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STUDY OF THE EFFECT OF IN-PLAN ASYMMETRY IN MULTI-STORY BUILDINGS SUBJECTED TO UNI- AND BI-DIRECTIONAL SEISMIC MOTIONS

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SUMMARY

Time history response analysis on three-dimensional models of multi-story asymmetric-plan buildings with nonlinear behavior considering both uni-directional and bi-directional earthquake action are carried out. The five-story buildings structure is formed by six resisting planes (columns and beams) connected at each floor to a flat slab and fixed at the base. The flat slab was modeled as a rigid diaphragm (aspect ratio 2:1) and in the center of mass of the floor slab three degrees of freedom (two horizontal displacements and in plan rotation) and lumped masses were considered.

To study structures with different characteristics the models were defined based on the following parameters: T_y (coupled translational period, Y direction), ω_x/ω_y (ratio of coupled translational frequencies along X and Y directions), ω_θ/ω_y (ratio of coupled torsional and translational frequency), e_x/r (normalized static eccentricity, X-axis), and the seismic response modification factor R^*.

The earthquake action is defined by the two horizontal components recorded in Llo-Lleo during the Central Chile earthquake of March 3, 1985. These components were applied in Uni-Directional and Bi-Directional forms, changing the angle of incidence in 15º increments with the purpose of finding the critical angle of incidence where the local and global responses are maximized.

The results show that depending on the type of response considered (e.g.: displacement of center of mass, base shear of a resisting plane) its magnitude varies in proportion with the structure lack of asymmetry in-plan. On the other hand, in symmetric structures the critical angle of incidence for responses measured along the X direction tends to occur around the X-axis, and for responses measured in the Y direction around the Y-axis. For asymmetric structures this effect does not occur.

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INTRODUCTION

It is difficult to think that for the seismic design of buildings in actual professional practice more sophisticated methods than Response Spectrum Analysis (RSA) will be used. Therefore, it is very important to have a measure of the reliability of such methods in predicting the seismic behavior of the structure. If the two horizontal components of the actual earthquakes are of similar characteristics and magnitude, it is clear that that the resulting ground movement will have changes in the direction of the earthquake action and its magnitude during the duration of the event. This generates a question with respect to knowing which is the most unfavorable condition so that the seismic response of each element in the structure is maximized. The simultaneousness of the orthogonal seismic loads on three-dimensional structural systems has not been considered in explicit form in the code recommendations for earthquake resistant design of buildings [1]. In fact, present codes recommend to make two independent analyses using the RSA method and defining the earthquake action based on a design spectrum. The earthquake action is applied in Uni-directional form along certain directions arbitrarily defined by the designer.

Most of the buildings designed by normal engineering offices have several resisting planes that are usually oriented along two directions (that are usually orthogonal). Thus the selection of the main analysis directions for the building is done in an arbitrary manner. A common alternative is to use the two directions of the facades of the building that the designer considers more relevant. With this strategy, most of the resisting planes are oriented either parallel or perpendicular to the main analysis directions. In such sense, they are usually called “principal directions” of the plan although they are not necessarily the two directions in which the dynamic response is fully uncoupled, which would be the proper definition of principal directions.

Thus the design of the resistant elements of the building is made using the responses obtained after applying the Uni-directional seismic loads acting separately in each one of the two “principal directions” of the plan. In some cases the results of both seismic analyses are combined by means of some empirical rule (e.g. 100/30, SRSS, etc.) in order to obtain an estimation of the response that would be obtained when considering the Bi-directional effect of the seismic ground motion; these rules generally lack theoretical basis and have limited if any experimental background.

As a consequence, it is expected that the results obtained in the analyses that consider a single Uni-Directional horizontal earthquake component, like for example that the structural elements in the resisting planes that are perpendicular to the earthquake action remain elastic [2], could not be representative of the actual behavior of a structure when subjected to a real earthquake. A literature survey has confirmed that systematic studies to test different alternatives to consider in simple form the effect of the Bi-directionality in the analysis and design of real structures with the possibility to go into the non-linear behavior range by comparing it to the actual responses of systems under the action of the real earthquake ground motions (two horizontal components only) do not in fact exist.

Ceballos *et al.* [3] studied parametric 3D models of one-story RC buildings with in-plan asymmetry in two directions and elastic behavior. They were structured by means of four perimeter frames and a single double-T shear wall at the center of the plan. The earthquake load was defined as the two recorded horizontal components in Llo-Lleo during the earthquake that occurred in the central region of Chile on March 3, 1985. Such components were applied in Uni- and Bi-directional form varying the angle of incidence in 15º increments to identify the “critical angle” where the responses reach their maximum values. Depending on the kind of response, the results show that the magnitude of the maximum response varies if the structure is asymmetric in one or in both directions. Also, for structures that are symmetric in one-direction it was observed that the maximum responses obtained in resisting elements along directions

X and Y are associated to “critical angles” which tend to be aligned with the X- and Y-axis, respectively. In asymmetric structures this is not observed.

Faella *et al*. [4] studied the influence of the Bi-directional ground motions on the seismic response of real symmetric RC buildings by comparison with Uni-Directional seismic response. The buildings are conformed by frames (four perimeter frames and two interior frames) of four stories and were tested in the laboratory at real size [5]. It was found that the inter-story displacements clearly showed the significant damage produced by the simultaneous action of both horizontal components in the lower floors of the buildings. Ghersi and Rossi [6] examined the influence of Bi-directional seismic excitations on the inelastic behavior of in-plan irregular systems and in systems symmetric in one direction, represented as one-story models with resisting elements arranged along two orthogonal directions. The analysis results showed that the inelastic response is slightly affected by the simultaneousness of the two seismic components, although the results have large dispersion. De Stefano and Pintucchi [7] presented a refined structural model of an in-plan asymmetric building with which it is possible to overcome the limitations of the simplified models used before. The new model is used to evaluate the effects of the inelastic interaction in torsionally rigid asymmetric systems, considering two component earthquake actions. The inelastic interaction between the axial force and the Bi-directional horizontal forces in the vertical resisting elements is reflected in a reduction of the rotation of the floor of the order of 20% to 30% for systems that have uncoupled lateral period greater than 0.2 seconds.

