The Influence of Gravity Column Continuity on the Seismic Performance of Special Steel Moment Frame Structures

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Abstract

This paper investigates the influence of the stiffness and strength of gravity columns on the collapse performance of buildings that utilize steel special moment frames as the main seismic lateral load resisting system. The research centers on the development of a simple lumped column approach, where all the gravity columns are combined into one elastic, continuous, and pinned-base element. Once this approach was validated, two, four and eight story buildings that were originally analyzed without the influence of gravity columns in the ATC 76-1 project were re-analyzed including gravity columns. The FEMA P-695 methodology was used to assess the collapse performance. The effect of having a different number of gravity columns and different column sections was modelled by varying the moment of inertia of the lumped gravity column. The results show that the gravity columns have a profound influence on the collapse performance of buildings, especially for taller structures. Moreover, it was found that the correlation between the results of the nonlinear static analyses and the nonlinear dynamic analysis, based on collapse evaluation, provides insight into the choice of an optimal section for the lumped column. This means, in turn, that the designer can determine the number and the size of gravity columns that are required to significantly improve the building's collapse performance.

Keywords: Steel Special Moment Frames, Gravity Framing, Gravity Columns, Column Splices, Continuous Stiffness, Seismic Performance, Collapse Probability.

1. Introduction

While it has been suggested that the strength and stiffness of the gravity system enhances the performance of Steel Special Moment Frames (SFMs) when they are subjected to high intensity ground motions, the gravity system is typically not included in the design and subsequent performance assessment of the structure. The main reason the gravity system is not considered is the difficulty of modeling the beam-to-column connections, and the lack of information regarding the expected behavior of these connections. As a result it is not generally practical to develop a "full system" mathematical model that includes the stiffness and strength of the numerous beams, columns, and beam-to-column connections in the gravity system. However, studies have shown that the effect of the gravity columns alone (with no strength assigned to the gravity beam-to-column connections) is beneficial to the seismic performance of SMF systems. The benefit is principally manifested by the delay or prevention of the formation of story mechanisms that lead to premature collapse. In order to provide some quantification of the influence of gravity columns and to avoid complex analysis, this paper introduces a "Lumped Column" approach wherein a single column is used to represent all of the gravity columns in the structure. An important consideration in the assessment of the contribution of the gravity columns is the continuity of the columns along the height of the structure, which is a function of the stiffness of the columns and the location, stiffness, and strength of splice connections. Where sufficient continuity is provided, the Lumped Column approach acknowledges that: "If gravity columns are continuous over several stories of a structure, then the stiffness of the gravity columns will limit the amount of drift concentration that can occur" [1].

The Lumped Column approach is demonstrated through the analysis of several SMFs that were originally developed for use in the ATC-76-1 project [2]. Before this is done, some background is provided on the influence of the gravity system on the seismic performance of steel structures, and more specifically on the influence of the columns of the gravity system.

Tremblay and Stierner [3] proposed the use of continuous columns as a backup stiffness to mitigate P-Delta effects in braced frames. The study used nonlinear dynamic analyses to investigate the influence of the continuous columns on the collapse of eight buildings that varied in height from two to twelve stories. The studied conditions were: columns with moment releases at the top and bottom of each floor (no continuous stiffness was provided),

moment releases at the top and bottom of pairs of contiguous stories, and continuous over the entire building height. The last case was analyzed using pinned connections at the base of the columns and was repeated for fixed connections at the base of the columns. The results showed that the worst case (more collapses occurred) was when columns had moment releases at each level (no continuity) and the best case was when the columns were continuous over the entire height and fixed at the base. The authors concluded that the continuous stiffness concept is very attractive to mitigate P-Delta effects because it increases the post-yielding stiffness of the structure, but more research is required.

An analogous investigation was carried on by Qu et al. [4] where the influence of continuous column stiffness was studied by adding a rocking wall frame to a ductile reinforced concrete moment frame. The results showed that interstory drift concentrations are reduced considerably when the rocking wall was incorporated. This was a supporting study for the retrofit of a 11-story reinforced concrete frame on the campus of the Tokyo Institute of Technology [5].

Tagawa [6] studied the continuous column effect on steel moment resisting frames when gravity columns are included in the analysis. Nonlinear static and dynamic analyses were performed using different buildings with and without the gravity columns. The results showed that the post-yield stiffness increased in pushover response and that interstory drift concentrations (large drifts in one story relative to an adjacent story) predicted by the dynamic analysis were reduced when the gravity columns are added. The author concluded that continuous columns have a more significant effect on taller structures (9- and 20-story buildings). It was also concluded that as continuous column stiffness increases, drift concentration decreases to a point where the columns rotate about the base as a rigid body. The mathematical models of the moment frames used by Tagawa did not include stiffness and strength degradation in the inelastic components. As a result, one of the recommendations for future research made by this study is to incorporate the stiffness and strength degradation in beams and columns to obtain the influence of the continuous columns on SMFs.

Flores et al. [7] investigated the influence of the gravity system on the collapse performance of SMFs. This study examined the influence of both the gravity columns and the gravity beam-to-column connections on the collapse performance of SMFs using the FEMA P-695 methodology [8]. The analyzed buildings, 2, 4 and 8 stories in height, have the same configuration as those used in the ATC 76-1 project [2], and were designed for a seismic

design category D_{max} ($S_s=1.5g$, $S_1=0.6g$) using the modal response spectrum analysis method (RSA). The seismic design category descriptions in P-695 are not specific to a particular location and represent a regional seismic hazard. More information regarding this matter can be found in the FEMA P-695 document. However, the seismic design category D_{max} is representative of the highest level of shaking expected in the U.S. The collapse performance was evaluated with and without the gravity system and key findings were that: (1) the gravity connections improved the collapse performance of the 2- and 4-story buildings but had almost no influence on the collapse of the 8-story building, and (2) the gravity columns had a very important influence on the taller 8-story building but that their influence decreased gradually for the shorter 4- and 2-story buildings. An important additional conclusion that Flores et al. [7] made, and it is the basis for the study performed herein, is that gravity columns are not likely to yield unless the gravity beam-to-column connections have significant moment capacity (greater than approximately 35% of the full plastic moment capacity of the gravity beam). It is noted in the aforementioned study that the gravity columns were modeled in two ways; using elastic elements and using elements with the capability to yield. The results indicated that the response was insensitive to the influence of yielding in the gravity columns.

