. These deflections can lead to additional moments, known as 2nd order moments, and non-linearities in the structure. The P-Delta Bay portrays the entire structure and is designed to represent the stiffness and strength resulting from the interaction of the frame elements. To ensure an appropriate representation, the area and stiffness of the P-Delta Bay columns and trusses are set larger compared to the frame elements. The loads applied to the nodes of the P-Delta Bay are determined by summing all the loads on the frame nodes present at the same floor level. This ensures that the effects of the loads from the entire structure are properly accounted for within the P-Delta Bay. Truss elements that are axially rigid are used to transfer p-delta effects. In order to transfer the second-order moments to the frame elements, the P-Delta Bay is pinned at the bottom. This ensures that the moments induced by the deflections are properly transmitted to the frame elements for accurate analysis and representation.

To represent the façade in the model, **Façade Springs** are incorporated. These springs portray the behavior of the connections within the façade and are assigned force-displacement curves obtained from experimental data. The experimental study conducted by Luigi Fiorino provides the backbone curves used in the model. Considering that two types of connections exist in the façade model, namely sliding and fixed connections, two distinct backbone curves are utilized. This accounts for the different behavior exhibited by each type of connection. The backbone curves are shown below:



Graph 5.4: Backbone Curves for Sliding and Fixed Connection

The Façade Springs are connected to truss elements, which, in turn, are connected to the main structure. The truss elements have a specified area of 106 square inches and are made of steel with a yield strength of 60 kips. These truss elements serve as the means to transfer the structural demands to the Façade Springs. The Façade Springs themselves are modeled using the Pinching 4 material. This material model is a uniaxial representation that can accurately capture the pinched load-deformation response, including the ability to exhibit degradation under cyclic loading. By employing this material model, the Façade Springs accurately simulate the behavior of the connections, accounting for any pinching or degradation effects that may occur during cyclic loading conditions. Façade characterization in the model is shown below:

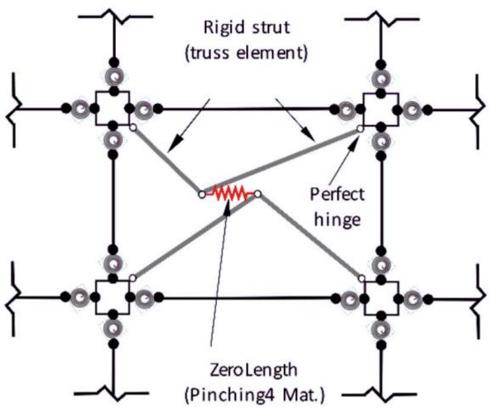


Fig 5.4: Façade characterization in the model

5.3.3 Schematic Representation of Model

This is the schematic representation of the model with conventions explained.

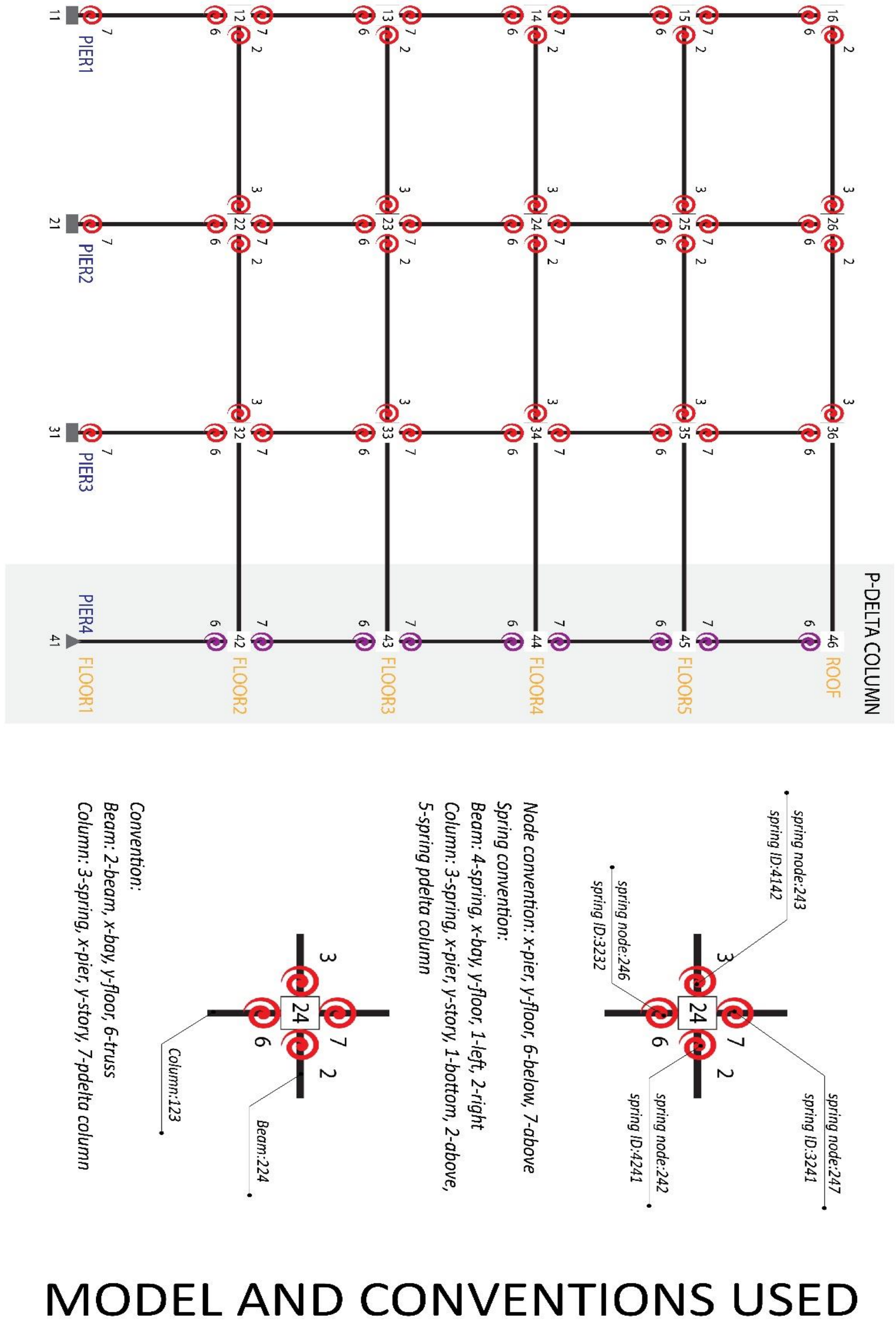


Fig 5.4 Schematic Representation of the Model

5.4 Analysis in OPENSEES

5.4.1 Modal, Pushover and Dynamic Analysis

Modal analysis is aimed at determining the time period of a given structure. The time period, in turn, relies on the mode shape that the structure can exhibit during the modal analysis process. The mode shape refers to the characteristic pattern of motion that the structure demonstrates when subjected to vibrations or oscillations. During modal analysis, the structure's response to harmonic excitations is examined to identify its natural frequencies and corresponding mode shapes. These natural frequencies represent the inherent vibration frequencies at which the structure tends to oscillate.