The scope of the study presented here is restricted to real five-story reinforced concrete structures conformed by beams and columns, discarding the use of wall elements due to the difficulty to have a shear force-deformation relationship. The objective is to try to shed light on topics such as: a) parametric study of the 3D structural models with inelastic behavior, that consider torsional coupling in plan and axial coupling of the corner columns, b) study of the influence of the static stiffness eccentricity, and c) study of the influence of the angle of incidence when Bi-directional excitation is applied.

METHODOLOGY

Basic Structural Model

The 3D analysis model was created for a five-story reinforced concrete moment-frame structure subjected to actual ground movements represented by the two horizontal components of the ground acceleration. The elements were considered as “3D beam-columns” with six degrees of freedom (DOF) at each node. Flexural and shear deformations were considered for all the elements. The structural model considered a system of concentrated masses in each of the five stories. The floor diaphragms were considered to be rigid with three DOF at the center of mass (CM) of each story, two orthogonal horizontal displacements and the rotation around the vertical axis. The axial deformations of the columns were also taken into account. The buildings have the same floor plan throughout the height (Fig. 1). In the long direction, there are three identical resisting planes (frames in X direction). In the short direction, there are three resisting planes (frames in Y direction) with different stiffness: two resisting planes are at the edges of the plan (1 and 3), and one is at the CM (2). The resisting planes structural elements are the same throughout the height (Figs. 2, 3). The building has an inter-story height equal to 3m and the plan dimensions are 20m by 10m (aspect ratio 2:1). The 3D analysis was carried out using the computer program, ANSR-1 as implemented by Mondkar [8,9] and Rihai [10]. The non-linear behavior of each element was concentrated at the end nodes, and the force-deformation curve for the material behavior was of bi-linear type with a 90% loss of stiffness in the second branch. Degradation of stiffness and strength in the loading/unloading cycles was not considered (Fig. 4).

Analyzed Parameters

The structural element properties were varied in order to obtain different overall behavior characteristics of the structure. In this work, two symmetric and two asymmetric models corresponding to semi-flexible buildings that had similar lateral stiffness in both the longitudinal and the transverse direction, and that had similar lateral stiffness in both the longitudinal and the torsional direction, were studied. The parameters considered are: T_y (coupled traslational vibration period in Y direction), ω_x/ω_y (ratio of coupled frequencies of vibration in X and Y directions), ω_θ/ω_y (ratio of torsional and traslational coupled frequency of vibration in Y direction), e_x/r (ratio of static eccentricity in X and radius of gyration of the plan). Table 1 shows the values adopted for these parameters.

To make sure that the models correspond to real buildings and that, using the values of the parameters shown in Table 1, it was required that the models satisfy with the requirements of the Chilean Code (e.g. minimum design base shear and maximum relative inter-story displacements); other design requirements such as for example accidental torsion, were not taken into account. The parameters used, as defined by the code, for the earthquake definition were: Soil type II, Seismic Zone 3 (A_0 = 0.4g), and basic response modification factor R_0 = 11. Each model was analyzed independently in the X and Y directions considering linear elastic behavior and using as excitation the design spectrum of the code including the response modification factor R^*. Table 1 shows that the values adopted by R^*_x and R^*_y for each model are consistent with the corresponding average values in the Chilean code shown in Table 2. The Bi-directional maximum responses were estimated from the Uni-directional maximum responses (computed by RSA using the CQC combination rule) for each analysis direction using the SRSS combination rule (the less appropriate of the combination rules according to [11,12,13]). The geometric characteristics of the beam-columns elements (Fig. 5) used in the models are shown in Table 3.

The frame behavior in the resisting plane regarding the type of deformation of beams and columns when subjected to horizontal forces, is represented by the "beam-column stiffness ratio, ρ" [14] defined as the ratio of beam stiffness to column stiffness in the story closest to the mid-heigth of the frame. If the beams stiffness is small compared to the columns stiffness, ρ tends to 0, allowing the free rotation of the columns at each end so that the frame columns behave as uncoupled elements. On the other hand, if the beam stiffness is large compared to the column stiffness, ρ tends to ∞. The typical frame behavior is that of coupled columns due to the restriction on the rotation at the ends of the columns. Intermediate values of ρ represent intermediate levels of coupling of the columns. Table 3 shows that structures with different values of ρ for the X and Y directions (ρ_X, ρ_Y) varying from ρ = 0.001 up to 1 were analyzed. Table 4 shows the characteristics of the mass distribution of the models, and Table 5 shows the overall dynamic characteristics of both models as obtained from a SAP2000 model analysis [15].

External Loads

When studying structures with structural components that behave non-linearly, it is not possible to separate the effects of the gravitational load from the seismic load effects. Therefore, all the external loads that will be present during the occurrence of an important earthquake which may result in non-linear response of the structure have been considered to act simultaneously in this investigation.

Gravitational loads: The effects of dead and live loads in the structure are considered, taking an average value for a uniformly distributed load in plan, $\mathbf{w_g}$ = 1.0 T/m^2. Changes in the plan distribution of mass were ignored, so that the eccentricity of the structure is originated only by an unequal distribution of the frames lateral stiffness.

Earthquake loads: The two horizontal components of ground motion recorded in Llo-Lleo during the earthquake occurred in the central region of Chile in March 3, 1985 (LLN10E, with 0.668g PGA and

LLS80E with 0.424g PGA) are considered. In all cases analyzed, the two horizontal components of the earthquake were applied simultaneously. The component of the record that has the largest peak ground acceleration (PGA) was normalized to a PGA of 0.4g. The relative intensities of the two components were maintained as in the original records, by normalizing the X components with the same scale factor used for the Y component.

Non-linear Behavior Model

The non-linear analysis model allows the response in each time step to be known considering the stiffness variations experienced when some critical sections yield. The structure was solved with the program for non-linear dynamic analysis of three-dimensional structures, ANSR-1 [8]. The step-by-step solution strategy used was based on the numerical integration of the differential equations of motion in term of geometrical coordinates using a Newton integration scheme. The non-linear behavior of each element was assumed to occur at its end nodes only and the force-deformation behavior curve to be bi-linear with a loss of 90% of stiffness in the second branch (Fig. 4). The study also included the interaction surface for bending moments M_y and M_z and axial force N defined in the program which was built with two uni-axial curves, M_u-N and M_v-N related to a common axial force N (Fig. 6).