In the new study reported herein, the influence of the gravity columns on the collapse performance of SMFs is evaluated using the Lumped Column approach. Unlike previous studies that have used this method [3, 4, 6], this investigation focuses first on proving the validity of the approach and then establishes the influence of the gravity columns on the collapse performance using the P-695 methodology [8]. Based on the assumption that gravity columns do not yield, the approach lumps all the gravity columns into a single elastic and continuous element. In this study, the same 2-, 4- and 8-story SMFs buildings used by Flores et al. [7] to investigate the behavioral improvements that the full gravity system provides are studied in more detail in order to quantify the influence of the continuous stiffness of the gravity columns. To evaluate the influence of the number of different gravity columns or the number of different column sections, a very simple method is used, wherein the total moment of inertia provided by the gravity columns (per story) is assumed to be a percentage or ratio of the total moment of inertia provided by the SMFs (per story). Then, the FEMA P-695 methodology is used to quantify certain parameters that will evaluate the effect of continuous gravity column stiffness on SMF behavior.

2. Building Overview

The 2-, 4-, and 8-story buildings utilize the same floor plan, as shown in Figure 1. The connections at the base of the columns of the SMFs are fixed for the 4- and 8-story models and pinned for the 2-story model. As can be seen from Figure 1 the lateral resisting system of these buildings is provided by three-bay SMFs located at the perimeter on each side of the building. These SMFs have prequalified reduced beam sections (RBS) connections [9] and are proportioned and detailed to resist the seismic forces and provide stability against P-Delta effects. The buildings were designed according to the strength requirements given in AISC 341-05 [9] and the seismic design requirements in ASCE/SEI 7-05 [10] with the exception that the deflection amplifier C_d was taken equal to the response modification factor, R , as specified in FEMA P-695.

The bay width (center line dimensions) between columns of each SMF is 6.1 m. The height of the first story is 4.6 m (to top of steel beam), and the height of all other stories is 4.0 m. The dead load is 4.31 kN/m^2 uniformly distributed over each floor, and the cladding load is applied as a perimeter load of 1.2 kN/m^2 . The unreduced live load is 2.4 kN/m^2 on all floors and 0.96 kN/m^2 on the roof.

The structures were analyzed in the East-West (long) direction. The gravity frame is considered to be part of the system and it is also shown in Figure 1. In this study, the columns oriented on the weak axis (orthogonal SMFs columns) are not considered part of the lateral resisting system and they are modeled with moment releases at the top and bottom of each story so they do not provide any lateral resistance. While these columns would clearly add to the lateral resistance of the system, they were not included in the study as the intended focus was on gravity columns that are bent about their major axis. While the number of gravity columns in these buildings is smaller than expected for most buildings, this is not a significant issue because their properties will be varied to study their influence. However, it is common practice that the number of gravity columns is much larger than the columns that form part of the lateral resisting system. As a result, the orientation of the strong axis of gravity columns can be evenly distributed in the two orthogonal directions to improve the overall structure's performance. The shaded area in the figure represents the tributary area for gravity loading on the individual East-West direction SMFs.

3. Modeling Approach

The two-dimensional numerical models developed for the nonlinear analysis were created using OpenSees [11] and include material and geometric nonlinearities. In the case where the gravity system was not included, P-

Delta effects were considered using a leaning column (with no flexural stiffness) placed parallel to the SMF and the gravity load applied to this column represents half of the total load from the gravity system that was not tributary to the SMF columns. For the cases where the gravity system was included, a leaning column was not required because each column (including those that bend about their weak axis) was represented in the model with its gravity load and related geometric stiffness explicitly included.

The approach used to model the SMFs used elastic beam elements and the nonlinear inelastic behavior was lumped into phenomenological plastic hinges located at the ends of the members. The components of the SMF with nonlinear material behavior are shown in Figure 2 (left) and they are: panel zones, plastic hinges located at the RBSs of the beam and plastic hinges at the column ends.

A complete description of the inelastic behavior of each component can be found in the ATC 76-1 report. However, a brief overview is provided herein. The hysteretic behavior of the RBSs in the beams is modeled using a rotational spring. Using OpenSees, the behavior was characterized by the OpenSees material called “Bilin”. The constitutive law of this material is based on the modified Ibarra-Krawinkler deterioration model [12], and the parameters to define the hysteretic behavior are as described by Lignos and Krawinkler [13].

Beam-column joints of the SMFs are modeled using a rectangular region composed of eight very stiff elastic beam-column elements and one nonlinear rotational spring to represent inelastic distortions due to column web shear yielding and column flange flexure yielding in the panel zone [14]. These deformations are represented by a tri-linear backbone curve assigned to the rotational spring.

The phenomenological component used to characterize the nonlinear behavior of the SMF columns is the same as the one used for the beams. This approach was taken because, at the time the analysis was performed, OpenSees did not have a component model with a hysteretic behavior that includes the axial force – bending moment (P-M) interaction. To account for the reduced bending strength of the columns due to the axial compressive force, a representative axial force was computed. Even though the axial force changes during a dynamic analysis, this representative axial force is computed before the analysis and the bending strength is reduced accordingly. The axial force used to determine the reduced strength was obtained by performing a nonlinear static pushover analysis and it was considered to be equal to $P_{grav} + 0.5 P_{E,max}$, where P_{grav} is the gravity load that the column is subjected to and $P_{E,max}$ is the maximum load that the column receives during the pushover analysis. The pushover analysis was

performed until a reduction of 20% in the base shear, V_{max} , is reached. With the axial load determined, the reduced bending strength is computed using the AISC P-M interaction equation [15] and subsequently used in the models. Note that a similar approach was used in the analysis of SMFs reported in ATC 76-1 [2].

For the nonlinear dynamic response history analysis, a Rayleigh damping of 2.5% is assigned to the first modal period T_1 and to period $T = 0.2 T_1$ in all cases. As proposed by Zareian and Medina [16], initial stiffness proportional damping is assigned to elements that remain elastic, and mass proportional damping is assigned to nodes where the mass is lumped.