By understanding the mode shapes associated with these frequencies, valuable insights can be gained into the dynamic behavior and stability of the structure. Modal masses represent the distribution of mass among different modes and aid in assessing the system's energy distribution. The modal analysis aids in optimizing structural design, assessing structural integrity, and ensuring adequate performance under dynamic loading conditions. The Time Periods of the Structure depend upon the masses assigned to structure and stiffness of the whole structure that is the combination of stiffnesses of the cross section.

Pushover analysis is used to evaluate the structural response to lateral loads. It is a static analysis that can be done in two ways: load-controlled analysis and displacement-controlled analysis. In load-controlled analysis, lateral loads are gradually applied to the structure in defined steps. The load gradually increases so that the structure can be gradually deformed to its ultimate capacity. On the other hand, the displacement control analysis involves the set steps to increase the displacement until it reaches the height of the building.

The combined horizontal reaction force and roof displacement at the base of the structure are recorded and plotted during the analysis. These graphs provide valuable insight into the structure's behavior and response to lateral loads. Engineers can assess the ability of the overall structure to resist lateral forces by observing the combined horizontal reaction at the base. In addition, roof displacement measurements provide a clear understanding of the displacements experienced by the upper stories of the structure, providing insight into the potential for damage or excessive deformation.

Dynamic analysis is considered the most accurate method for evaluating the response of a structure to dynamic loads such as seismic events. During such events, this modeling approach closely replicates the actual conditions that structures face. Dynamic analysis provides valuable insight into structural behavior and helps ensure the safety and efficiency of structures, by simulating the time-varying nature of applied loads. In dynamic analysis, loads are applied in a cyclic fashion that simulates the repeated forces that occur during a seismic event. Each load point is assigned a specific time period, allowing a detailed study of the structural response throughout the event. Dynamic analysis represents the true dynamic behavior of structures, taking into account time-varying effects such as inertial forces, damping and stiffness.

5.4.2 Utilization of Analysis Methods in OPENSEES

In order to determine the time period in OPENSEES, **Modal Analysis** was performed. We enter the mode shape in this pattern. These modes are a predetermined deformation patterns that can show the structure. These mode shapes were derived based on the previously defined model in which masses and stiffnesses were assigned to structural elements. Using OPENSEES, we calculated time periods for two specific mode shapes. OPENSEES stores mode shapes as eigenvectors and circular frequencies as eigenvalues. The time period is indicated on the output screen.

Gravity Analysis in OPENSEES is used to evaluate the response of structures to dead and live loads. In this analysis, point loads representing the weight of individual components are applied to the relevant model nodes. This allows for an accurate assessment of the effect of gravity on the structure. Single Story loads are summed and applied to P-delta columns to account for P-delta effects. This effect takes into account the secondary deformation caused by lateral deflection. When performing a gravity analysis, the OPENSEES structural model can simulate the behavior of a structure under vertical loading conditions. The analysis provides insight into internal forces, deformations and overall stability, which is critical for structural optimization and structural integrity.

Pushover Analysis was performed using OPENSEES to analyze the structural response to static lateral loads. Lateral loads are determined according to ASCE 7-16 guidelines. The site class was defined as SDC D, and the spectral acceleration values for long and short time periods were determined as Sa=0.38 and S1=1.3. In pushover analysis, calculated lateral loads are assigned separately to each node of the structure. Analyzes were performed using the displacement-controlled test method, where the displacement was increased by 0.01 step. The maximum displacement was set to be up to 5 percent of the story height. The structural response was recorded at each displacement step, to examine how the structure reacted to the applied loads. In simpler terms, the Pushover Analysis simulated the behavior of the structure under lateral loads. The analysis focused on incrementally increasing the displacement while monitoring the structural response at each step. This approach allowed for a comprehensive assessment of the structure's performance and behavior under lateral loading conditions.

In order to access the structure against actual seismic loadings, **Time History Analysis** was utilized in OPENSEES. We obtained data from the PEER Ground Motion Database for three earthquakes to simulate seismic forces. These earthquakes were chosen based on their relevance to the project. To align the response spectrum of the selected ground motion data with the response spectrum of Pakistan, we utilized the Ss and S1 values specified in the Building Code of Pakistan (BCP) and ASCE 7-16. This matching process was performed using software called SEISMOMATCH. By adjusting the parameters, we were able to obtain a response spectrum that corresponded to the desired location. With the matched response spectrum data, we applied the seismic loads to the structural model and conducted the analysis. Several parameters were defined for the analysis, including a damping ratio of 5%, a view scale of 15, a time step of 0.01 seconds, and a total of 2495 analysis steps.

5.5 Recorders and Results Extraction

5.5.1 Function and Purpose of Each Recorder

Upon defining the model, both with and without the inclusion of a façade, establishing the necessary end conditions, applying loading conditions, and specifying the analysis approach, the model's output can be obtained through the utilization of recorders. However, in the case of Modal Analysis, the predefined analysis method generates the desired output directly on the interpreter screen, thereby eliminating the need for additional recorders. However, for the results obtained from the Pushover Analysis, two distinct types of outputs were required. Firstly, the base shear at the foundation level needed to be determined. This involved assigning recorders to specific nodes, namely the three fixed nodes of the actual model and the pin node of the p-delta column. By utilizing the appropriate type of recorder, the corresponding values of the x-direction reaction at these four nodes were acquired, thereby representing the individual base shears at each respective node. Secondly, the roof drift ratio at the roof node needed to be evaluated. This was accomplished by implementing a drift recorder at the top left node, enabling the extraction of the roof drift-to-building height ratio. This recorder facilitated the calculation of the extent to which the roof displaces relative to the overall height of the structure. The desired output is the roof drift ratio for dynamic analysis. The time frame for our analysis was 0.01 seconds and the total response time was up to 6 minutes. Another type of logger used in our analysis is the element logger, which is used to analyze the response of springs, beams, and columns. It is a force type recorder used to record the force in the cell. Drift recorders are also used in each story level to achieve story drift speed.

5.5.2 Output Files and Their Utilization

When the analysis is complete, the recorded data is saved to an .OUT file. To obtain the specific result required for a particular test, the type of test must be changed accordingly. The registration information stored in these .OUT files can be accessed and used using MS Excel or Notepad. Using the recorded data stored in the .OUT file, we can extract and manipulate the relevant information required for the results. In this particular study, the data was scrutinized in MS Excel to gain insights and draw meaningful conclusions. Additionally, to enhance the visual presentation of the results, Python was used to create visually appealing graphs from the analyzed data. By combining data analysis in MS Excel with graphing in Python, we can gain a comprehensive understanding of the recorded information, extract key insights and present the results in an engaging and illustrative way.