Analysis of the models

Results for each model were obtained from : i) RSA dynamic analysis applying independently each of the earthquake horizontal components (Uni-directional analysis) in both directions (X-Y, respectively); ii) Time history of the response (HRT) considering Non-linear behavior of the material and applying simultaneously the two earthquake components (Bi-directional analysis), varying in each case the incidence angle α of the earthquake action in 15° increments starting at the X-axis resulting in different cases (Fig. 7). The time interval ΔT used in the analysis was 0.002 sec. For the first and last modes, 5% of critical damping ratio was considered (Rayleigh type damping matrix).

Element Sections Design Strengths

*Response modification factor R**: The response modification factor *R** used in the Chilean Code (1996) to define the design spectrum depends on the type of foundation soil, the fundamental period of the structure, the type of structural system, and the material used. Table 1 shows the *R** values used for both models.

Beams: The design moment $\boldsymbol{M_d}$ is the maximum bending moment for the elements of each resisting plane resulting from the analysis of the structure subjected to gravitational loads (M_g) added to the maximum moment resulting from the analysis of the structure subjected to the seismic loads when only elastic behavior is considered (M_{sel}), divided by the response modification factor R^*; this is, $\boldsymbol{M_d = M_g + M_{sel} / R^*}$.

Columns: *M-N* interaction curves had to be determined for the columns from maximum forces obtained in each fixed end. The slopes of the interaction curves were defined based on the interaction curves commonly used in reinforced concrete column design, considering symmetrical reinforcement in each direction and a minimum steel ratio equal to 1% of the gross section. The shape for the interaction curve corresponds to that of a rectangular section of H25 concrete ($f`_c$ = 20 MPa) reinforced with A63-42H steel (f_y = 420 MPa) chosen as representative of a wide range of cases. The maximum of the time history responses of axial force N_{sel} and bending moment M_{sel} are determined considering elastic behavior, but these do not occur at the same time. It is impractical to obtain the most unfavorable combination of *N* and *M* for each column in the time history, due to the large number of steps used, therefore only the maximum responses were used. To evaluate the most unfavorable combination of *N* and M_y and M_z for each column, and considering that the seismic response value can be positive or negative, the following possibilities were evaluated: $M_{y1}=M_{yg}+M_{ysel}/R^*$, $M_{y2}=M_{yg}-M_{zsel}/R^*$, $M_{z1}=M_{zg}+M_{zsel}/R^*$, $M_{z2}=M_{zg}-M_{zsel}/R^*$, $N_1=N_g+N_{sel}/R^*$

and N_2=N_g-N_{sel}/R*. The design strength values were defined as an envelope to these required maximum values. The surface is further amplified by 25% to represent the use of the "strong column - weak girder" design concept. To simplify the design process a constant value of the strength reduction factor ϕ = 0.85 is considered.

Responses to study

As overall responses the maximum diaphragm displacements in the top story are studied. As local responses, in the beams the bending moments, maximum plastic hinge rotations, and accumulated plastic hinge rotations are considered; while in the columns only the bending moments and maximum plastic hinge rotations are considered.

ANALYSIS OF THE RESULTS

Maximum Diaphragm Displacements

Fig. 8 shows the comparison of the diaphragm displacements of the fifth floor of the two models. For model 1 it is observed that the symmetry is lost when incursions in the non-linear behavior range occur. In addition, the magnitude of the maximum displacements under Bi-directional action depends on α, the angle of incidence of the earthquake (Table 6). For the studied cases, it is observed that in general both models have very similar behavior for all the different displacements considered.

Beams

Fig. 9 shows the comparison of the Bending Moments, the Maximum Plastic Hinge Rotations, and the Accumulated Plastic Rotations of beams V3 and V5 on the second floor of the two models. In model 1 it is observed that the maximum bending moment of the beam located in the flexible side of the plan (V3) is slightly smaller than one corresponding to the stiff side (V5), with maximum averages values of +969 ton-cm and -1022 ton-cm respectively. In model 2 it is observed that the maximum bending moment for beam V3 is 30% larger than the value obtained for beam V5, with maximum averages values of +1116 ton-cm and -1175 ton-cm respectively. In both models the magnitude of the maximum value is approximately constant and independent of α. Although the negative maximum Plastic Hinge Rotations are small it is observed that in both models they depend on α, being the largest values those of beam V5. Finally, the negative Accumulated Plastic Hinge Rotations of beam V3 are almost five times smaller than those of beam V5; where the maximum value occurs when α tends to 0º or 180º.

Columns

Fig. 10 shows the comparison of the Bending Moments (on a y-axis) and the Maximum Plastic Hinge Rotations of Columns P1 and P2 of the first floor of the two models. In model 1 it is observed that the maximum bending moment of the column located in the flexible side of the plan (P1) is slightly smaller than that corresponding to the stiff side column (P5), with maximum averages values for both columns equal to +1176 ton-cm and -1136 ton-cm. In model 2 it is observed that the maximum bending moment of column P1 is sixteen times smaller than the value obtained for column P5, with maximum averages values equal to +1432 ton-cm and -1342 ton-cm. In model 1, the magnitude of the response is approximately constant and independent of α, whereas in model 2 the value of the bending moment of column P5 is obtained for α equal to 240º. Although the maximum Plastic Hinge Rotations are small it is observed that in both models they depend on α, being the largest values those in column P1.

Fig. 11 shows the comparison of the Bending Moments, and the Maximum Plastic Hinge Rotations of Columns P1 and P2 about the z-axis of the first floor of the two models. In model 1 it is observed that the maximum bending moment of the column located in the flexible side of the plan (P1) is slightly smaller than that corresponding to the stiff side column (P5), with maximum averages values for both columns equal to +1391 ton-cm and –1388 ton-cm. In model 2 it is observed that the maximum bending moment of column P1 is seven times smaller than the value obtained for column P5, with maximum averages values for the P1 column equal to +1459 ton-cm and –1534 ton-cm. In model 1, the magnitude is approximately constant and independent of α, whereas in model 2 the value of the bending moment of column P5 is obtained for α equal to 135º. Although the maximum Plastic Hinge Rotations are small it is observed that in both models they depend on α, with both columns showing very large values for α equal to 135º.

CONCLUSIONS

1. The symmetric structure (Model 1) shows asymmetric behavior when its elements enter the non-linear behavior range. In general it is observed that the forces in beams and columns located in the flexible side of the plan are slightly smaller than those obtained in the elements of the stiff side of the plan.
2. In the asymmetric structure (Model 2) it was observed that the forces in the columns located in the flexible side of the plan are much smaller than those obtained in the elements of the stiff side of the plan.
3. The maximum values obtained for the maximum Plastic Hinge Rotations and the Accumulated Plastic Hinge Rotations are in general small, for all cases considered.
4. In general, both the global and the local maximum responses obtained using Bi-directional excitation depend rather strongly on the angle of incidence α of the earthquake action.

