The Influence of Gravity-Only Framing on the Performance of Steel Moment Frames

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SUMMARY:

The gravity framing systems in structures are typically neglected in lateral-resisting design. However, it has been recognized that every connection has a certain amount of resistance. This additional stiffness and strength can be utilized in design by modeling the gravity connections as partially restrained connections, instead of the typical pin connections. In this study, the performance of a steel special moment resisting frame (SMRF) without the lateral contribution from the gravity system is compared with the performance where the gravity system is explicitly modeled using PR connections with varying levels of strength. The performance of the systems are assessed using different methods, including analysis at serviceability, immediate occupancy and life safety limit states, collapse analysis, and risked based analysis. For all metrics used, the systems with out the inclusion of the gravity system.

Keywords: Partially Restrained Connection, Steel Moment Frame, Performance Based Earthquake Engineering

1. INTRODUCTION AND BACKGROUND

Traditional analysis dictates that the capacity of special moment resisting steel frames to resist seismic loads is given solely by the frames that have moment connections that are specifically detailed for seismic resistance. The remaining frames, called the gravity system, are assumed to only resist the gravity loads.

However, according to ASCE 7-10 (ASCE 2010), a building designed without any seismic detailing has a seismic reduction factor coefficient, R, equal to 3. This is because the gravity system has a certain amount of stiffness and strength given by the connections, which behave as partially restrained (PR) connections. This is a fact that was proven during the Northridge Earthquake in 1994 when many special moment connections presented brittle failures during the earthquake. However, the buildings with the failed connections did not collapse, partially because the gravity system provided some seismic resistance (Lui and Astaneh-Asl 2000, Green et al. 2004).

PR connections represent the true behavior of gravity framing connections and can be classified as rigid, intermediate, or flexible according to FEMA 355D (FEMA 2000). The main drawback of including the behavior of these connections in the analysis is that PR connections are difficult to model, and more research is still required to fully understand their full behavior.

Since it is believed that the gravity system will contribute to the performance of the building at a variety of limit states, the objective of this project is to compare the seismic response of a bare special moment resisting steel frame with one that includes the behavior of the gravity system by using PR connections. In order to get a better insight of the influence of the connections, different levels of strength capacity for the PR connections are used. The comparison is quantified by multiple analyses

at varying levels of ground motions, including those associated with serviceability, immediate occupancy and life safety limit states and collapse.

Currently, the FEMA P-695 Methodology (FEMA 2009) that is used to investigate full performance groups does not allow the gravity system to be included in the assessment of performance. However, Appendix F of P-695, applicable to the performance assessment of a single structure, does allow the inclusion of the gravity system. Similarly, the PEER Tall Building Guidelines (2010) recommend including the gravity system in the analysis.

1.1. Building Overview

The building used for the comparison is taken from one of the examples illustrated in FEMA P752, *NEHRP Recommended Provisions: Design Examples* (FEMA 2012, anticipated). The example used is an office building located in Los Angeles, California. This seven-story building of rectangular plan configuration is 177 feet, 4 inches long in the east – west direction and 127 feet, 4 inches wide in the north – south direction. It is framed in structural steel with 25-foot bays in each direction. The typical story height is 13 feet, 4 inches, with the one exception being the first story, which is 22 feet, 4 inches high. Floors consist of 3-1/4-inch lightweight concrete over metal deck, and the elevators and stairs are located in the central three bays.

Seismic force resistance is provided by special moment frames (SMF) with prequalified Reduced Beam Section (RBS) connections located on the perimeter of the building. There are five bays of moment frames on each line. Loads are provided according to ASCE 7 (ASCE 2010), and the seismic design and detailing is in accordance with the AISC Seismic Provisions (AISC 2005).

Even though traditional design and analysis includes the effect of the gravity system from a stability point of view (P-Delta effects), it does not include the contribution of the gravity system as part of the lateral resisting system, since they are typically modeled as pinned connections. For this project, the gravity connections are modeled as partially restrained connections, which accounts for the stability, as well as the stiffness and strength in every connection.