6. Results

6.1 Modal Analysis Results

We used OPENSEES to perform a modal analysis to determine the natural frequencies and mode shapes of the system, the time periods were calculated for the following models: (1) Bare Ordinary Moment Resisting Frame (OMRF) (2) OMRF with fixed facade (3) OMRF with sliding facade. The time periods for the first and second states are calculated as follows:

Frame Type	T_1	T_2
Bare Frame	0.764	0.244
Fixed Façade	0.184	0.004
Sliding Façade	0.185	0.003

Table 6.1: Time Periods (T_1, T_2)

Table 6.1 shows findings about the time period (T1) for different frames, including bare, fixed facade and sliding facade frames. The bare frame showed a significantly higher T1 value of 0.764 compared to 0.184 and 0.185 for the fixed and sliding facade frames, respectively. This indicates that adding the facade element has significantly reduced the time period.

The reduction of the time period can be attributed to a number of factors, such as the introduction of additional masses and alteration of mass distribution in the structure caused by the facade element. These elements contribute to the overall lateral stiffness of the frame, increasing resistance to lateral loads. The design of facade elements also plays a critical role in improving the overall lateral stability of the structure, influencing its response to lateral forces and potentially changing its dynamic behavior.

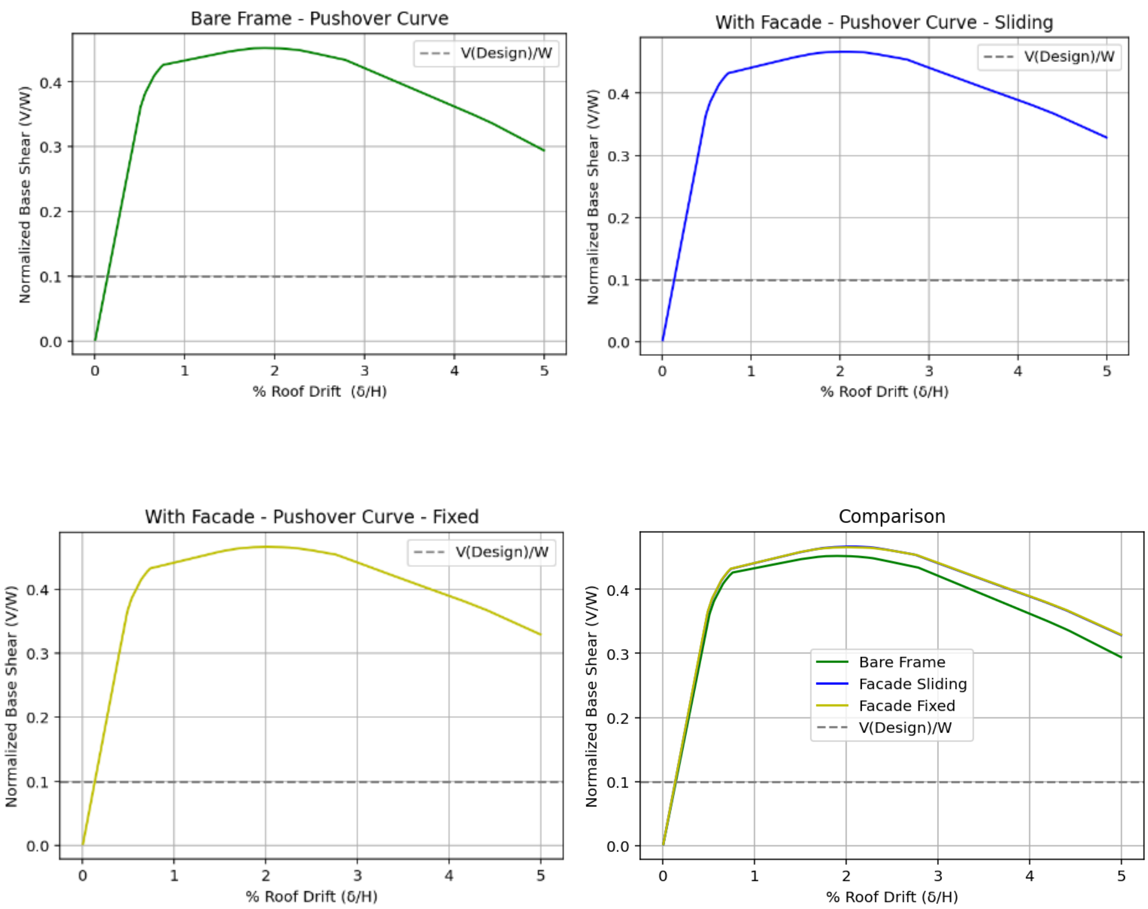
The results showed that the type of connection on the facade, whether fixed or sliding, had no significant effect on the time period. This observation suggests that the influence of the facade connections on the time period becomes relatively smaller in shorter and stiffer structures. In such cases, the dynamic behavior is dominated by the inherent stiffness of the structure, making the effect of facade joints less noticeable.

To summarize, the findings from Table 6.1 highlight that the inclusion of facade elements in the frame leads to a considerable reduction in the time period. The additional mass and altered mass distribution, coupled with the improved lateral stiffness and stability provided by the facade design, contribute to this effect.

6.2 Pushover Analysis Results

Pushover analysis, a nonlinear static analysis is employed to evaluate how the modelled frames perform under seismic conditions and to determine their capacity to withstand such forces. During the pushover analysis, the predefined loading pattern is applied incrementally, step by step. The magnitude of the lateral load is gradually augmented until the 5% displacement target is achieved. To conduct the analysis, specific commands, and procedures are performed in OPENSEES as explained in section 5.

6.2.1 Plotted Graphs from Pushover Analysis



Graph 6.2.1: Individual Pushover curves and a comparison curve

6.2.2 Description and Interpretation of the Graphs

Point	Base Shear(V)	%RDR	Type of Frame
yield point (V_y)	505.82 Kips	0.80%	**Bare Frame**
peak point (V_p)	535.33 Kips	1.87%	
yield point (V_y)	513.713 Kips	0.80%	**Sliding Façade**
peak point (V_p)	552.63 Kips	2.08%	
yield point (Vy)	514.014 Kips	0.80%	**Fixed Façade**
peak point (V_p)	551.28 Kips	2.00%	

Table 6.2.2: Yield (V_y), Peak (V_p) point base shear with respective IDR%

In Fig 6.2.1, we can observe a comparison of pushover curves for different frames. The y-axis represents the Normalized Base Shear (V), while the x-axis represents the %Roof Drift (d/H). For this analysis, the ratio of V(Design) to the weight of the structure (W) is set at 0.1. The frames are allowed to undergo a maximum Roof Drift of 5%.

The results demonstrate that frames incorporating an outer moment resisting frame (OMRF) with either a fixed or sliding facade exhibit higher pushover curves compared to the bare OMRF frame. This increase in curve height is primarily due to the added stiffness provided by the facade. The pushover curves for the different frames closely align with each other until reaching the yield point (Vy) at 0.8% RDR. However, the peak of the curves occurs at 1.9% RDR for the bare frame and at 2.1% RDR for frames with a sliding or fixed facade. Addition of façade delays the onset of the peak and creates a higher lateral resistance.