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Table 1: Values of the parameters that define the model.

Model	T_y (s)	ω_x/ω_y	ω_θ/ω_y	e_x/r	R^*_X	R^*_Y
1	0.84	0.94	0.96	Symmetric	9.029	8.893
2	0.85	1.08	1.71	Asymmetric	8.762	8.935

Table 2: Strength Reduction Factors R^* for Ductile Concrete Frame Buildings Founded in Different Types of Soils (Adapted from Chilean Seismic Code [1]).

Foundation Soil Characteristics				
T_v (sec)	I Rock	II Dense gravel $q_u > 0.2$ MPa	III Sand or gravel 0.05 Mpa < q_u < 0.2 MPa	IV Cohesive soil $q_u < 0.05$ MPa
0.25	7.6	5.7	3.6	2.8
0.75	10.0	8.6	6.2	5.0
2.00	11.2	10.4	8.8	7.6
Average	9.6	8.3	6.2	5.1

Table 3: Geometric Characteristics of the Structural Elements.

Model 1

Column	D_1 (cm)	D_2(cm)
P1	40.00	40.00
P2	35.00	70.00
P3	50.00	50.00
P4	60.00	75.00
P5	40.00	40.00
P6	35.00	70.00

Beam	L (cm)	b (cm)	h (cm)
V1	1000.00	25.00	57.50
V2	1000.00	25.00	79.50
V3	500.00	25.00	45.50
V4	500.00	25.00	63.00
V5	500.00	25.00	45.50

RP	ρ
A	0.1254
B	0.1697
1	0.0825
2	0.0992
3	0.0825

Model 2

Column	D_1 (cm)	D_2(cm)
P1	40.00	40.00
P2	35.00	70.00
P3	50.00	60.00
P4	60.00	80.00
P5	60.00	150.00
P6	60.00	150.00

Beam	L (cm)	b (cm)	h (cm)
V1	1000.00	25.00	57.50
V2	1000.00	25.00	79.50
V3	500.00	25.00	45.50
V4	500.00	25.00	63.00
V5	500.00	25.00	45.50

RP	ρ
A	0.0336
B	0.0715
1	0.0825
2	0.0717
3	0.0023

Table 4: Mass Distribution Characteristics of the Models.

Story	CM Coordinates			Model 1			Model 2		
	X (cm)	Y (cm)	Z (cm)	M_X (M_Y) (tonf-s^2/m)	J (tonf-m-s^2)	W (tonf/m^2)	M_X (M_Y) (tonf-s^2/m)	J (tonf-m-s^2)	W (tonf/m^2)
5	0	0	15	14.963	954.168	0.734	15.829	1046.702	0.776
4	0	0	12	15.758	1008.276	0.773	17.491	1186.524	0.858
3	0	0	9	15.758	1008.276	0.773	17.491	1186.524	0.858
2	0	0	6	15.758	1008.276	0.773	17.491	1186.524	0.858
1	0	0	3	15.758	1008.276	0.773	17.491	1186.524	0.858

Table 5: Dynamic Characteristics of the Models.

Mode	Model 1				Model 2			
	T (sec)	U	V	θ	T (sec)	U	V	θ
1	0.892	0.821	0.000	0.000	0.855	0.000	0.484	0.318
2	0.870	0.000	0.000	0.809	0.791	0.782	0.000	0.000
3	0.838	0.000	0.805	0.000	0.500	0.000	0.279	0.426
4	0.277	0.108	0.000	0.000	0.256	0.000	0.070	0.046
5	0.264	0.000	0.000	0.113	0.226	0.126	0.000	0.000
6	0.252	0.000	0.116	0.000	0.131	0.000	0.030	0.020
7	0.150	0.044	0.000	0.000	0.108	0.055	0.000	0.000
8	0.137	0.000	0.000	0.048	0.105	0.000	0.076	0.113
9	0.130	0.000	0.048	0.000	0.082	0.000	0.015	0.010
10	0.098	0.021	0.000	0.000	0.064	0.028	0.000	0.000
11	0.087	0.000	0.000	0.023	0.061	0.000	0.005	0.003
12	0.082	0.000	0.024	0.000	0.046	0.009	0.000	0.000
13	0.076	0.007	0.000	0.000	0.042	0.000	0.027	0.041
14	0.066	0.000	0.000	0.007	0.024	0.000	0.012	0.018
15	0.062	0.000	0.007	0.000	0.018	0.000	0.003	0.005

Table 6: Maximum Diaphragm Displacements of the fifth floor.