For this building, there are six frames in each direction: two special moment frames and four gravity frames. The architectural layout (e.g. bay spacing) of the gravity system is the same as the special moment resisting frame; however, the section sizes are designed just for the gravity loads. These section sizes are modeled and designed using SAP2000 (Computers and Structures 2000). Beams are designed as composite, engaging the concrete of the slab.

2. MODELING METHODOLOGY

2.1 Special Moment Resisting Frame Model

In order to analyze the structure at all possible limit states, nonlinear behavior of the structural system has to be included. All the analyses are performed using the nonlinear dynamic analysis program, OpenSEES (OpenSEES 2012). After the design is performed and the sizes of the gravity sections are obtained from SAP2000, the hysteretic behavior for any component that could behave in an inelastic fashion is determined.

The nonlinear components that are modeled in the special moment frames are shown in Figure 2.1 and include the plastic hinges located at the reduced beam section, the panel zones, the column bases, and the plastic hinges at the extremes of the columns that have moment resisting connections.



Figure 2.1. Special Moment Resisting Frame Model

The hysteretic behavior of the reduced beam sections (RBS) is modeled using the Modified Ibarra Krawinkler (Lignos and Krawinkler 2010) deterioration model in OpenSEES (Bilin material). An example of this type of hysteretic behavior for the moment-rotation of a beam is shown in Figure 2.2.



Figure 2.2. Modified Ibarra Krawinkler Deterioration Model

Panel zones are included in the special moment steel resisting frame. The model that is used to predict the hysteretic behavior of these panel zones is the Krawinkler model (Charney and Marshall 2006). This model is formed by 8 elements, 12 nodes and 2 rotational springs. The two rotational springs of this model represent panel zone shear yielding and column flange flexure yielding. These two behaviors result in a tri-linear force displacement relationship.

Plastic hinges at the columns could be modeled in two ways: using fiber elements on the entire column or using a phenomenological model at the extremes of the column. Both methods have their benefits and problems. The fiber elements method can predict the nonlinear behavior and includes the axial load interaction. Phenomenological models can predict the nonlinear behavior, including the deterioration of the section, but cannot include the axial interaction.

Between the two methods, the phenomenological model is chosen for the columns of the moment frame where the beams frame into the strong axis of the column. This model was chosen because of the model's capacity to predict deterioration, which means the axial interaction behavior of the column is sacrificed in these locations. Although the phenomenological model is used for columns of the moment frame where the beams frame in the strong axis of the column, the fiber section model is used for columns of the moment frame where the beams frame in the weak axis of the column, which occurs at the corner columns of the moment frames used in this project. This way, the complete nonlinear behavior will be captured in case any yielding occurs in the columns. The hysteretic behavior of the columns that use the phenomenological model is modeled using the Modified Ibarra Krawinkler deterioration model in OpenSEES, which is the same material used for the RBS, while the columns with fibers use an elasto-plastic material.

2.2 Gravity System Model

For the gravity system, the columns are modeled using fiber sections to check for any nonlinearity. In order to capture the full nonlinear behavior of the gravity system, the partially restrained connections are also modeled assuming the strength of partially restrained connections is lower than the plastic capacity of the beam it is connecting. Modeling PR connections can be cumbersome (Rassati et. al. 2004) depending on the level of detail required in the analysis, so for this analysis, a simple model given by ASCE 41-06 (ASCE 2007) is used to represent the moment rotation relationship of these connections, as shown in Figure 2.3.



Figure 2.3. Moment – Rotation for PR connections (ASCE 2007)

This simple model was obtained from experimental data and varies depending on the PR connection. The parameters required to define the moment rotation curve, besides the ones shown in Figure 2.3, are the moment capacity of the connection (Q_{CE}) and the initial stiffness of the connection (K_{CE}). The main principle of PR connections is that they have strength that is a percentage of the strength of the connected beam. Therefore, the capacity of the PR connection (Q_{CE}) is determined as a percentage of the plastic moment of the beam. The initial stiffness depends on the moment capacity of the connection and is taken as Q_{CE} divided by the yielding rotation of the connection, which is assumed to be 0.003 radians. The rest of parameters of Figure 2.3 are given in ASCE 41-06 Table 5-6 (ASCE 2007) and depend of the type of connection. For the analysis, the "top and bottom clip angle connection" is chosen, and it is assumed that the connection angles fail due to flexure.

