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EFFECT OF NON-STRUCTURAL ELEMENTS ON STRUCTURE AND DAMAGE ASSESSMENT USING NON-LINEAR STATIC AND DYNAMIC ANALYSIS

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Abstract

The façade is categorized as an architectural element and is not considered in the structural design, even though it could significantly influence the overall performance of the structure. This study examines the impact of conventional light gauge steel (LGS) façades with sliding and fixed connections to surrounding structure on the behavior of a case study steel frame building and evaluates the resulting damage. Two types of façades are considered: (1) a façade that has fixed connections at all sides and (2) a façade that has a sliding connection at the top and a fixed connection at the bottom. A two-dimensional, 5-story 3 Bay Moment Resisting Frame is modeled in Opensees. The behavior of the façade is modeled through a non-linear spring by using a Pinching4 material. Static and dynamic non-linear analyses are conducted, with, and without the façade, and the results are compared. Damage estimation is then done by comparing the results using fragility curves. The results showed that the presence of a façade increased the stiffness of the building and, thus, decreased drifts and time periods. Results from damage fragility analysis indicated that façades with sliding connections are less prone to damage as compared to façades with fixed connections.

Keywords: Performance-based seismic design, damage estimation, dry wall facades, non-structural elements, Moment Resisting Frame.

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1 INTRODUCTION

Non-structural elements in any building include partitions, façades, ceilings, electrical and mechanical systems. The damage to non-structural elements can significantly hamper the occupancy of a building and can create significant life safety risk in an earthquake. Non-structural elements contribute to 82% of investment cost for typical offices, and up to 92% in the case of hospitals [1]. Light gauge steel (LGS) stud framing has widespread use in the construction of residential, commercial and hospital buildings in many parts of the world. In the United States, approximately 60% of light gauge metal stud framing is non-structural [2]. Cases from past earthquakes have shown severe damage to nonstructural LGS elements. Damages such as cracking of gypsum boards, bending of studs, failure of connections between track and slab, and partial/complete failure of LGS façades and partitions were reported during these earthquakes [3].

There are two main types of connections for LGS façades to the surrounding structure: sliding or fixed. In a sliding connection, the bottom and sides of the LGS façade are fixed to the building structure while the top is allowed to slide in plane. The sliding connection allow to isolate façade from structural displacements. In a fixed connection, all sides of the façade are fixed to the surrounding structure. Fiorino et al. tested façades made with sliding and fixed connections under in plane cyclic and dynamic loading [4], [5], [6], [7] and evaluated the damage fragility against inter story drift limits proposed by Eurocode 8 [8], [9] for non-structural elements. Wang et al.[9] tested the shake shaking table on a five-story reinforced concrete structure with LWS façades. The tested façades were not infilled and were secured to the floor slabs using steel clips. Apart from these two studies, there have been no other experimental or numerical research completed on LGS façades so far. Additionally, studies focusing on the impact of LGS façades rather than LGS partitions [10] [11] on building structures remains scarce in the literature. Therefore, assessing the effect of façades on building performance becomes crucial. Furthermore, existing studies have not provided conclusive findings regarding repair cost estimation specifically for façades. To address this gap, one must consider design scenarios where façades are analysed alongside the building structure. This approach allows to estimate actual drifts and potential damages. In this context, the present study aims to achieve two main objectives: (1) understand the effect of LGS façades (with both types of connections) on the structural performance of a steel MRF and (2) Assess damage and repair costs for sliding and fixed connections of façades to determine which type of connection performs better in terms of both structural integrity and repair costs.

2 CASE STUDY FRAME

A residential structure with the main structural system as an Ordinary Moment Resisting Steel Frame (OMRF), is used as a case study building. The structure is five stories tall and has a 100 by 150 ft2 rectangular footprint. Columns are made of W24x131 section while W27x102 section was used for the beams. The steel material used for beams and columns in this study is 50ksi, with a yield strength (f_y) of 50 ksi and a tensile strength (f_u) of 65 ksi.

The vertical sections of the façade chosen in this study are shown in Figure 1. These facades are selected from the series of experiments conducted by the corresponding author [4], which involved cyclic testing on series of façade specimen with varying construction details. Two types of connections, sliding and fixed are used in this study. Fixed connections are rigid, preventing any movement laterally. Sliding connections allow lateral movements and reduce the impact of earthquakes.