α	Model 1 (Symmetric)						Model 2 (Asymmetric)					
	U+	**V +**	**θ +**	**U-**	**V -**	**θ -**	**U+**	**V +**	**θ +**	**U-**	**V -**	**θ -**
0º	6.980	2.982	0.003	-5.029	-4.261	0.004	5.872	4.535	0.004	-6.885	-3.491	0.004
15º	6.858	3.104	0.003	-4.528	-4.655	0.005	5.221	4.002	0.003	-6.728	-4.213	0.006
30º	7.106	3.421	0.004	-4.178	-4.284	**0.006**	5.101	3.533	0.004	-5.343	-4.714	0.006
45º	6.437	3.954	0.004	-4.368	-3.767	0.006	5.639	3.170	0.004	-5.219	-4.912	**0.007**
60º	6.959	3.789	0.004	-5.048	-3.732	0.005	5.489	2.535	0.004	-6.445	-4.659	0.006
75º	7.259	3.102	0.003	-5.331	-3.851	0.004	6.964	2.677	0.003	-6.123	-4.218	0.005
90º	7.408	2.874	0.003	-5.189	-4.257	0.004	6.969	3.191	0.003	-6.220	-4.397	0.004
105º	7.827	2.855	0.004	-5.438	**-4.709**	0.004	7.194	3.774	0.003	-6.493	-5.092	0.004
120º	**8.81**	3.13	0.004	-5.24	-4.66	0.004	**7.87**	4.533	0.004	-6.99	**-5.163**	0.004
135º	8.70	3.50	0.004	-5.44	-3.92	0.003	7.80	4.922	0.004	-7.11	-4.946	0.004
150º	7.392	3.886	0.004	-6.245	-3.053	0.003	7.238	4.748	0.004	-7.528	-4.938	0.004
165º	5.637	3.868	0.004	-6.767	-2.853	0.003	5.820	3.999	0.003	-7.823	-4.833	0.004
180º	5.266	4.232	0.004	-6.774	-2.991	0.003	5.664	3.543	0.004	-7.069	-4.491	0.004
195º	4.800	**4.688**	0.005	-6.589	-3.053	0.003	5.583	4.287	0.006	-6.336	-3.964	0.003
210º	4.375	4.343	**0.006**	-6.902	-3.348	0.004	4.103	4.790	0.006	-6.343	-3.491	0.004
225º	4.540	3.886	0.006	-6.258	-3.837	0.004	4.013	4.984	**0.007**	-6.876	-3.113	0.004
240º	5.126	3.859	0.005	-6.870	-3.682	0.004	5.161	4.713	0.006	-6.737	-2.505	0.004
255º	5.357	3.954	0.004	-7.246	-3.044	0.003	4.902	4.232	0.005	-8.140	-2.658	0.003
270º	5.225	4.238	0.004	-7.371	-2.901	0.003	4.943	4.428	0.004	-8.170	-3.125	0.003
285º	5.460	4.653	0.004	-7.819	-2.912	0.004	5.215	5.107	0.004	-8.437	-3.718	0.003
300º	5.267	4.607	0.004	**-8.744**	-3.201	0.004	5.608	**5.146**	0.004	**-9.084**	-4.498	0.004
315º	5.492	3.874	0.004	-8.585	-3.564	0.004	5.744	4.939	0.004	-8.983	-4.907	0.004
330º	6.320	2.977	0.004	-7.253	-3.970	0.004	6.187	4.918	0.004	-8.387	-4.747	0.004
345º	6.913	2.812	0.003	-5.440	-3.937	0.004	6.561	4.844	0.004	-6.941	-3.987	0.003
360º	6.981	2.981	0.003	-5.028	-4.259	0.004	5.874	4.480	0.004	-6.883	-3.523	0.004

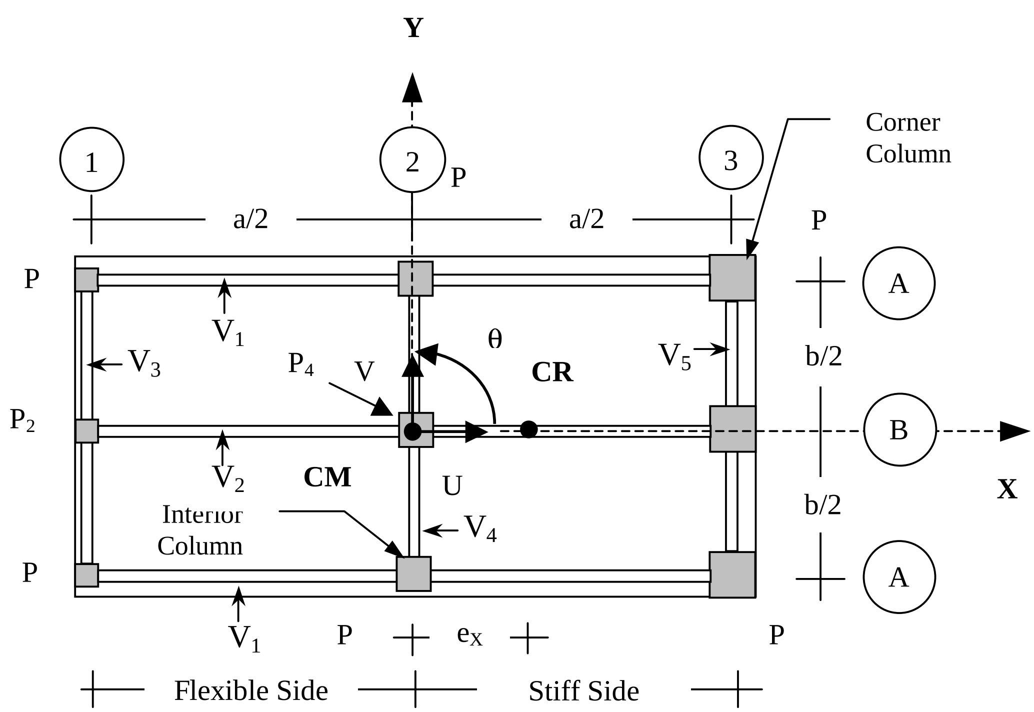
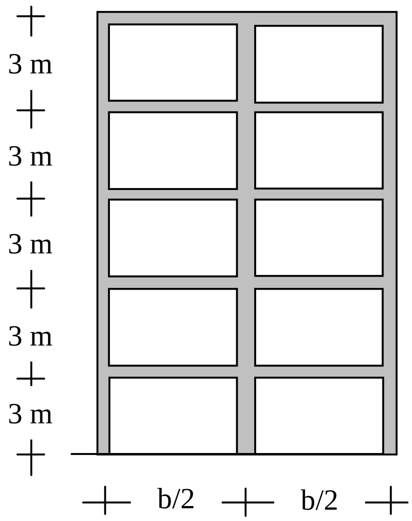
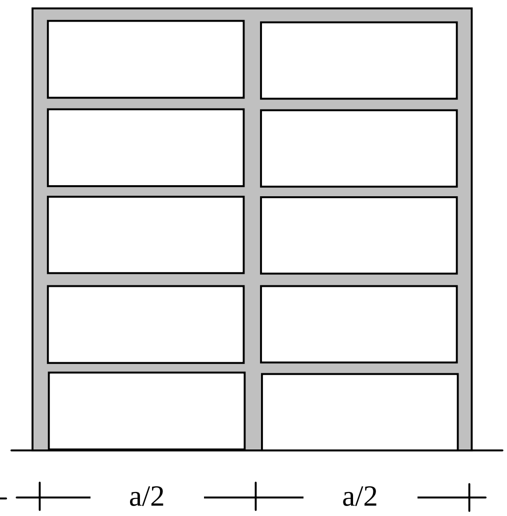


Figure 1: Typical Plan of the Three-Dimensional Structural Model.





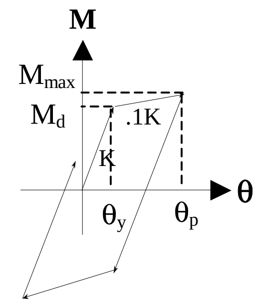


Figure 4: Force-Deformation Behavior of the Flexural Elements.