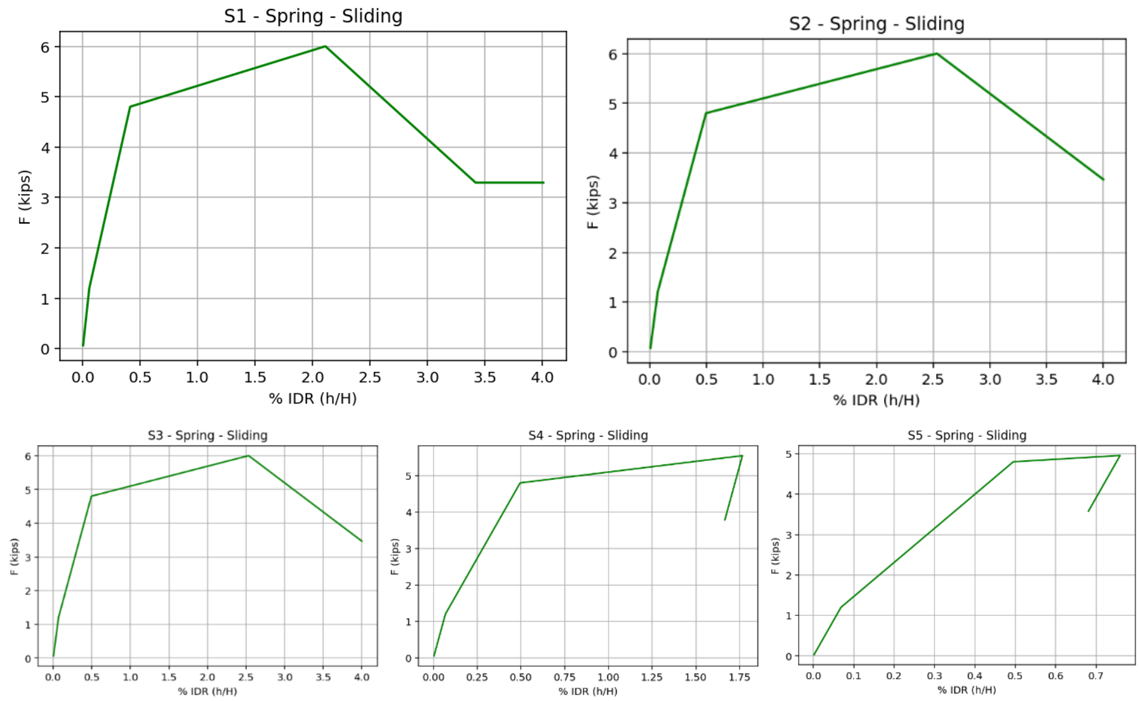
After reaching the peak, the Peak Base Shear (Vp) is slightly different between the sliding facade frame and the fixed facade frame, indicating that Vp is not significantly affected by the change in connection type. However, the Vp on the exposed frame is 15 kips lower than the frame with the facade attached. Similarly, at 5% RDR the base shear difference between frames reaches 40 kips, indicating that the shear curve gradually diverges after the peak point.

In general, Graph 6.2.1 shows that frames with fixed or sliding facades have higher shear capacity compared to bare frames because the facades provide increased stiffness. The peak of the pressure curve is delayed by the presence of the facade, resulting in an increase in lateral resistance. Although there is a small difference in Vp between sliding and fixed facade framing, changing the connection type does not significantly affect the end-to-bottom displacement.

6.3 Façade Springs Results

6.3.1 Behaviour of Springs at Different Stories in the Structure

Axial Force Vs IDR% at story level for Sliding connection.



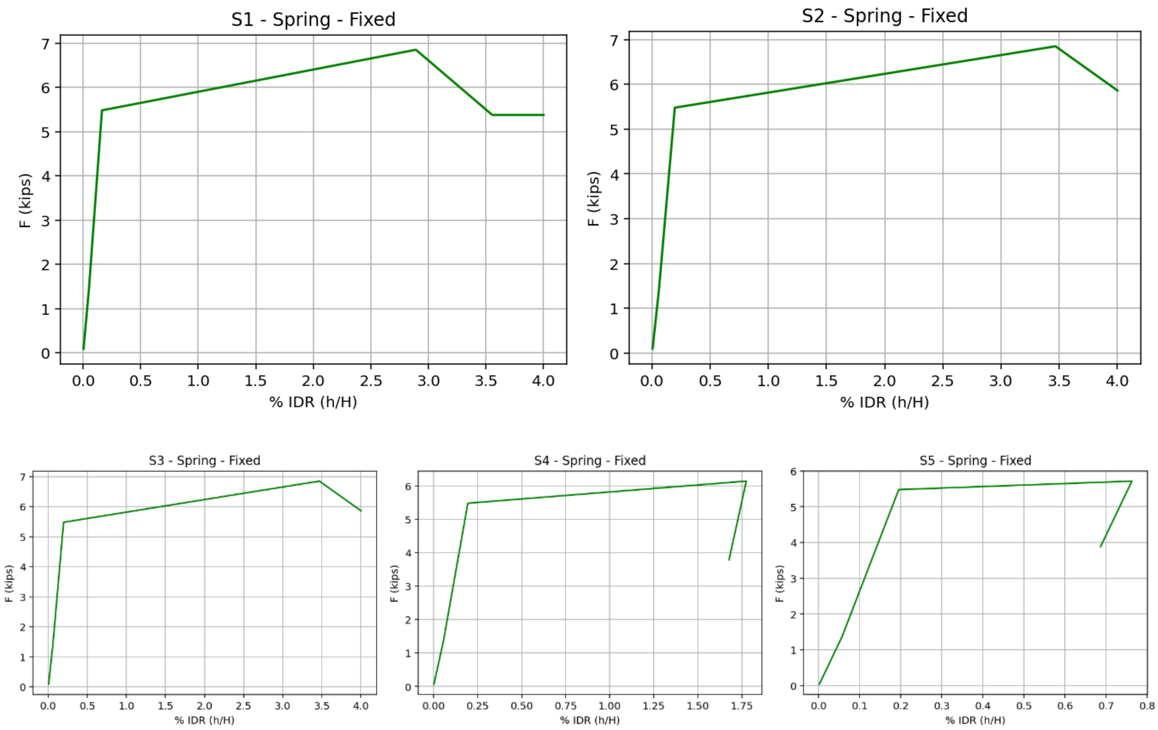
Graph 6.3.1: Axial Force Curves (Sliding)

Graph 6.3.1. presents the Axial Force-IDR (Inter-Story Drift Ratio) Plots for each individual story's facade. It illustrates the behaviour of the facades as they resist external loads. At a specific IDR of 2%, the maximum load applied is 6 kips, primarily resisted by the facade of story 1. This is because the demand for load is higher at lower stories. The first three stories show similar behavior, with the maximum load being reached at an IDR of 2.5%. Compared to the 2% IDR scenario, there is a slight delay in reaching the peak load. The curves of these stories show comparable functions in terms of load resistance.