In order to get an understanding of the influence of the partially restrained connections, different percentages are considered. Moreover, partially restrained connections that frame into the weak axis of the columns are taken into account by also assigning them a percentage of beam strength. The nomenclature used to differentiate the strength of the connections is SMRF_M_{CE}_WA, where M_{CE} is the percentage of beam strength assigned to connections framed into the column strong axis, and M_{CE} WA is the percentage of beam strength assigned to connections framed into the column weak axis. The gravity system has two different frame configurations, which are shown in Figure 2.4. The first configuration has all the beams framing into the weak axis of the columns but has the beams framing into the strong axis of the interior columns.



Figure 2.4. Gravity Frame Configurations

2.3. Modeling Assumptions

The building is modeled in 2D, as shown in Figure 2.5. Since there are two moment frames and four gravity frames in each direction, two gravity frames are analyzed with one moment frame.



Figure 2.5. Full model including moment frame and gravity systems

Besides the assumptions described earlier regarding the modeling of the nonlinear behaviour, a few more modeling assumptions must be mentioned. For these analyses, floor constraints among frames are used in order to perform this 2D analysis, and the columns at the base of the gravity system are assumed to be pinned. Also, for the analyses where the influence of gravity framing is not included, no strength or stiffness is assigned to PR connections.

3. PERFORMANCE EVALUATION METRIC

Multiple methods are used to assess the performance of the different systems. Three different methodologies comprise this performance metric: a limit state methodology, a collapse methodology and a risk-based methodology. The limit state methodology presents the results in terms of physical behavior of the systems (e.g. drift ratio) at different limit states, which are defined by the levels of ground motion that would be experienced. This methodology could allow the designer to better understand the physical behavior of the structure, which aids in making design changes to ensure the structure behaves satisfactorily at numerous levels of earthquakes. The collapse methodology provides information on the probability of collapse of a structure and the levels of ground motion needed to cause this collapse. The risk-based methodology presents the results in terms of consequences (dollars, deaths and downtime), which combines all potential behaviors into total values that are useful for the decision making process of the structure's stakeholders.

3.1. Limit State Based Analysis

The limit state based analysis for this project investigates the behavior under three different levels of ground motion, which include the 43 year mean return interval (MRI), the design basis earthquake (DBE), and the maximum considered earthquake (MCE). The 43 year MRI corresponds to a ground motion with 50% probability of occurring in 30 years, the MCE corresponds to a ground motion with 2% probability of occurring in 50 years (which has a 2475 year MRI), and the DBE corresponds to a ground motion that is two-thirds of the MCE.

The smallest level of ground motion, the 43 year MRI limit state, is commonly used as the seismic serviceability performance level (PEER 2010). The DBE and the MCE levels of ground motion are used to examine behavior at the life safety and collapse prevention limit states, respectively. For these three analyses, ground motion scaling and selection is performed under the ASCE 7 (ASCE 2005) guidelines.

For the DBE and MCE level ground motions, the spectrum is developed from the ASCE design response spectrum (ASCE 2005). Because the response spectrum of the 43 year MRI is not prescribed

in ASCE 7, its uniform hazard response spectrum is determined from hazard information provided by the U.S. Geological Survey (USGS 2008). The drift ratios for each level of ground motion are calculated and compared to determine how the gravity system affects the behavior of the structure for the different limit states.

3.2. Collapse Analysis

For the collapse analysis, the P-695 methodology (FEMA 2009) and its companion software, the P-695 Analysis Toolkit (NIST 2012), are utilized. The P-695 process of developing collapse fragilities and collapse margin ratios (CMR) is used in this analysis to compare performance under collapse level ground motions. Although not explicitly discussed in the P-695 method, the probabilities of collapse are also determined for each analyzed system.