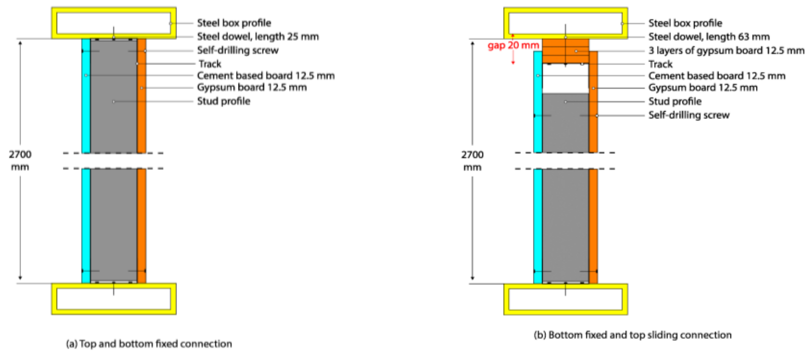

Figure 1 Vertical Section of façade with fixed and sliding connection [10]

3 MODELING AND ANALYSIS

OpenSees software is used to model the building frame. A two-dimensional non-linear model is created as a steel frame with plasticity concentrated at hinges near beam-column joints. Façades are incorporated in the model as spring lumped with the hysteretic response calibrated based on the experimental results [4]. A schematic representation of model is shown in Figure 2.

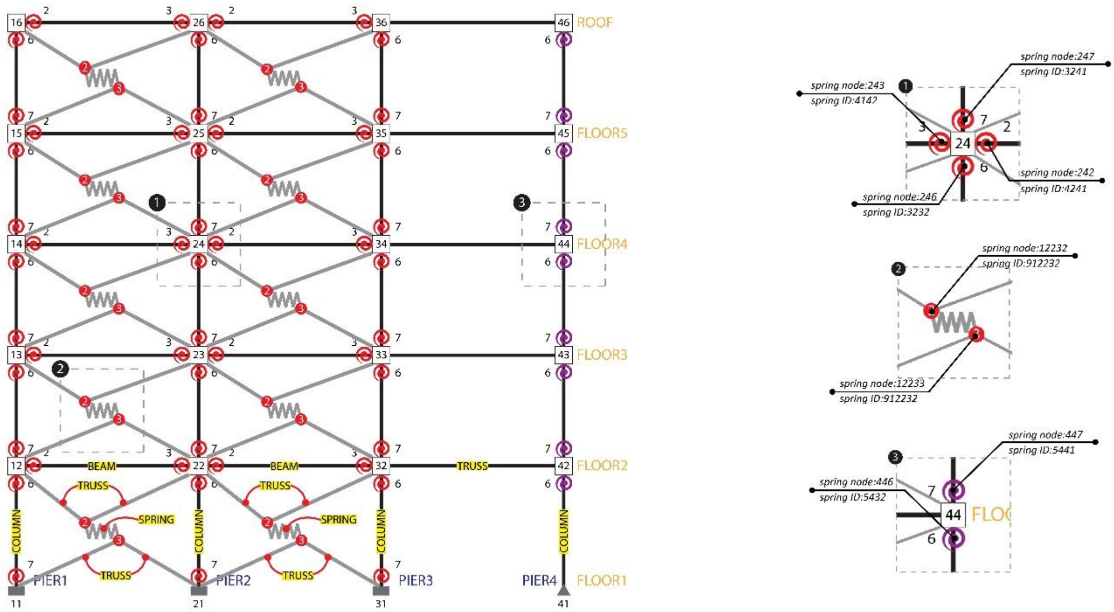

Figure 2 Schematization of model in OpenSees

W-shape beams and columns are modelled as beam column elements with elastic material properties. Beam and columns are connected using concentrate plastic hinges at their ends. The P-delta bay was modeled using rigid truss elements, where the column in this bay was pin-connected, while others were fixed. Concentrated Plastic Hinges (CPHs) are used in structural analysis to capture nonlinear behavior at the member level.

To incorporate the façade elements into the structural model, zero length spring elements connected at their ends to truss elements was used. Actual façade force-displacement envelope curves are assigned to springs as backbone curves that are connected by these truss elements with structure frame. Different backbone curves are used for sliding and fixed connections based on the experimental results [4]. The Façade Springs are modeled using the Pinching4 material [12], accurately capturing pinched load-deformation responses and degradation under cyclic loading. The site class was defined as SDC D, and the spectral acceleration values for long and short time periods were established as Sa=0.38 and S1=1.3 for a high seismic risk site (Islamabad, Pakistan).