4. Validation of the Models

The SMFs investigated in this study were previously analyzed by Zareian et al. [17] without the gravity system and the results were presented in the ATC 76-1 [2] project report. To validate the models that do not include the gravity system, a complete collapse evaluation of the 2-, 4- and 8-story models was performed and compared with the published results. The difference between the results reported by Zareian et al. and the ones found in this study was marginal. A more detailed description of the validation process can be found in Flores et al. [7].

5. Influence of Gravity Columns

In order to validate the approach where all the gravity columns are lumped into one elastic element, a number of supplemental analyses with the gravity system modeled explicitly were performed. The explicit model of the gravity system included all the gravity columns and the gravity beams but the beam to column connection's strength stiffness was neglected. This section of the study is devoted to show the influence of the gravity columns on the performance of SMFs. The gravity system is modeled explicitly as shown in Figure 2 (right). The gravity columns are pinned at the base, continuous all over their height (splices considered to provide full moment capacity), and to capture yielding they were modeled using force-based fiber elements with a bilinear material with strain hardening equal to 10% of the initial stiffness. The material used has a yielding strength capacity equal to 414 MPa . This value accounts for the expected material strength and strain hardening. Modeling columns with force-based fiber elements makes possible to incorporate the axial-moment interaction in the analysis but strength degradation is not considered. This is why plastic hinges at columns of the SMFs were modeled using a phenomenological approach. The gravity connections, are considered to be Partially Restrained (PR) connections

and they are assumed to have no flexural strength because the influence of the gravity column continuity without any consideration for gravity beam-to-column moment resistance is to be evaluated.

A typical design was conducted to obtain the size of beams and columns of the gravity framing. The system was designed using A992 steel and the same loads as previously mentioned for the SMF. For the purpose of member selection it was assumed that the gravity beams were non-composite and simply supported. The design was performed using SAP2000 [18]. The size of the gravity framing members is provided in Table 1, with grid identifiers (e.g. A3) as referenced in Figure 1.

The 2-, 4- and 8-story models with and without the gravity columns were subjected to the 44 far field records (two orthogonal components for each of 22 earthquakes) provided by FEMA P-695. The ground motions were scaled using the FEMA P-695 procedure. The intensity level at which the gravity columns are evaluated represent the Maximum Considered Earthquake (MCE) with 2% probability of occurrence in 50 years for the seismic design category D_{max} . The main nomenclature taken in this paper to indicate if the gravity column is included is: nStory + sGS, where n is the number of stories and s is the percentage of full strength assigned to the PR connections. If sGS is not included, the model does not include the gravity system, and instead includes a leaning column to account for P-Delta effects. Where the nomenclature uses the term 0GS, the gravity columns are included, but the beam-to-column connection capacity is taken as zero.

The observed influence of the gravity columns in the results is similar for the 2-, 4- and 8-story models. Drift concentrations at bottom stories and residual displacements were decreased considerably for the ground motions that caused large deformations in the building. However, it has to be mentioned that at the MCE level the influence of the gravity columns is minimum and just some of the ground motions caused the structures to undergo deformations that could result in the collapse of the building. Therefore, only the results of the nonlinear time history analyses that illustrate the influence of the gravity columns are shown in Figure 3.

Figure 3 displays the roof displacement histories and maximum inter-story drifts for the 2-, 4- and 8-story buildings. Starting with the 2-story model (Figure 3(a) and (b)), it can be seen how the residual displacement is decreased when the gravity columns are included in the analysis and this is reflected directly in the maximum

interstory drift. At the MCE level, just one out of the 44 ground motions had this effect on the 2-story building, while for the remaining motions, the influence of the gravity columns on the response was negligible. Contrary to the 2-story model, the 4- and 8- story models are more influenced by the gravity columns because taller buildings are more susceptible to have story drift concentrations at lower stories. This is the case for the 4- and 8-story models without the gravity columns where drift concentrations at the bottom stories is causing the collapse as can be seen in Figure 3(c) and (d) and Figure 3 (e) and (f) respectively. In the case of the 4-story building, two collapses were prevented by the gravity columns. On the other hand the gravity columns prevented five collapses for the 8-story building. These results verify what has been already observed by several authors: that continuous stiffness provided by gravity columns reduces interstory drift concentrations, thereby improving the seismic performance of buildings [3, 5, 6, 19].

6. “Lumped” Gravity Column Approach

Based on the assumption that the gravity columns do not yield even when the system undergoes large deformations, the Lumped Gravity Column approach combines all the gravity columns into one elastic element. However, in order to use this method two conditions have to be fulfilled: (1) the gravity columns are pinned at the base and continuous over their full height (splices provide full moment capacity), and (2) beam-to-column gravity connections have low flexural strength and high rotation capacity. If these conditions are not met the gravity columns may yield invalidating the method or the continuous elastic stiffness provided by the columns would not be achieved.

Figure 4 shows a sketch of a mathematical model that utilizes the lumped gravity column approach. The SMF is modeled with two vertical elements placed adjacent to it: (1) a "leaning column" that incorporates P- Δ effects, and (2) the lumped gravity column, which represents all the strong-axis oriented columns that are not part of the lateral load resisting system. It can be seen from this figure that the ratio of the sum of the moment of inertia of the gravity column (I_{Gcol}) to the moment of inertia provided by the columns of the SMF (I_{SMF}) has been computed, with the values shown to the right of the lumped gravity column. This value simply provides a measure of the stiffness of the gravity columns relative to the stiffness of the SMF columns, and is used only to allow the comparison of results when the size of the gravity columns is increased.

Prior to demonstrating the influence on the performance that gravity columns provide, it is necessary to execute several preliminary analyses to investigate a variety of different issues that might arise regarding the method, including (1) are the gravity columns yielding, (2) what is the strength required by the gravity connections to induce yielding in the gravity columns, (3) what is the strength required by the gravity column's splices to provide continuity, and (4) are similar results obtained when a lumped column is used in lieu of modeling the individual columns? Each of these issues are investigated in the following subsections.