The collapse fragilities are created using a lognormal distribution defined by the median collapse intensity (the acceleration where half of the ground motions have caused collapse of the structure) and the record-to-record dispersion, β_{RTR} (0.4 for all systems, since their period ductilities are all greater than 3.0). Also presented are the CMRs, which represent the scaling needed on the design MCE spectral acceleration to cause collapse. Therefore, as the CMR increases, the probability of collapse decreases. These probabilities of collapse are defined as the compliment of the lognormal distribution of the acceptable collapse margin ratios (ACMR), defined by a λ of zero and a β equal to the total dispersion of the system (β_{TOT}). For this project, β_{TOT} was equal as 0.52915, as the design requirement dispersion (β_{DR}), test data dispersion (β_{TD}), and modeling dispersion (β_{MDL}) were all taken as "B" (FEMA 2009).

3.3. Risk-Based Analysis

For the risk-based analysis, the results are put in terms of consequence, instead of the physical behavior of the structure, and the ATC 58 methodology (ATC 2011), and its companion software, PACT (ATC 2011), are used. This methodology quantifies the performance measures in terms of casualties, repair cost, repair time, and unsafe placards. This methodology also includes the ability to integrate fragility with hazard, which determines the results by including the conditional probability that the level of ground motion used in analysis actually occurs at a given site. This analysis methodology determines these consequences on a component, or fragility group, basis.

Many structural and nonstructural component fragilities have been developed and are included in the PACT program, and for the purpose of this project, only fragilities already developed are used. The fragilities used in this analysis include the reduced beam section steel connections, the column base plate connections, curtain walls, steel stairs and internal wall partitions. While this obviously does not include all the components of a typical commercial building, it includes effects from both structural and nonstructural components, and the results are used to compare the general behavior of the two building types, as opposed to truly predicting the full potential consequences of the entire structure.

The time based assessment option is how the methodology integrates fragility with hazard and is the assessment type used in this performance metric. This assessment runs numerous nonlinear dynamic analyses at varying intensities and finds a weighted average of the results based on the hazard. This option then assesses the probable consequences of a building's response to earthquake shaking, based on the component fragilities and the results from the intensity analyses. The consequences, although incomplete for a whole building, give a good comparison between the systems from a risk perspective.

4. RESULTS AND DISCUSSION

4.1 Pushover Results

Several nonlinear static pushover analyses using different PR strength combinations are performed to obtain the influence of the gravity system on the strength and stiffness of the building. Figure 4.1 shows the pushover results, comparing the system with no gravity effect included to the systems that have varying levels of strength in the partially restrained connections.



Figure 4.1. Pushover curve for numerous systems

As shown in Figure 4.1, the strength and stiffness of the systems increase as the strength of the partially restrained connections increase. This is especially true as more strength is added to both the connections at the strong axis of the columns and the connections at the weak axis of the columns.

4.2. Limit State Results

For the limit state analysis, the results are presented by comparing drift ratios. The median drift ratio from the analysis of seven ground motions chosen from the P-695 Far Field set is given in Table 4.1 for two levels of gravity system inclusion: the SMRF_0_0 and the SMRF_30_30. For the model where the gravity system's effect is included, Table 4.1 also includes the percent decrease in drift ratio when compared to the drift ratios calculated when the gravity system's effect is not included.

SMRF_0_0	SMRF_30_30	
Drift Patio	Drift	Percent
Dinit Katio	Ratio	Decrease
0.00659	0.00502	23.8
0.01800	0.01511	16.1
0.02428	0.02045	15.8
	SMRF_0_0 Drift Ratio 0.00659 0.01800 0.02428	SMRF_0_0 SMRF Drift Ratio Drift Ratio 0.00659 0.00502 0.01800 0.01511 0.02428 0.02045

Table 4.1. Median drift ratios for the limit state analysis

As expected, the drift ratios increase as the level of ground motion increases from the 43 year MRI to the DBE to the MCE. For the SMRF_30_30, the median drift ratios decreased by an average of 18.6% when compared to the SMRF_0_0. The largest influence was seen at the 43 year MRI level of ground motion, which had a percent decrease in the drift ratio of nearly 24 percent.