Figure 2: Resisting Planes 1, 2, 3 y 4.

Figure 3: Resisting Plane A.

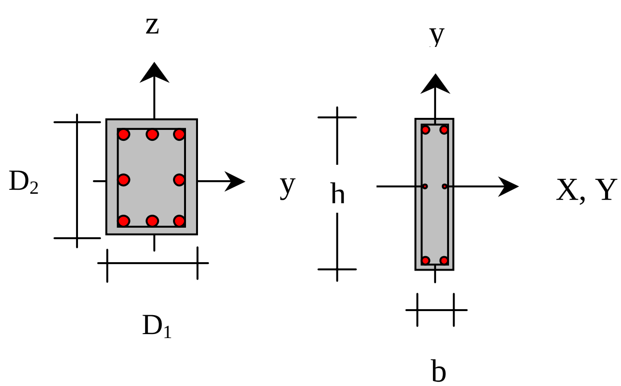
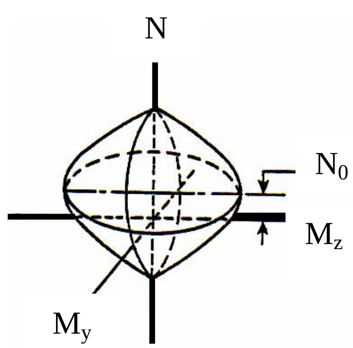


Figure 5: Cross-sections of the Columns and Beams. ($D_1/D_2 \geq 0.4$ for all cases).



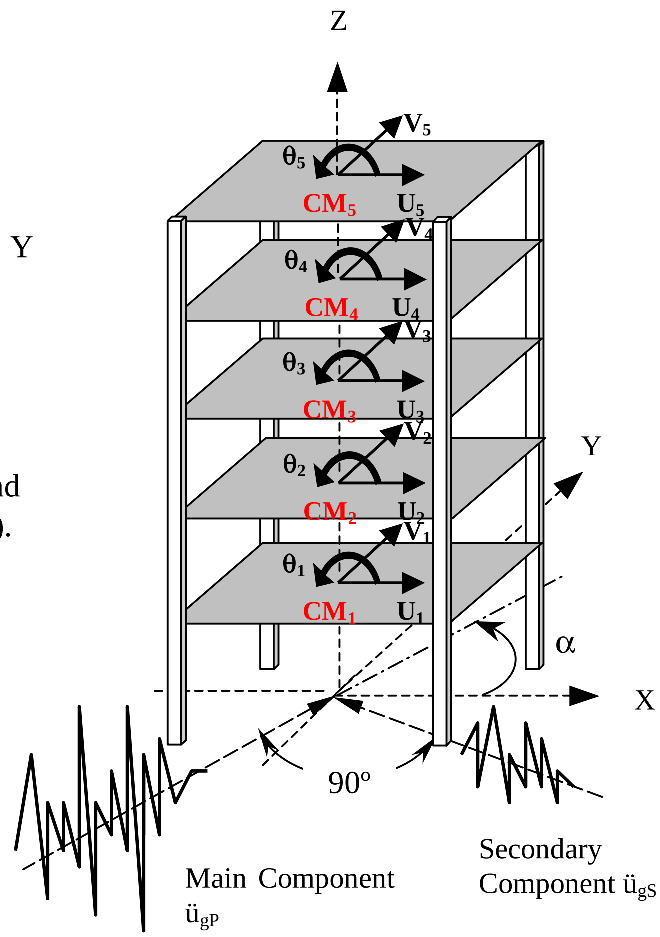


Figure 6: Typical Interaction Surfaces for Elasto-Plastic Component (Type 2).

Figure 7: Schematic Representation of the Three-Dimensional Building Model subjected to Bi-directional Ground Motion.

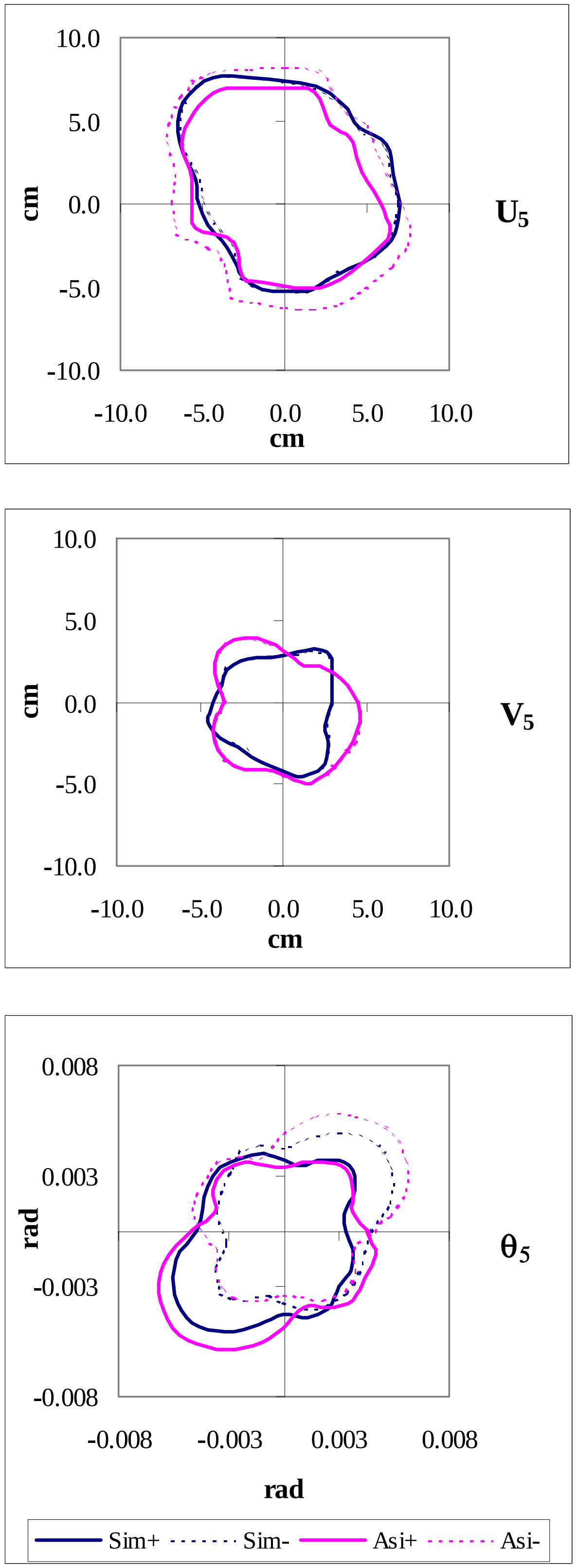


Figure 8: Comparison of the Fifth Floor Diaphragm Displacements for Model 1 (Sim) and Model 2 (Asi).