6.1. Yielding in Gravity Columns

The assumption that gravity columns remain elastic even when the structure is subjected to large displacements is fundamental to the concept that one elastic element with properties that represent all the gravity columns will give virtually the same results as a model with each individual gravity column included explicitly. In this section, the potential for yielding in the gravity columns is investigated by performing nonlinear static pushover and nonlinear dynamic analyses. In order to evaluate the influence of the gravity columns when the gravity beam-to-column strength is not included, the PR connections are considered to have no strength. However, since it is known that a perfectly pinned beam-to-column gravity framing connection is not possible in an actual building, yielding in the gravity columns is also studied when the PR connections have some strength.

The flexural strength of the PR connections is considered to be a percentage of the full plastic moment strength (M_p) of the bare gravity beam. Depending on the type of connection, the flexural strength can be high but it is known that shear tab connections and other commonly used gravity connections do not have significant flexural strength capacity. Liu and Astaneh [20] carried out cyclic tests to evaluate the flexural capacity of simple connections. The results showed that shear tabs, which are common connections used as part of the gravity system, had a moment capacity approximately equal to 20% M_p if the contribution of the concrete slab is neglected and 35% M_p if it is considered. Therefore, it is assumed based on literature and other investigations, such as that performed by Gupta and Krawinkler [21], that a realistic value to represent the flexural strength of the gravity connection is 30% M_p . Another important characteristic that gravity connections must comply with is to have a high rotation capacity. The experimental results published by Liu and Astaneh [20] demonstrate that this characteristic can easily be achieved in simple connections. For example, shear tabs connections reached an inelastic rotation that varied from 0.09 to 0.14 radians. It was also found that the composite action (contribution of the concrete slab) was

lost due to damage at a rotation approximately equal to 0.04 radians. After this point the connection behaved the same as a bare connection.

Even though the cyclic behavior of the PR connections is complicated to characterize, a simple model, such as the one given by the ASCE 41-13 [22], is considered to be adequate for the purpose of this study. One of the parameters given by this model is the rotation at which the PR connections yield (0.005 radians). The backbone curve of this model presents a plateau after the yielding point and it provides the rotation at which the connection losses the majority of its capacity. The moment capacity of the connection for this model depends on the type of connection and the limit state (failure mode) that controls the design. A “top and bottom clip angle connection” is chosen as a PR connection for this analysis, and the connection is configured so that the controlling limit state is flexural failure of the angles. More information regarding the modeling of the PR connections can be found in Flores et al. [7].

To establish whether or not the gravity columns yield, two analyses are performed: one where the material assigned to the fibers of the cross section that form part of the force-based elements used to model the gravity columns is elastic, and the other one where the material assigned to the fibers is bilinear with strain hardening equal to 10% of the initial stiffness. The nonlinear static pushover analysis is performed on the 8-story building with and without the gravity system for the cases where the gravity columns behave elastically and inelastically. Additionally, nonlinear dynamic time history analysis is performed using eight ground motions chosen from the FEMA P-695 Far-Field record set. The ground motions used for this check were chosen arbitrarily and they are listed in Table 2. Incremental Dynamic Analysis [23] is used systematically to subject the building to increasing ground motion intensity, resulting in large displacements.

Figure 5 shows selected results for the analyses performed on the 8-story building with gravity beam-to-column connections with no strength. From the pushover curves it could be inferred that the gravity columns are not yielding because the curves with elastic and inelastic columns are identical. It can be further concluded that the gravity columns significantly improve the building’s post-yield stiffness. A close investigation into the fiber behavior of the column’s section confirmed that no yielding occurred in the gravity columns. For the dynamic analysis, the ground motions were scaled following the FEMA P-695 methodology. The ground motions are collectively scaled (or “anchored”) to the ASCE7-05 [10] response spectrum (MCE level) such that the median

spectral acceleration ($S_a=0.55g$) of the record set matches the target spectral acceleration at the fundamental period, ($T=C_uT_a=1.64$ sec) where C_u is a coefficient that represents an upper limit on the calculated period and T_a is an empirical approximation of the fundamental period. Note that this period is computed according to ASCE7-05 and is not the actual period of the structure. However, for the purpose of checking if yielding occurs in the gravity columns this is not relevant since the comparison is made at level intensities higher than the MCE level. The results from the nonlinear time history analysis shown in Figure 5(b) represent a level of intensity 1.8 times larger the MCE level intensity ($S_a=0.99g$). Based on the roof displacement time history results it appears that even at large deformations the gravity columns do not yield.

The next step required to validate the assumptions that the gravity columns do not yield is to assign a flexural capacity to the PR connections. A flexural strength of 30% M_p has been chosen as a realistic value for the connections based on previous investigations [20, 24]. The same analyses performed for the case where the gravity connections have no strength are executed and the results are shown in Figure 6 (a) and (b). The pushover curves show that gravity columns are still not yielding since there is no difference in the response for the two cases. Again, this was confirmed by checking if the fibers reached the yielding point.

On the other hand, the results for one of the cases for the dynamic analysis under large deformations (Figure 6 (b)) display a discrepancy between the elastic and inelastic results. A close inspection of fiber stresses in the column indicated that yielding occurred at 10% drift in the first story (Figure 7). This value of drift is very large and it is commonly considered collapse in steel moment frame structures [25].

Even though yielding in the gravity columns occurred for one of the eight analyzed ground motions, it was for an earthquake with a hazard level much larger than the MCE and it occurred after the deformation exceeded 10%. Thus, it can be said that the gravity columns do not yield if the gravity connections have a low flexural strength capacity (less than 30% M_p). However, the following subsection will show that gravity connections will induce yielding if they have a significant moment capacity.

6.2. Yielding in Gravity Columns Induced by Gravity Connections

The previous section showed that the gravity columns do not go into the inelastic range when the gravity connections have a flexural capacity less or equal than 30% M_p . In this section PR connections with high flexural

capacity are used to check if the gravity columns yield. Nonlinear static pushover analysis is used for this purpose. Whether or not yielding occurs is established by comparing the results obtained with gravity columns behaving in an elastic or inelastic manner. Furthermore yielding in the fibers of the sections was closely inspected. Figure 8 shows the pushover curves obtained for the buildings with PR connections with strength equal to 50% and 70% M_p . The difference between the pushover curves indicate that yielding occurred in the gravity columns and this was validated by investigating the stresses on the fibers of the sections. This demonstrates that high moment capacity connections induce yielding in the gravity columns. Even though yielding occurred in the gravity columns at a large drift (0.025), it can be concluded that in order to use the Lumped Column approach, the gravity connections must have a relatively low flexural strength.