In contrast, the demand for upper stories is small, resulting in a relatively low curve, and the curve cannot be close to the maximum force that the facade can bear. This suggests that the upper floors do not carry as much load as the lower ones. Moreover, Story 4 and Story 5 achieved the maximum IDR values of 1.76% and 0.75%, respectively. This suggests that load requirements decrease as you move higher.

In summary, Graph 6.3.1 provides an insight into the performance of individual floor facades with respect to axial forces and IDR. 1st Story facade resists for maximum load if the specific IDR is 2% and the first three stories have similar load resistance and have a slight delay to reach the top. Demand for load in the upper story is low, resulting in a relatively low curve. Stories 4 and 5 show maximum IDR values of 1.76% and 0.75%, indicating that the load requirement decreases as one moves up the structure.

Axial Force Vs IDR% at story level for Fixed connection.



Graph 6.3.1: Axial Force Curves (Fixed)

Compared to sliding facades, fixed facades have a relatively high axial force bearing capacity. At floor 1, the fixed facade can withstand a maximum force of 6.9 kips at an IDR of 2.8%. This increased capacity is due to the increased stiffness achieved with fixed connections. However, the facade performance lags in response to peak loads, with an IDR higher than 3.45% in stories 2 and 3. This delay indicates that the facade needs larger displacements between floors to achieve maximum load-bearing capacity.

It should be noted that although fixed connections increase the axial load capacity, the resulting curves are flatter compared to sliding connections. This means that as the displacement between the floors increases, the axial force of the fixed facade changes less.

In addition, it can be observed that the fixed connections produce more pronounced second-order effects. These effects increase floor deflection due to increased bending moments. This means that the fixed facade experiences larger deformations and displacements due to these bending moments.

In summary, the fixed facade demonstrates a higher axial force capacity compared to the sliding facade. However, the response of the facades to the maximum load is delayed to a higher IDR at the 2nd and 3rd stories. While the fixed connection increases the axial load capacity, the resulting curve is more flattened. The fixed connection also amplifies second-order effects, leading to higher story drifts caused by bending moments.

6.3.2 Overstrength Factor

The ability of MRF to resist lateral loads or deformations beyond its design or nominal strength is calculated to check the effect of the Façade components incorporated. Overstrength is calculated by the following formula:

$\Omega = V_{max}/V$

V_{max} = Maximum Base Shear Resistance

V = Design Base Shear

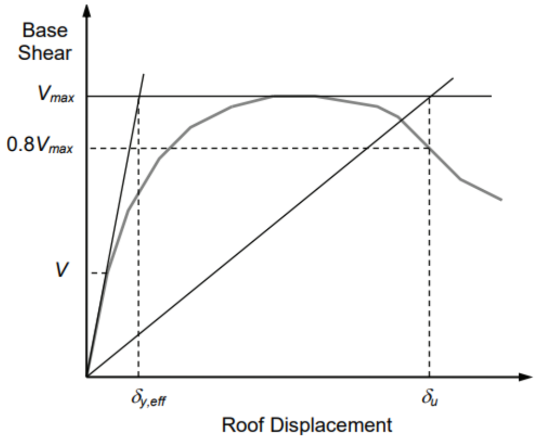
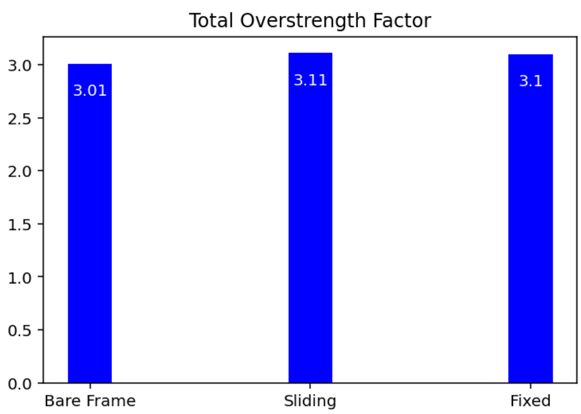


Figure 6-5 Idealized nonlinear static pushover curve

In Graph 6.3.2, the overstrength factor for different frame configurations is presented. The overstrength factor, denoted as Ω, quantifies the structural capacity beyond the design level. The sliding facade exhibits an overstrength factor of $\Omega = 3.11$, which is slightly higher than the value of 3.1 for the fixed facade. In comparison, the bare frame without a facade has an overstrength factor of 3.01. It is worth noting that the ASCE limit for the overstrength factor is set at 3. This indicates that the sliding facade configuration provides better resistance against base shear compared to the bare frame. However, this improvement in resistance can only be observed through non-linear modelling after incorporating the facade elements into the structure.



Graph 6.4.1: Overstrength Factor

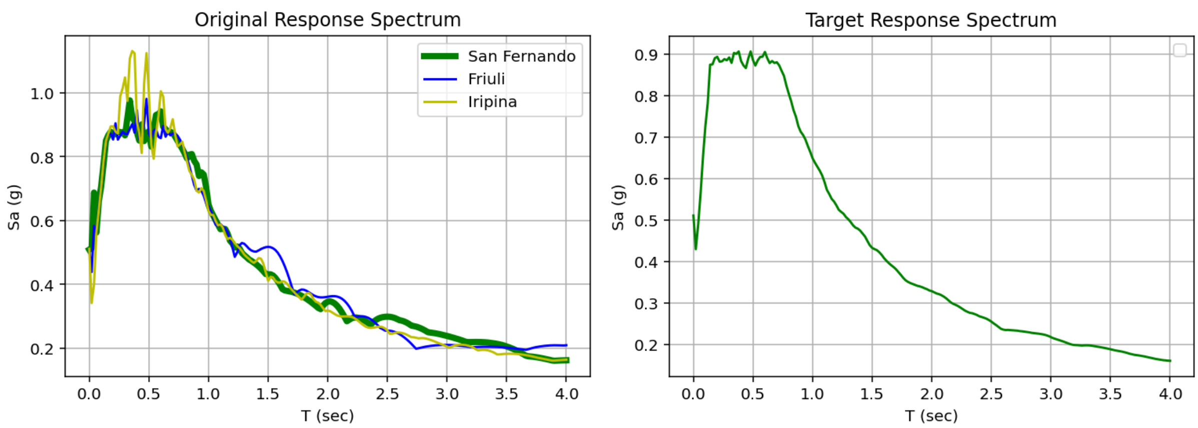
6.4 Time-History Analysis Results

To better understand how a Moment-Resisting Frame (MRF) with a façade behaves during dynamic events, we conducted a Time-History Analysis. We chose Islamabad as our hypothetical location, assuming it has a site class of D. By following the seismic design guidelines from ASCE 7-16, we determined the seismic design coefficients as Ss = 1.308 and S1 = 0.381. Simulating the MRF's response to dynamic forces, like those encountered during earthquakes, was the aim. This analysis allowed us to study how the structure performs over time, considering the time-varying nature of the forces. By considering the specific characteristics of Islamabad ground motions and applying seismic design factors, we gain insight into the behavior of MRF with facades under these conditions. This information is vital for assessing the effectiveness of the façade in enhancing the overall structural integrity and safety of the building during seismic events.