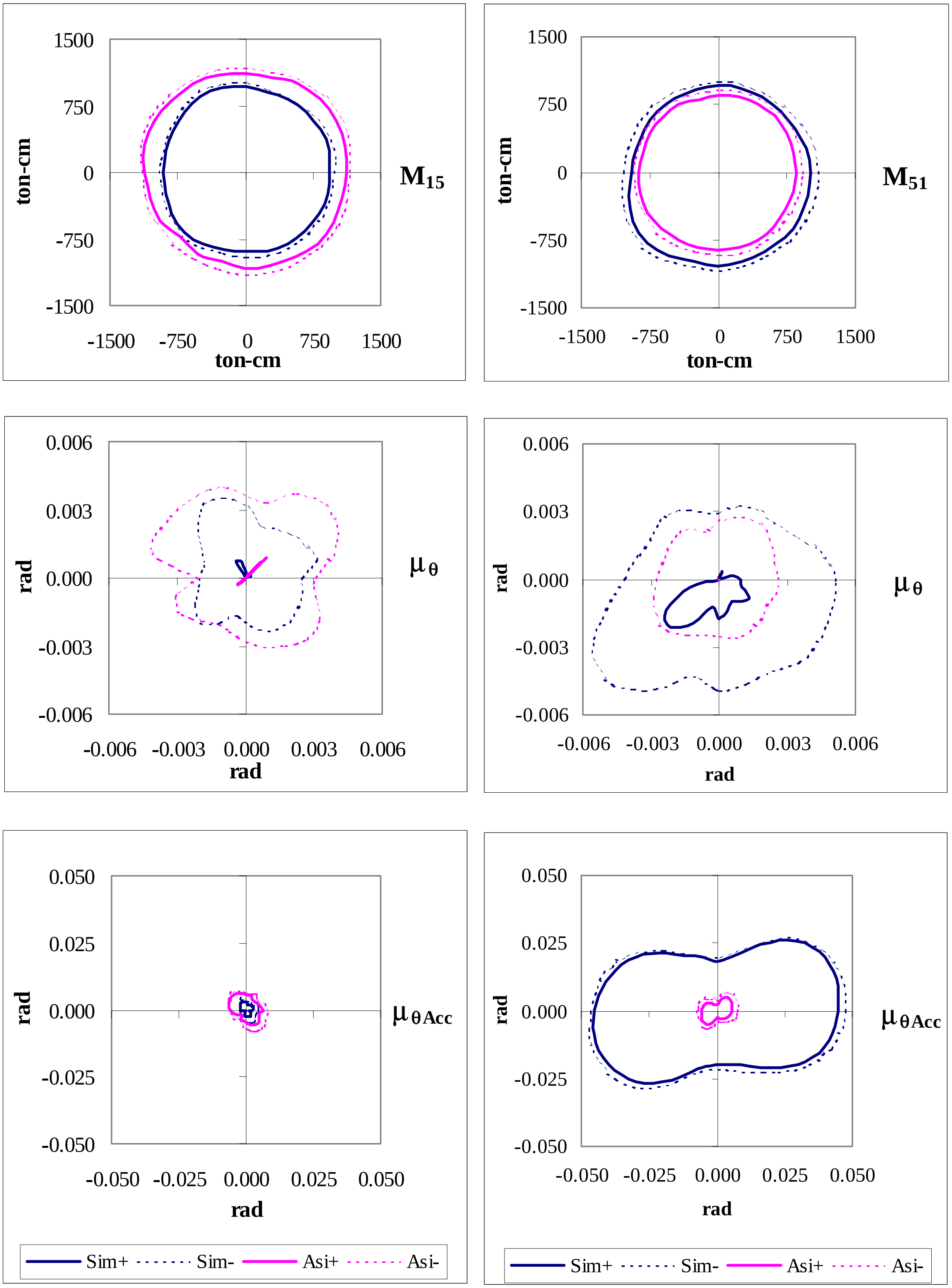


Figure 9: Comparison of the Bending Moments, the Maximum Plastic Hinge Rotations, and the Accumulated Plastic Rotations of Beams V3 and V5 (second floor) for Model 1 (Sim) and Model 2 (Asi).