6.3. Effect of Gravity Columns Splices

One of the most important issues when improving the seismic performance of SMFs using the gravity columns is that they have to provide continuous stiffness, and this is achieved by making them continuous over the full height. However, in buildings taller than two stories, splices in the gravity columns are typically used. Therefore, in order to use the continuous column approach the flexural strength provided by the splices has to be sufficient to simulate the continuity in the gravity columns.

Section J1.4 of the AISC Specification [15] states that compression splices in columns need to hold the parts secure in place. As a result of this requirement, AISC provides a series of standard column splices that can be found in Table 14-3 of the specification. Even though splices are not designed to carry moments, they can provide significant flexural capacity. There are two main sources of bending capacity for column splices: the first one, coming from the compression load in the column, is the bending moment required to cause tension in one of the column flanges, and the second one is the moment capacity that splice plates and bolts provide [26]. The first source depends on the compression load acting on the section, and the second one depends on the section depth and number of bolts. For instance, the gravity columns with section W18x143 located on axes B3 and E3 (Figure 1) have a 2669 kN compressive load from the gravity loads at the fourth story (splice location). The flexural capacity of a standard splice connection with eight A325N 1.9cm diameter bolts will have a flexural capacity of 30% M_p , of which 12% comes from the moment required to induce tension in one of the flanges and 18% from the moment capacity that the bolts provide. The same connection could have a flexural strength of 53% M_p if twelve 2.2cm diameter bolts

and A325X are used. The capacity of the splices is much larger for lighter column sections and it was also found that due to a larger lever arm, column splices provide more flexural capacity when W18 are used instead of W14 sections.

In this part of the study, the flexural strength of the gravity columns splices is investigated. Additionally, as a result of the study performed by Flores et al. [7] where it was found that the splice location is very important, four different configurations are used to investigate the influence of the splice location on the performance. The splices are located at the same level in all the gravity columns (including the two end gravity columns that are placed on the extremes of the SMF), but the number of splice locations along the height and story level(s) in which the splices are located is varied. The cases considered are shown in Figure 9 of which cases III and IV would require at least two levels of splices but the importance of the upper splices is to be evaluated using these cases.

Initially, the splices are considered to have no flexural strength capacity so the results to be obtained represent a lower bound of the building performance. The influence of the splice location is quantified by means of nonlinear static pushover analysis. It can be seen from the cases to be studied that the difference between case I and III, and the cases II and IV, evaluates the influence of the upper story splices. On the other hand the difference between the cases I and II and the cases III and IV evaluates the influence of locating the splices at different stories.

Figure 10 (a) and (b) presents the results obtained from the nonlinear static pushover analyses. By observing the pushover curves for cases I and III and cases II and IV it can be concluded that the upper story splices have almost no influence. However, the story where the splices are placed is very important and this is noticed by the difference in performance between cases I and II and cases III and IV. The main conclusion obtained from these analyses is that the first splice is the most important and it should be located as high as possible from the base of the column. Although this is true for the buildings investigated in this study, more research is required to generalize the conclusion to other structural systems.

Based on these results, the flexural strength required by the splices to provide continuity is investigated. Since the upper story splices have a negligible influence on the performance, only cases III and IV are considered. Figure 10 represents the lower bound performance of the building because the splices were modeled with zero flexural strength capacity. Thus, in this section a flexural strength capacity is assigned to the splices using an elasto-plastic hysteretic behavior and the influence on the behavior is analyzed. The rotational stiffness of the splices do

not have major influence on the performance of the structure and that is why a rigid connection (infinite stiffness) is used. A side study was performed to validate this finding and it also matches what was already stated by Ladani et al. [27] where a parametric study changing the splice stiffness was performed and the variation on the results was minimum. As a result of the assumption that the rotational stiffness of the splice connection is large, the flexural strength capacity of the splice connections is considered the parameter to be studied and it is taken as a percentage of the plastic moment of the column. The flexural strength is varied and the performance compared with the performance obtained for the building with no splices

The required strength to be provided to the splices is computed by performing nonlinear static pushover analysis. Figure 11 presents the gravity column's splice strength requirements for cases III and IV in order to deliver continuity. It can be seen that if case III is used (Figure 11 (a)), the splice connection has to be fully restrained (around 90% M_p) while the case IV (Figure 11 (b)) requires splice connections with flexural strength equal to 50% the full plastic moment capacity of the columns. This illustrates once more that the first splice should be located as far as possible from the base of the column because they provide more continuity and the strength requirements for the splices are reduced.

6.4. Lumped vs Explicit Model

Once the assumptions of the approach have been proved to be correct, the final step, which is the comparison between the “lumped” and the explicit model is carried out and the results are described in this section. As already indicated, the lumped method utilizes one elastic element with the combined properties of all the gravity columns that are in the strong axis, whereas the explicit model includes all the gravity columns and beams (with hinged connections for the beams).

The results of performing a nonlinear static pushover analysis and nonlinear dynamic analysis are shown in Figure 12. It can be seen that the pushover curves and the roof displacement time history are exactly the same even when the structure is subjected to large displacements. The outcome was the same for all the ground motions used for the analysis.

The discussion presented previously was dedicated to demonstrate the concept that accumulating all the gravity columns that are in the strong axis into a single elastic, continuous and pinned base element is a valid approach to represent the behavior of all of the gravity columns in a structure. Since the influence of the gravity

columns is most significant when the structure is subjected to large displacements, the collapse performance of SMFs including the gravity columns is evaluated in the following section of this investigation.

7. Collapse Performance of SMFs Including Gravity Columns

As known, in the present, SMFs are designed and analyzed without considering the gravity system as part of the lateral resisting system. One of the reasons for not including the gravity connections is the complexity of modeling the gravity connection's hysteretic behavior. However, the previous sections in this paper were dedicated to prove that a relatively easy method to include at least the gravity columns in the analysis is possible. The improvement that these provide is more appreciable when the structure undergoes large deformations and that is why the next section addresses the effect of the gravity columns on the collapse performance of SMFs using the FEMA P-695 methodology. The buildings to be analyzed with and without the gravity columns are the 2-, 4- and 8-story structures from the ATC 76-1 project which were already described in previous sections.