EARTHQUAKE NAME	YEAR	COUNTRY	MAGNITUDE
SAN FERNANDO	1971	USA	6.6
FRIULI	1976	ITALY	6.5
IRIPINA	1980	ITALY	6.9

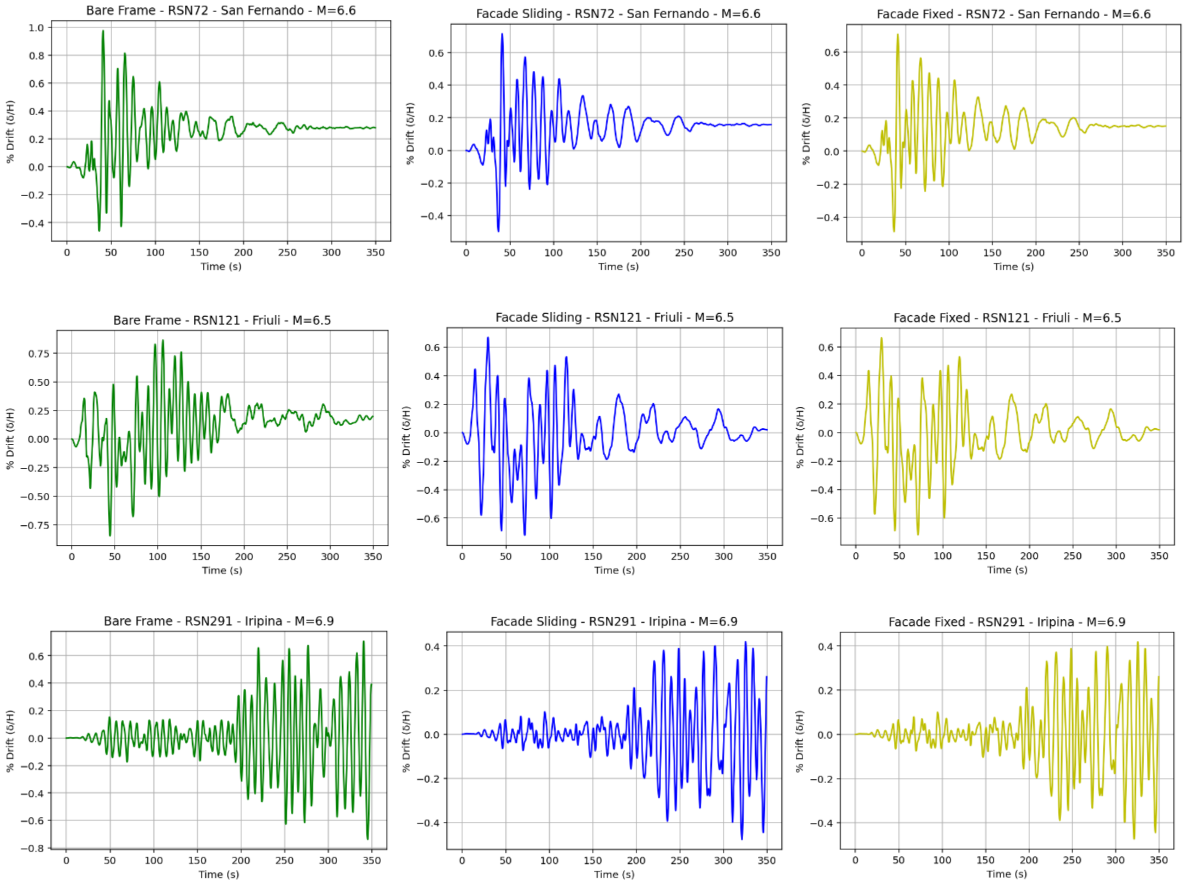
Table 6.4: Applied earthquakes data

The ground motions are obtained from the PEER ground motion database using the above-mentioned criteria. Initially, 3 ground motions were selected named San Fernando (RSN-72), Friuli (RSN-121), and Iripina (RSN-291) as shown in Table 6.4.1. Seism match is used for spectral matching to scale the selected ground motion according to our design spectrum of the site so that lateral loadings are within the limit for which the structure is to be designed. Graph 6.4.1,2 shows the original response spectrum of ground motions and the target response spectrum of the site.



Graph 6.4: Original and Target Response Spectrum

6.4.1 Plotted Graphs from Dynamic Analysis



Graph 6.4.2: Dynamic Analysis Results

6.4.2 Description and Interpretation of the Graphs

In section 6.4.1 of the study, several graphs were included to visualize the roof-drift ratio over time for moment-resisting frames (MRFs) with and without facades. The data specifically pertained to the San Fernando earthquake scenario. The results revealed notable differences in the maximum roof-drift ratio (%RDR) among the different frame configurations. The bare frame exhibited a significantly higher maximum roof-drift ratio of 0.975%, whereas frames with sliding and fixed facades showcased lower values of 0.714% and 0.708%, respectively.

During dynamic events such as earthquakes, the introduction of the facade plays a crucial role in reducing drift by increasing the overall stiffness and increasing the structural stability of the frame. The presence of the facade also affects the natural frequencies of the structures, possibly bringing them closer to the dominant frequencies of dynamic stresses. This alignment further contributes to frame performance by reducing roof deflection. Analyzing these diagrams and taking into account the effect of facades on the roof drift ratio, it is clear that including facades in the design results in more flexible structures that better withstand earthquake forces and maintain structural integrity.

In the Iripina earthquake scenario, nominal roof deflections were observed for extended durations, posing a potential risk to the overall structural stability of the framework. However, the inclusion of the facade in the design played a crucial role in increasing the rigidity and stability of the structure. The percentage of roof deflection (%RDR) was reduced to 0.47% As a result.

The facade itself provides inherent benefits to the structure, contributing to its overall resilience. A significant advantage is the ability of the facade to provide additional energy dissipation and attenuation. This means that the facade has the potential to reduce the effects of dynamic loads during seismic events. When a structure experiences seismic forces, the facade deforms and absorbs energy through a process called material hysteresis. This allows the facade to dissipate energy efficiently and reduce the overall impact on the structure. The design demonstrates an enhanced ability to resist seismic forces and maintain structural integrity by incorporating a façade with energy-dissipating and attenuating properties,

The facade's ability to absorb and dissipate energy helped to minimize the potential risks associated with roof drifts, thus contributing to the overall stability and safety of the structure during seismic events.