To better understand the behavior of the model used in this study under dynamic loading, time history analysis was conducted. The ground motions are obtained from the PEER ground motion database. 3 ground motions were selected named San Fernando (RSN-72), Friuli (RSN-121), and Irpinia (RSN-291). Spectral matching was done to scale the selected ground motion according to design spectrum.

4 RESULTS AND DISCUSSION

During the pushover analysis, the lateral load is applied incrementally till the 5% displacement target is achieved. The limit state of collapse prevention is reached when the structure attains a 5% drift threshold. The value for the design base shear calculated using the Equivalent Lateral Force method was 546.3 kips. In Figure 3, we can observe a comparison of pushover curves for different frames.

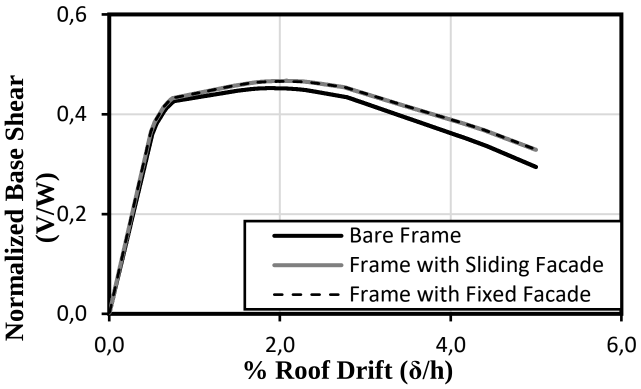

Figure 3 Comparison of Pushover Curves

The results demonstrate that an ordinary moment resisting frame (OMRF) with either a fixed or sliding facade exhibits better performance than the bare OMRF frame. 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. The addition of a 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 base shear is not significantly affected by the change in connection type. However, base shear on the bare frame is 15 kips lower than the frame with the facade attached.

Non-linear dynamic analysis results show notable differences in the maximum roof-drift ratio (RDR) among the different frame configurations. The bare frame exhibited a significantly higher peak roof-drift ratio of 0.98%, whereas frames with sliding and fixed facades showcased lower values of 0.71% in case of San Fernando Earthquake. The introduction of facades played a crucial role in reducing drifts by enhancing the overall stiffness during dynamic events like earthquakes. The facade itself offered inherent benefits to the structure, contributing to its overall resilience.

5 DAMAGE ASSESSMENT

In order to assess the level of physical damage in façades and estimate the extent of damages in them, the drifts obtained from the analysis shown in previous sections are correlated with the damage fragility data. Damages developed in the façades are defined in terms of damage states and are classified into three damage states (DS) based on the work of Pali [13]. 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 curves utilized in this study are used from a Previous study on drywalls by Pali [13].

The cost associated with the damage that occurred in the façades is addressed up to a limited extent in the past [7]. 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 (CDS) is taken as a percentage (P) of overall repair cost (C) shown in Table 1. This percentage (p) for each damage state is given in the table below. Initial cost (IC) are estimated to be 1170 PKR /ft2 while replacement cost (RC) are estimate around 1060 PKR. /ft2. The damage cost are estimated based on two façades per storey in case study model, The formula for the Calculation of Cost at the DS level: CDS = P (IC + RC).

DS	P	CDS/Initial Costs
DS1	10%-20%	0.19 to 0.38
DS2	20%-50%	0.38 to 0.95
DS3	50%-100%	0.95 to 1.90

Table 1 Cost at Various DS levels [10]

By comparing the IDR values with the fragility data, the total number of facades at various damage levels can be determined. 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 are damaged and need to be replaced, while all the facades with fixed connections are completely damaged. 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 save 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 effectively resist yielding base shear, thereby minimizing potential structural damage. This suggests that opting for sliding connections can prove beneficial in terms of cost savings and enhanced structural durability with minimal investment on making connections sliding.

6 CONCLUSIONS

This study examines the impact of conventional light gauge steel (LGS) façades with sliding and fixed connections to surrounding structure on the behavior of a case study steel frame building and evaluates the resulting damage under seismic design situation. A 5-story 3 Bay Moment Resisting Frame is modeled in OpenSees. The behavior of the façade is modeled through a non-linear spring by using a Pinching4 material. Static and dynamic non-linear analyses are conducted, with, and without the façade, and the results are compared. The results show that the facades made of light gauge steel, especially those with sliding connections, would offer minimum damage during seismic events with much less repairing cost. The addition of façades, regardless of connection technique, greatly increases the rigidity of MRFs. Both sliding and fixed façade connections reduce peak roof drift ratios during seismic events, demonstrating their importance in increasing structural resilience.