The effect of the gravity columns on the collapse performance of the SMFs could be evaluated by analyzing different gravity systems with different gravity columns but it would not be practical. Instead, the properties of the lumped gravity column is varied to represent different number of gravity columns or different column sections and establish the influence on the structure's performance. The stiffness per story of the gravity columns was assumed to be a percentage or ratio of the stiffness per story of the SMF's columns. Therefore, as shown in equation Eq. (1), the moment of inertia of the gravity column, I_{Gcol} , is a constant (α) times the total moment of inertia provided by the columns in the SMF (I_{SMF}). The α values computed for the gravity system that form part of the 8-story building are shown in Figure 4. This is an approximate estimation of the SMF stiffness per story because it considers the beams of the SMF to be rigid. On the other hand the stiffness provided by the gravity columns, acting independently of the SMF, is null because the gravity connections have no strength and the columns are pinned at the base. So the apparent stiffness of the gravity columns comes from the relative displacement (if it exists) of the building's floors. A side study was performed to evaluate the actual influence of the gravity columns on the lateral stiffness per story. Even though the coefficient alpha can be large, it was found that the lateral stiffness per story provided by the gravity columns is significantly lower.

$$I_{Gcol} = \alpha \sum I_{SMFC} \quad (1)$$

Following this procedure, the total moment of inertia of the gravity columns per story was lumped in one continuous element placed in parallel to the SMF as is illustrated in Figure 4. Each story of the gravity column has a percentage (100 times α) of the total moment of inertia of the SMF's columns. In other words, instead of trying different number of gravity columns and different sections, an accumulated inertia per story was assigned and it was obtained as a percentage of the inertia per story of the SMF columns.

The approach taken herein is somewhat different from what other researchers have done in the past [4, 19]. The general trend is to compute a Drift Concentration Factor (DCF), which is the ratio between the maximum inter-story drift ratio and the roof drift ratio. The DCF is obtained from the nonlinear dynamic results and the closer the DCF gets to unity, the lower the possibility of developing drift concentrations. In order to decrease the DCF to a value that is considered acceptable, the stiffness of the gravity column is varied using Eq. (2).

$$I_{Gcol} = \alpha \frac{EI}{kh^3} \quad (2)$$

where EI is the sectional bending stiffness of the continuous column, h is the story height and k is the story stiffness of the frame. The ratio α then is chosen by performing nonlinear dynamic analyses and the DCF is decreased to a value that is considered reasonable.

In order to avoid performing nonlinear dynamic analyses, this study does not use the DCF to establish the size of the gravity columns. The approach used instead is to assign a specific inertia to the gravity columns and then a nonlinear static pushover analysis is performed. From these results the most adequate or “optimal” stiffness is found by evaluating the improvement on the post-yield stiffness. Even though this approach is somehow easier, more research is required to develop a simplified methodology or guideline that does not involve nonlinear analyses to obtain the “optimal” stiffness. The following sections will clarify how the stiffness could be obtained from the pushover results.

7.1. Nonlinear Static Pushover Analysis

One of the required steps in the FEMA P-695 procedure is to execute a nonlinear static pushover analysis. The parameters computed from this analysis are the overstrength (Ω) and period based ductility (μ_T). The overstrength measures the improvement on the building strength. On the other hand, the period based ductility is a measure of the post-yield stiffness improvement and from the FEMA P-695 perspective it has a direct effect on the Adjusted Collapse Margin Ratio (ACMR) which in turn is used to compute the probability of collapse at the MCE level. In order to evaluate the influence of the gravity columns on the aforementioned parameters, nonlinear static pushover analysis was performed on all the SMFs. The ratio of the inertia per story of the gravity columns was varied progressively from 10% to 100%. The results for the 2-story model are not included in this section because the influence of the gravity columns on the pushover curve is relatively not important since the P-Delta effect is insignificant for this building. The results of the pushover analyses and the influence on the period based ductility (μ_T) for the 4-story building are shown in Figure 13 (a) and (b). These results are for the gravity columns with stiffness per story that varied from 5% (4Story +5%GSCol) to 100% (4Story +100%GSCol).

Figure 13 (a) illustrates the influence of the gravity columns on the post-yield stiffness of the 4-story model. This influence is directly related to the period based ductility results shown in Figure 13 (b). The main conclusion from these figures is that it appears to exist an optimal performance that could be obtained when the gravity columns have the inertia per story in the neighborhood of 40% the inertia per story of the SMF. According to this analysis, it is not necessary to increase the gravity columns sections more than this ratio to improve the performance because the post-yield stiffness does not increase. Another aspect to point out is that the gravity columns have a minimum effect on the building overstrength.

The results of the pushover analyses and the calculated period based ductilities for the 8-story model are shown in Figure 14 (a) and (b). For these analyses, the inertia of the gravity columns was varied from 5% (8 Story + 5%GsCol) to 100% (8Story + 100%GsCol) the inertia of the SMF. In this case, the improvement is more noticeable than for the 4-Story model. The increase in the total moment of inertia per story of the gravity columns does not get saturated as in the case of the 4-story model, where after a ratio of 0.4 the results did not vary. To explain this behavior it has to be recalled the final effect of continuous stiffness on buildings. If the gravity columns are large enough, the structure will tend to rotate with respect to the base, producing the same drift ratio in each level. In the case of smaller buildings the stiffness required by the gravity columns to accomplish this effect is lower than for taller buildings.

The influence of the gravity columns, when they are continuous, is important for the 8-story model. The benefits of improving the post-yield stiffness are directly related to the dynamic stability since it counteracts the P-Delta effects. From Figure 14 (a), it can be seen that the post-yield stiffness improves faster when the gravity columns inertia vary from 5% to 50%. The period based ductility increases around 20% when the gravity columns have inertia per story equal to 50% the inertia per story of the SMF (Figure 14 (b)). So, this ratio could be the “optimal” to be used for the gravity columns. Additionally, the originally designed gravity system is shown in Figure 14 (a) to illustrate the behavior of this system relative to the ones where the influence of the columns is considered.