EQ NAME	**%RDRmax**	**Type of Frame**
RSN 72	0.975	Bare Frame
	0.714	Sliding Façade
	0.708	Fixed Façade
RSN 121	0.865	Bare Frame
	-0.721	Sliding Façade
	-0.718	Fixed Façade
RSN 291	-0.737	Bare Frame
	-0.477	Sliding Façade
	-0.473	Fixed Façade

Table 6.4.2 – Max Roof Drift

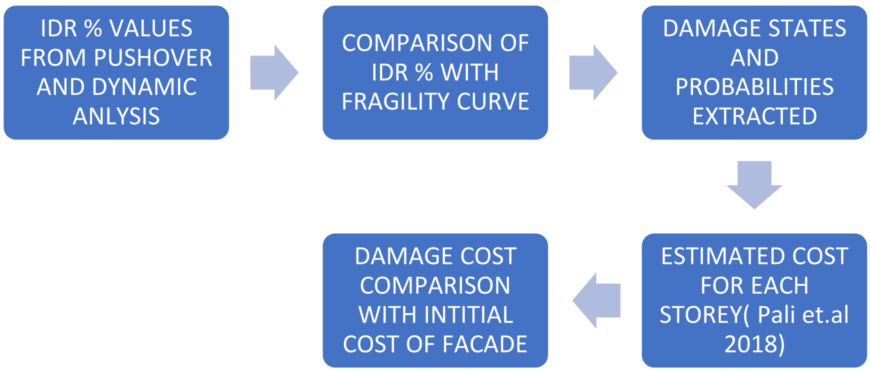
7. Damage Assessment

Damages developed in the façades were defined in terms of damage states. There are classified into three damage states (DS): [Pali et al.]

- The first level (DS1) is characterized by superficial wall damage and requires plaster, tape, and paint repair.
- Second (DS2) is marked by local damage to sheathing panels and steel frame components and necessarily requires removal and replacement of sheathing panels and local repair of steel frame components.
- The third DS (DS3) is marked by severe wall damage that requires partial or complete wall replacement.

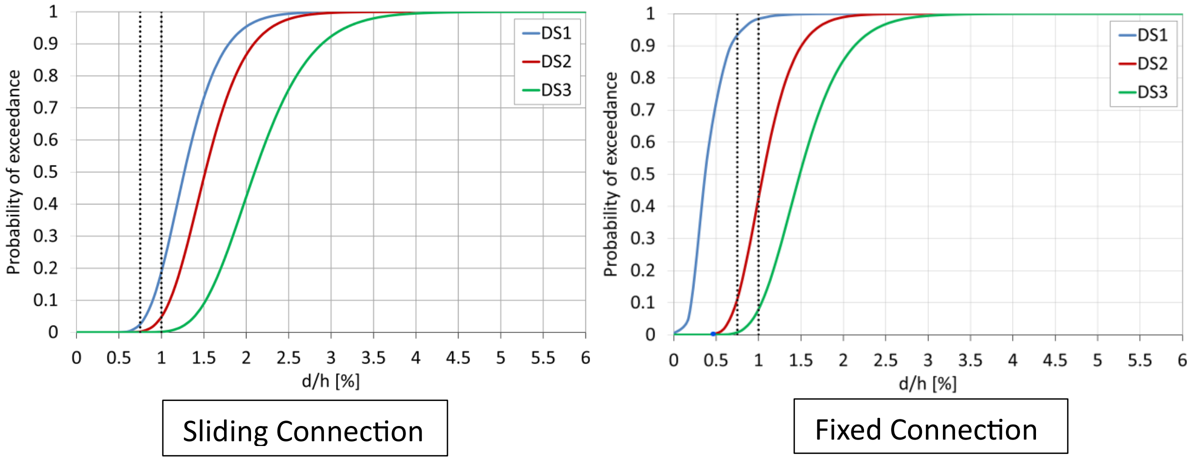
The triggering of each damage state in the facades is primarily determined by the Inter-Story Drift Ratio (IDR) due to their deformation-sensitive nature. To assess this, seismic fragility curves are employed for facades with fixed and sliding connections. The fragility data utilized in this assessment is sourced from Pali et al., as their experiments encompassed numerous specimens that were nominally similar. It is important to note that the type of connection employed has a substantial impact on the behaviour of the facades. The choice between a fixed or sliding connection significantly influences how the facades respond to seismic forces and the resulting damage states that may be triggered.

7.1 Methodology



7.2 Fragility Curves (Probability of Exceedance Vs IDR)

The fragility curves obtained from the fragility analysis serve as the basis for damage estimation. Fragility curves provide a probabilistic relationship between an intensity parameter (e.g., Story Drifts) and the likelihood of exceeding different damage scenarios. These curves are used to estimate the expected damage state according to the severity of the hazard. By applying the hazard intensity to the corresponding fragility curve, the probability of reaching each damage state can be determined, yielding an estimate of the magnitude of damage.



Graph 7.2: Fragility Curves of Façades

7.3 Damage Estimation

The cost associated with the damage that occurred in the façades is addressed up to a limited extent in the past. Different types of costs such as Initial Cost (IC) and replacement cost (RC) are calculated using manufacturers' support and market analysis. The formula for the cost of damage state (C_{DS}) is taken as a percentage (p) of overall repair cost (C). This percentage (p) for each damage state is given in the table below. The formula for the Calculation of Cost at the DS level:

$$C_{DS} = p\ (IC + RC)$$

DS	*p*	C_{DS}/Initial Costs
DS1	10% to 20%	0.19 to 0.38
DS2	20% to 50%	0.38 to 0.95
DS3	50% to 100%	0.95 to 1.90

Initial Cost (IC) = 1170.6 Rs/ft^2

Replacement Cost (RC) = 1059 Rs/ft^2

Total Façades per Story= 2

7.4 Damage Assessment Using Pushover Results

In the process of damage assessment using pushover analysis, the evaluation of facade damage involves analysing the extent and severity of damage based on the response obtained from the pushover analysis. This analysis includes extracting the Inter-Story Drifts (IDR%) for each story at the Peak and Yield points of the Pushover Curve for both sliding and fixed connections. These IDR values are then compared with fragility data to determine the probability of exceedance for achieving different damage states (DS).

By comparing the IDR values with the fragility data, the total number of facades at various damage levels can be determined, as presented in the tables. It is worth noting that the maximum Inter-Story drifts are observed at the 2nd story for both types of connections, at both the Yield and Peak points. Furthermore, it is evident that sliding facades exhibit relatively less damage compared to fixed facades. This difference in damage can be attributed to the higher capacity of sliding facades to resist lateral forces.

In the case of sliding connections, none of the facades in the entire frame are damaged at the yield point. This is due to the very low probability of exceedance for all five stories. However, the situation is completely different for fixed facades, with exceedance values ranging from 0.8 to 0.95. At the peak point, eight out of the ten employed sliding facades need to be replaced, while all the facades with fixed connections are completely damaged.

Table 7.4.3 presents the comparison of the damage cost to the initial cost of the facades for both sliding and fixed connections. The estimation of complete facade damage highlights that the addition of sliding connections can significantly reduce damage compared to fixed connections from 106% to 93% of initial cost and saves up to half a million rupees. This indicates that the sliding connection outperforms the fixed connection in terms of minimizing damage.