4.3. Collapse Analysis Results

This section presents an example of the IDA analysis, the collapse fragility curves, the CMR results of the P-695 analysis, and the probability of collapse for all the analyzed structures.

To calculate the CMRs, IDAs are performed until at least half of the ground motions have caused collapse. The acceleration where this occurs is defined as the median collapse intensity. The ratio of the median collapse intensity to the design MCE spectral acceleration is the CMR. An example of one of the IDA analyses, marked with the CMR, is shown in Figure 4.2 for the system with 30% gravity in both directions.



Figure 4.2. Example of an Incremental Dynamic Analysis Results

Figure 4.3 shows the fragility curves, as well as the corresponding CMRs and probabilities of collapse, for varying percentages of strength in the connections.



Figure 4.3. Collapse fragility curves, CMRs and probability of collapses for the varying gravity systems

As the fragility curves have a higher CMR (they are further to the right in Figure 4.3), their probability of occurrence for the same ground motion decreases, which decreases their overall probability of collapse. All of the median collapse intensities (and therefore CMRs) stay the same or increase as further gravity framing influence is added. From the lowest CMR (SMRF_0_0) to the highest (SMRF_70_70), the percent increase was 14.3%.

However, the probabilities of collapse have negligible change, due to the fact that the probability of collapse is already very low for the system where gravity framing strength and stiffness is not included in the analysis. To see how influential this inclusion of the gravity system strength and stiffness could be if the structure was worse off to begin with, the systems are analyzed with higher design ground motion. The design MCE spectral acceleration is scaled up, and the new CMRs and probabilities of collapse are determined. As the design spectral acceleration increases, the CMR decreases for the same median collapse intensity, which increases the ACMR and therefore, the probability of collapse. As shown in Figure 4.4, the gravity system has significantly more influence as the probability of collapse increases for the system without the effect of the gravity system included.



Figure 4.4. Comparison of gravity system benefit on the probability of collapse

In other words, the weaker a system is initially, the more benefit will come from including the strength and stiffness of the gravity system. Although these ground motions are unrealistically high, this result is included primarily to show the trend of improvement from the gravity system.

4.4. Risk Based Results

In order to determine the effect that the inclusion of the gravity system has on risk, two models were analyzed in PACT: the SMRF_0_0 and the SMRF_30_30. As an example of the PACT risk outputs, Figure 4.5 shows the repair costs for the two models analyzed. These plots show the total risk (the weighted average of the result from each intensity), as well as the contribution from each intensity, represented by the colored sections.



Figure 4.5. PACT Risk annualized repair costs (L: SMRF 0 0; R: SMRF 30 30)

For the annualized repair costs, the addition of the contribution of the gravity system decreased the risk by 25 percent. Additional connection strengths should be analyzed to determine possible trends in risk reduction, but this initial analysis provides proof that these systems could be benefical in reducing the risk calculations of moment frame systems.

5. CONCLUSIONS

While the process of including the contribution to the seismic resistance from the gravity systems may add a significant amount of complexity to the modeling procedure, the results of the analysis reported herein show that the inclusion of the gravity system strength and stiffness can have significant effects on the response of the structure, especially as the systems without the gravity included become weaker initially. As anticipated, the performance computed for the system with the gravity framing included in the mathematical model is better than the computed performance with the gravity system not included, which has been quantified using a variety of different metrics in this report. Whether the focus of the design is to improve a structure's behavior at low, moderate or high levels of ground motion, the inclusion of the contribution of the gravity system modeled as partially restrained connections improves the accuracy of the model. From this improvement in the model accuracy and variety of analysis metrics, the true behavior of the structure and the levels of potential risk involved could be significantly more understood, and these performance metrics can be expanded to numerous other lateral-resisting systems to determine behavior, with or without the gravity system resistance.

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