Sliding façades encountered limited damage during, resulting in significantly lower repair costs. Fixed connections are more likely to sustain significant damage, as seen by fragility curve comparisons and dynamic analysis results. Incorporating sliding connections in façade design proves to be more cost-effective. Façades with sliding connection have much lower total repair costs than fixed façades. During dynamic analysis, none of the facades with sliding connections are damaged which demonstrates the economic viability of sliding connections under real world scenarios.

7 REFERENCES

[1] A. K. Kazantzi, D. Vamvatsikos, and E. Miranda, Evaluation of Seismic Acceleration Demands on Building Nonstructural Elements, *Journal of Structural Engineering*, vol. 146, no. 7, Jul. 2020, doi: 10.1061/(ASCE)ST.1943-541X.0002676.

[2] JoséI. Restrepo and A. F. Lang, Study of Loading Protocols in Light-Gauge Stud Partition Walls, *Earthquake Spectra*, vol. 27, no. 4, pp. 1169–1185, Nov. 2011, doi: 10.1193/1.3651608.

[3] C. Jenkins, S. Soroushian, E. Rahmanishamsi, and E. M. Maragakis, Experimental fragility analysis of cold-formed steel-framed partition wall systems, *Thin-Walled Structures*, vol. 103, pp. 115–127, Jun. 2016, doi: 10.1016/j.tws.2016.02.015.

[4] L. Fiorino, S. Shakeel, A. Campiche, and R. Landolfo, In-plane seismic behavior of lightweight steel drywall façades through quasi-static reversed cyclic tests, *Thin-Walled Structures*, vol. 182, p. 110157, Jan. 2023, doi: 10.1016/j.tws.2022.110157.

[5] S. Shakeel, R. Landolfo, and L. Fiorino, Behaviour factor evaluation of CFS shear walls with gypsum board sheathing according to FEMA P695 for Eurocodes, *Thin-Walled Structures*, vol. 141, pp. 194–207, Aug. 2019, doi: 10.1016/j.tws.2019.04.017.

[6] V. Macillo, A. Campiche, S. Shakeel, B. Bucciero, T. Pali, M.T. Terracciano, L. Fiorino, and R. Landolfo, Seismic Behaviour of Sheathed CFS Buildings: Shake-Table Testing and Numerical Modelling, *Key Eng Mater*, vol. 763, pp. 584–591, 2018.

[7] R. Landolfo, T. Pali, B. Bucciero, M.T. Terracciano, S. Shakeel, V. Macillo, O. Iuorio, and L. Fiorino, Seismic response assessment of architectural non-structural LWS drywall components through experimental tests, *J Constr Steel Res*, vol. 162, p. 105575, 2019.

[8] L. Fiorino, A. Campiche, S. Shakeel, and R. Landolfo, Seismic design rules for lightweight steel shear walls with steel sheet sheathing in the 2nd-generation Eurocodes, *J Constr Steel Res*, vol. 187, p. 106951, Dec. 2021.

[9] CEN, *EN 1998-1 Eurocode 8: Design of Structures for earthquake resistance-Part 1: General rules, seismic actions and rules for buildings*. Brussels: European Committee for Standardization, 2004.

[10] A. Poursadrollah, R. Tartaglia, L. Fiorino, S. Shakeel, and R. Landolfo, Effect of lightweight steel partitions on seismic behaviour of moment resisting frames, *J Constr Steel Res*, vol. 206, p. 107925, Jul. 2023.

[11] L. Fiorino, S. Shakeel, V. Macillo, and R. Landolfo, Seismic response of CFS shear walls sheathed with nailed gypsum panels: Numerical modelling, *Thin-Walled Structures*, 2017.
[12] S. Mazzoni, F. McKenna, M. H. Scott, and G. L. Fenves, OpenSees, 2009, *Berkeley*.
[13] T. Pali, V. Macillo, M. T. Terracciano, B. Bucciero, and R. Landolfo, In - plane quasi - static cyclic tests of nonstructural lightweight steel drywall partitions for seismic performance evaluation, *Eq. Eng. and Str. Dynamics*, no. January, pp. 1566–1588, 2018.