7.2. Collapse Performance Evaluation

Following the P-695 methodology, the collapse performance of the structures is obtained performing nonlinear dynamic time history analyses, specifically using the incremental dynamic analysis (IDA) procedure [23]. Each of the 44 different far-field ground motions is scaled to increasing intensities until the structure collapses which was considered to occur when the global drift is 0.1 or when the slope of the IDA curve is less than or equal to 20% the elastic slope of the IDA. This definition of collapse was established by FEMA 350 [25]. Once the IDA was performed, the collapse margin ratio (CMR) was obtained using the procedure defined in the P-695 methodology. The CMR is the ratio between the spectral acceleration that causes the median collapse (22 out of the 44 ground motions) to the spectral acceleration at MCE level.

The parameters to be used to evaluate the influence of the gravity columns on the performance of the SMFs are the Adjusted Collapse Margin Ratio (ACMR) and the probability of collapse. The ACMR is a ratio as described by P-695 that adjusts the CMR with the Spectral Shape Factor (SSF), which in turn accounts for the differences between the spectral shape of rare ground motions in California with the design spectrum or a uniform hazard spectrum. The probability of collapse is computed at MCE level (ACMR=1). For further information regarding these parameters, the readers are referenced to the P-695 methodology [8] and to the study performed by Deierlein et. al [28].

The performance evaluation is carried out using a subset of the ratios (than the ones used for the pushover analysis) of the total moment of inertia per story of the gravity columns to the total moment of inertia of the SMF. The results of the analyses computed on the 2-story model are shown in Figure 15 (a) and (b).

Figure 15 (a) illustrates the results of performing multiple IDAs to compute the ACMR of different models. Even though the gravity columns did not improve the post-yield stiffness on the pushover curves, it can be seen from Figure 15 (a) and (b) that they have some influence in the collapse performance. The ACMR increases slightly but in a regular manner until the gravity columns have a ratio of 0.5 with respect to the SMF per story stiffness. After this point the results are close but they do not necessarily keep increasing. The probability of collapse decreases from 2.49% to 1.33 % when the gravity columns stiffness per story ratio is 0.5. In general, the influence of the gravity columns is not as noticeable as it will be seen for the 4- and 8- story models. Moreover, the results fluctuate making difficult to establish an adequate inertia for the gravity columns.

The results of the analyses performed on the 4-story model are shown in Figure 16 (a) and (b). The ratios of the inertia per story of the gravity columns were chosen based on the results of the nonlinear static analysis. As Figure 13 displays, the post-yield stiffness improves progressively when the gravity columns are included until the ratio of the inertia per story is equal to 0.4. After this point the post-yield stiffness is the same and this is reflected in the results obtained from the collapse performance evaluation.

It can be seen from Figure 16 (a) and (b) that the gravity columns improve the ACMR and the probability of collapse. However, in comparison to what was found for the 8-story building (see Figure 17) the improvement is not drastic. It is important to note that having more gravity columns or increasing their section sizes do not enhance the results after certain point. Probably the best ratio to use would be 0.4 because the ACMR goes from 1.99 to 2.10 (10% increment) and the probability of collapse decrease from 8.44% to 5.89%. Using larger ratios could represent an increase in the cost of the structure (more gravity columns or larger sections) and the improvements obtained are virtually the same. These results correlate with the results obtained in the pushover analysis where the post-yield stiffness improvement was saturated for ratios larger than 0.4.

The performance evaluation for the 8-story model is shown Figure 17(a) and (b). As it was expected from the pushover curves, the improvement of the ACMR and probability of collapse is noticeable when gravity columns are included as part of the lateral resisting system. From Figure 17 (a) it can be seen that the ACMR goes from 1.99 for the 8-story building without the gravity column influence to 2.86 when the 8-story model includes the gravity columns with stiffness per story ratio equal to 1. It is interesting to note again the direct relation between the nonlinear dynamic with the nonlinear static results. Figure 14 shows how the post-yield stiffness improves

progressively when the stiffness per story of the gravity columns increases. Unlike the 4-story model the improvement does not reach a limit for ratios smaller than 1.

Even though the use of larger gravity columns brings more benefits to the performance, it could be uneconomical. Therefore, a ratio that is considered to be the optimal has to be established. This ratio will benefit the performance and represent an economical solution. The pushover curves have an important role on determining this ratio. From Figure 14, it can be seen that the post-yield stiffness increases rapidly up to a point where the ratio of inertias is equal to 0.5. After this point, the post-yield stiffness still improves but in smaller increments. Thus, this ratio could be in this case considered to be the optimal for design. The ACMR for this ratio goes from 1.99 to 2.59 which represents an increase of 30.5%. On the other hand, the probability of collapse decreases from 8.51% to 2.84% which is a decrease of 66%.

7.3. Fragility Curves

Another method to present the results is using the collapse fragility curves. As indicated by the FEMA P-695 methodology, the collapse fragility curve is characterized by a lognormal cumulative distribution where the mean is the ACMR and the uncertainties are indicated by the standard deviation. More information can be found in the FEMA P-695 document or in the paper by Deierlein et al. [28]. Figure 18 illustrates the fragility curves for the 2-, 4- and 8-story models. Noting that not all the analyzed cases are included, the curves represent the range of variation between not having gravity columns and having a ratio of inertia in the gravity columns equal to 100%. It can be seen how the probability of collapse decreases when the gravity columns are included in the analysis. However, it is clear from Figure 18 (c) that the range of variation (8-story model) is significant in comparison with that of the 2-, and 4- story models, proving once more that this approach is more suitable for taller buildings.

Figure 19 displays a portion of the fragility curves to illustrate the probability of collapse. As already mentioned this is computed at MCE level (ACMR=1) and it can be seen how it decreases for all cases but it is especially significant for the 8-Story model (Figure 19c).