The initial cost of the façade for the entire 5-storey frame amounts to a significant sum of 3 million. However, when considering damages specifically in the case of fixed connections, the costs incurred reach approximately 1.1 million, which accounts for approximately 36% of the initial cost. On the other hand, sliding connections exhibit a notable advantage as they do not incur any associated damage costs. They demonstrate effective resistance to yielding base shear, thereby minimizing potential structural damages. This suggests that opting for sliding connections can prove beneficial in terms of cost savings and enhanced structural durability.

To summarize, the results show that sliding connections effectively prevent facade damage at the yield point, while fixed connections have a higher probability of facade damage. At the peak point, a substantial number of sliding facades require replacement, but all facades with fixed connections are completely damaged. Considering the damage cost compared to the initial cost, it is evident that adding sliding connections can greatly reduce damage and provide superior performance compared to fixed connections.

Façade with Sliding Connection										
Story	**1**		**2**		**3**		**4**		**5**	
Base Shear Points	Yield	Peak	Yield	Peak	Yield	Peak	Yield	Peak	Yield	Peak
IDR%	0.83	2.91	1.18	3	0.99	2.34	0.64	1.37	0.35	0.62
Damage State Achieved	1	3	1	3	1	3	1	1	1	1
Probability of Exceedance	0.1	0.9	0.35	0.92	0.18	0.68	0.04	0.6	0.01	0.04
No. of Damaged Facades	none	2	none	2	none	2	none	2	none	none

Table 7.4.1: Damage Assessment of Entire Frame with Sliding Façades

Façade with Fixed Connection										
Story	**1**		**2**		**3**		**4**		**5**	
Base Shear Points	Yield	Peak	Yield	Peak	Yield	Peak	Yield	Peak	Yield	Peak
IDR%	0.83	2.77	1.18	2.88	0.98	2.25	0.63	1.324	0.342	0.595
Damage State Achieved	1	3	2	3	1	3	1	2	1	1
Probability of Exceedance	0.95	0.94	0.6	0.98	0.98	0.9	0.82	0.79	0.5	0.8
No. of Damaged Facades	2	2	2	2	2	2	2	2	2	2

Table 7.4.2: Damage Assessment of Entire Frame with Fixed Façades

Damage Cost Summary		
Damage Cost in million Rs.	Sliding Façade	Fixed Façade
C_{DS} at Yield	0	1.1
C_{DS} at Peak	2.85	3.24
Initial Cost	3.05	3.05
Percent Damage Cost of the Initial Cost		
C_{DS} / IC (at yield)	0	36%
C_{DS} / IC (at peak)	93%	106%

Table 7.4.3: Damage Cost Summary using Pushover Results

7.5 Damage Assessment Using Time-History Analysis Results

Damage assessment using time-history results provides a more detailed and realistic understanding of the structural response under dynamic loading conditions compared to static methods like pushover analysis. Maximum IDR values for San Fernando (RSN 72) ground motions are extracted from Time-History Analysis. Associated damages and their costs are calculated using the same methodology and fragility data mentioned above.

Façade with Sliding Connection					
Story	**1**	**2**	**3**	**4**	**5**
IDR%	0.643	**0.974**	0.928	0.683	0.359
Damage State Achieved	1	1	1	1	1
Probability of Exceedance	0	0.25	0.22	0	0
No. of Damaged Facades	none	none	none	none	none

Table 7.5.1: Damage Assessment of Entire Frame with Sliding Façades

Façade with Fixed Connection					
Story	**1**	**2**	**3**	**4**	**5**
IDR%	0.645	**0.973**	0.916	0.671	0.35
Damage State Achieved	1	1	1	1	1
Probability of Exceedance	0.85	0.98	0.95	0.9	0.48
No. of Damaged Facades	2	2	2	2	2

Table 7.5.2: Damage Assessment of Entire Frame with Fixed Façades

Damage Cost Summary		
Damage Cost in million Rs.	Sliding Façade	Fixed Façade
C_{DS} at Peak	0	0.87
Total Initial Cost	3.05	3.05
Percent Damage Cost of the Initial Cost		
C_{DS} / IC (at peak)	0%	28%

Table 7.5.3: Damage Cost Summary using Time-History Results

A Peak IDR of 0.97 occurs in 2nd story showing the same trend as seen in pushover results. The probability of exceedance is negligible for sliding connection as none of the added façades show any considerable damage at this specific ground motion. Comparatively, all facades with fixed connections are partially damaged and the cost associated with damage reaches up to 28% of the initial cost as calculated in Table 7.5.3.

8. Conclusion

1. LGS Infills have proven to be more effective than Masonry Infills, due to their added stiffness which helps in reducing the Time Periods of buildings.

2. Adding facades to the MRF increases its lateral stiffness, improving its resistance to lateral loads. However, this also reduces the time period of the structure, regardless of whether the facades have fixed or sliding connections.

3. Facades delay the peak base shear and enhance the structure's lateral resistance, providing improved stability during seismic events.

4. Both sliding and fixed facades contribute to reducing maximum inter-story drifts over time compared to a bare frame, indicating improved structural performance.

5. Sliding facades exhibit better behaviour with reduced damage and a more gradual curve compared to fixed facades, indicating enhanced resilience.

6. Light gauge steel facades, with sliding connections, demonstrate lower probabilities of exceeding damage states, suggesting greater capacity to withstand seismic forces and minimize damage.

7. Cost analysis shows that sliding facades result in lower damage costs compared to fixed facades, making them a more cost-effective long-term solution.

In summary, the findings suggest that light gauge steel facades, particularly those with sliding connections, offer improved structural performance, reduced damage, and lower repair costs. These factors make them a promising choice for enhancing the resilience and cost-effectiveness of buildings in seismic-prone areas.

9. Recommendation

The main aim of this study was to explore how using non-structural façades in buildings affects their overall performance, with a specific focus on comparing the damages caused by sliding and fixed connections. To ensure more accurate results regarding structural performance and damages in sliding connections, researchers could consider implementing the distributed plasticity hinge approach or Finite Element Method (FEM). It is crucial to develop models that represent different building types and façade configurations in order to have a comprehensive understanding of the subject. Additionally, validating the findings using alternative software like ABBACUS would provide valuable insights into the strengths and limitations of the OPENSEES modelling approach.

In terms of managing risks, this approach could be a solid foundation as it eliminates the need for extensive physical experimentation. The practical implications of the research findings are particularly significant for insurance companies and real estate investors. Insurance companies can use the results to accurately assess risks and determine appropriate premiums for buildings. On the other hand, real estate investors can make informed investment decisions by taking into account the improved resilience and market value associated with properties that have these types of façades. Collaborative efforts between insurance companies and investors have the potential to drive risk mitigation strategies and encourage the adoption of sliding façades, leading to improved risk management and increased property value.

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