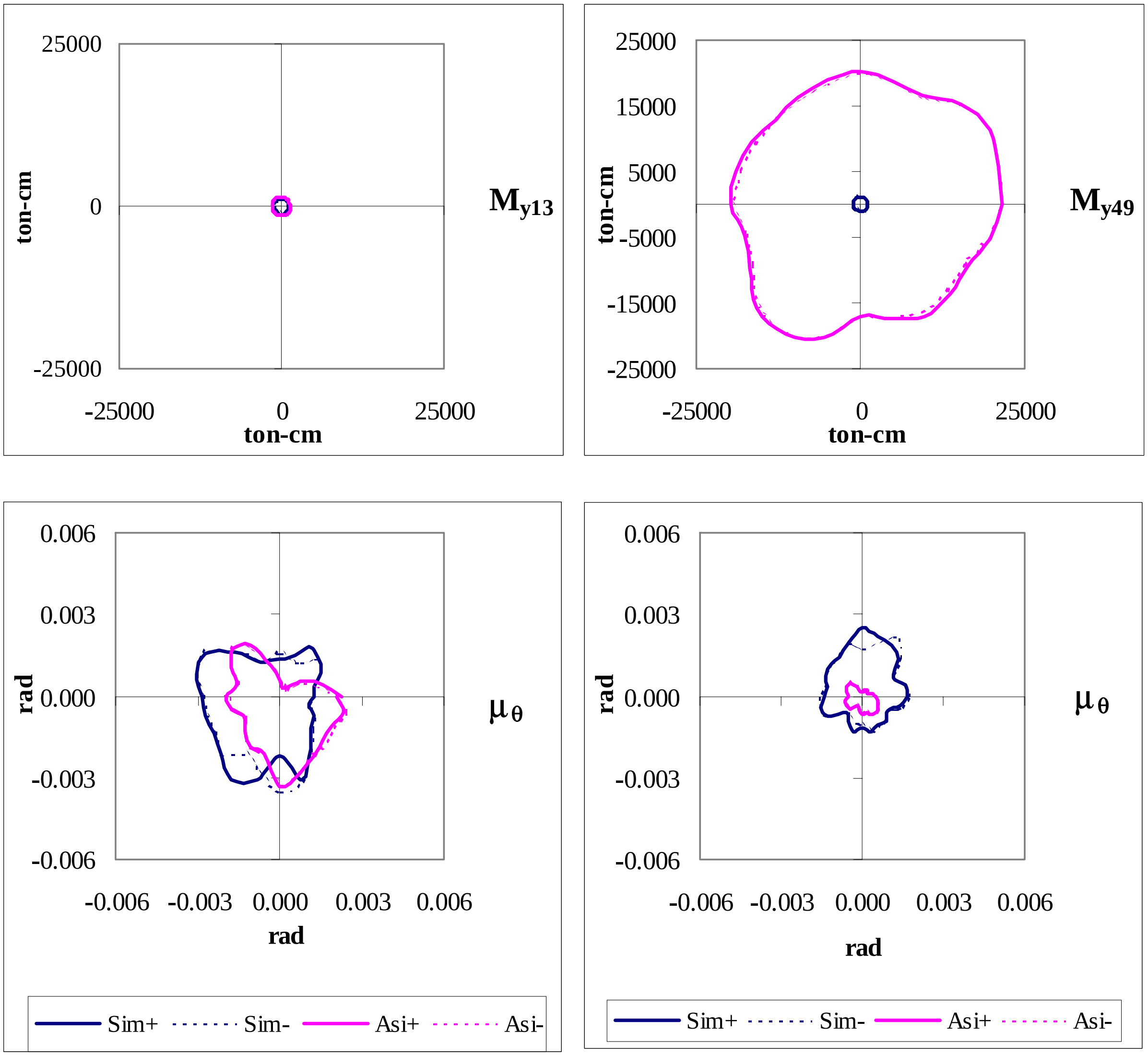


Figure 10: Comparison of the Bending Moments and the Maximum Plastic Hinge Rotations of Columns P1 and P5 (about y-axis, first floor) for Model 1 (Sim) and Model 2 (Asi).

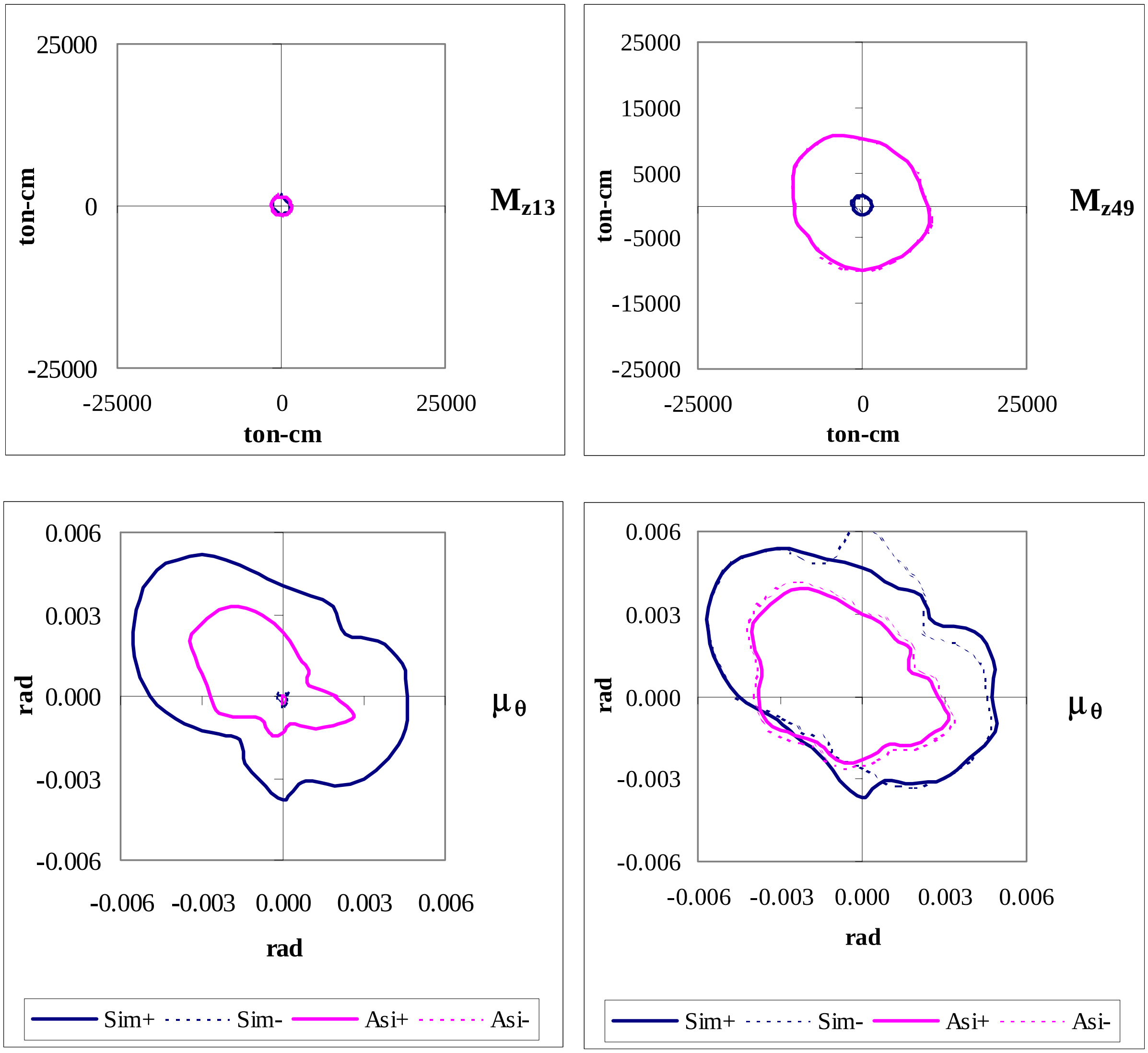


Figure 11: Comparison of the Bending Moments and the Maximum Plastic Hinge Rotations of Columns P1 and P5 (about z-axis, first floor) for Model 1 (Sim) and Model 2 (Asi).