8. Conclusions

The purpose of the investigation carried out herein was to quantify the influence of the gravity columns continuity on the seismic performance of SMFs. For this purpose, the gravity columns were included as part of the lateral resisting system and the approach used to incorporate them was by lumping all the gravity columns that are located in the strong axis into one elastic, continuous element pinned at the base. The first part of this paper was dedicated to validate the approach used to include the gravity columns in the analysis. Then, in order to quantify the benefits of including the gravity columns, the collapse performance of SMFs including the gravity columns was evaluated and the conclusions obtained are:

- The gravity columns do not yield if the gravity connections have low flexural strength capacity (less than $30\%M_p$), and pinned at the base.
- The flexural capacity of the splices depends on the gravity column section, the compressive force, the number of bolts and the story location. Placing the column splices as far as possible from the base of the column is beneficial to decrease the flexural strength requirement to provide continuity.
- The influence of the gravity columns is to improve the performance for all three buildings. However, the improvement on the collapse performance of the 8-story model is significant in comparison to the other two buildings.
- It is known that continuous stiffness can prevent or reduce the probability of buildings to develop drift concentrations or weak story mechanisms. This was reflected in this study by reducing the probability of collapse of the structures.
- The stiffness to be assigned to the gravity columns in this study was taken as a ratio of the SMFs column's inertia. This was an arbitrary way of measuring the number of gravity columns that the building could have. However, the adequate stiffness of the gravity columns could be obtained directly by performing nonlinear static pushover analysis.
- From the results, it can be seen that there is a direct relation between the pushover curves and an adequate stiffness to be assigned to the gravity columns. However, in order to provide guidelines on how to compute this adequate stiffness without performing nonlinear static pushover analysis more research is required.
- The gravity system is considered to be an implicit factor of safety and that is why it is not typically included in the analysis. However, if gravity columns are purposely made continuous and an adequate

stiffness is assigned to the gravity columns, the performance of the building could be enhanced considerably.

- One of the conclusions presented in the ATC 76-1 project was: “there is a need to develop design concepts that prevent (or at least delay) the formation of partial mechanisms in lower stories of special SMF systems” [2]. By observing how the collapse performance of SMFs improved in this study, it could be concluded that the gravity columns may be an option to fulfill this need.

The study presented herein focused on the seismic performance of SMFs when the gravity columns are included in the analysis. Despite the improvement on the performance, more research is required to establish the feasibility of designing the gravity columns with the purpose to improve the seismic behavior of SMFs. Different aspects regarding the gravity system such as buckling in the gravity columns, design forces along and at the base of the columns, and base column connections with flexural capacity should be investigated more in depth. On the other hand topics like the incorporation of the concrete slab on the SMF model needs further investigation. Moreover, further investigations should be performed to extend the lumped column approach to three dimensional analysis. In this case it would be required to have more than one lumped column in order to capture completely the torsional P-Delta effects which can be extremely important as is shown in the study by Flores et al. [29].

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Table 1 a) 2-Story b) 4-Story c) 8-Story Gravity System Member Sizes

a) 2-Story Model

Story	Columns			Beams		
	A1,F1	B3, E3	C3, D3	A1 - B1, E1 - F1	A3 - B3, E3 - F3	B3-E3
1	<i>W14x43</i>	<i>W14x90</i>	<i>W14x61</i>	W24x62	W24x94	<i>W16x36</i>
2	<i>W14x43</i>	<i>W14x90</i>	<i>W14x61</i>	W24x55	W24x84	<i>W16x36</i>

b) 4-Story Model

Story	Columns			Beams		
	A1,F1	B3, E3	C3, D3	A1 - B1, E1 - F1	A3 - B3, E3 - F3	B3-E3
1	<i>W14x43</i>	<i>W14x90</i>	<i>W14x61</i>	W24x62	W24x94	<i>W16x36</i>
2	<i>W14x43</i>	<i>W14x90</i>	<i>W14x61</i>	W24x62	W24x94	<i>W16x36</i>
3	<i>W14x30</i>	<i>W14x90</i>	<i>W14x61</i>	W24x62	W24x94	<i>W16x36</i>
4	<i>W14x30</i>	<i>W14x90</i>	<i>W14x61</i>	W24x55	W24x84	<i>W16x36</i>

c) 8-Story Model

Story	Columns			Beams		
	A1,F1	B3, E3	C3, D3	A1 - B1, E1 - F1	A3 - B3, E3 - F3	B3-E3
1	<i>W18x76</i>	<i>W18x143</i>	<i>W18x76</i>	W24x62	W24x94	<i>W24x94</i>
2	<i>W18x76</i>	<i>W18x143</i>	<i>W18x76</i>	W24x62	W24x94	<i>W24x94</i>
3	<i>W18x76</i>	<i>W18x143</i>	<i>W18x76</i>	W24x62	W24x94	<i>W24x94</i>
4	<i>W18x50</i>	<i>W18x76</i>	<i>W18x50</i>	W24x62	W24x94	<i>W24x94</i>
5	<i>W18x50</i>	<i>W18x76</i>	<i>W18x50</i>	W24x62	W24x94	<i>W24x94</i>
6	<i>W18x50</i>	<i>W18x76</i>	<i>W18x50</i>	W24x62	W24x94	<i>W24x94</i>
7	<i>W18x40</i>	<i>W18x50</i>	<i>W18x40</i>	W24x62	W24x94	<i>W24x94</i>
8	<i>W18x40</i>	<i>W18x50</i>	<i>W18x40</i>	W24x55	W24x94	<i>W24x94</i>

Note: Gravity system shown in *bold italic* text.

Table 2. Ground Motions Chosen From Far-Field Set

Component Name	Filename	npts	dt	PGA(g)
Manjil-Abbar	ABBAR--L.AT2	2676	0.02	0.407
Duzce-Bolu	BOL000.AT2	5590	0.01	0.458
Superstition Hills-Poe	B-POE270.AT2	2230	0.01	0.522
Loma Prieta-Capitola	CAP090.AT2	7991	0.005	0.483
Hector-Hector	HEC090.AT2	4531	0.01	0.367
Northridge-CC	LOS270.AT2	1999	0.01	0.400
Kobe-Nishi Akashi	NIS000.AT2	4096	0.01	0.525
San Fernando-LA	PEL090.AT2	2800	0.01	0.366

Figure 1. Building Overview

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Figure 6. Checking Yielding on 8-Story Model a) Pushover Curves b) Nonlinear Dynamic Analysis (PR = 30% M_p)

Figure 7. Inter-Story Drift 1st Story ("Manjil-Abbar") SF=1.8 $S_a=0.99g$

Figure 8. Inducing Yielding in Gravity Columns

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