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NCh2369 vs ASCE7 - STRENGTH vs DUCTILITY?

INDUSTRIAL STEEL BRACED FRAMES

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Abstract

Earthquake-resistant design philosophies behind ASCE7-10/AISC341-10 and NCh2369.Of2003 are different. The differences make very difficult the task of satisfying simultaneously both design criteria, both from the conceptual and the practical points of view.

This paper presents the comparison of the inelastic seismic response of a concentrically brace frame of an industrial structure. The structure was designed to meet independently with both American and Chilean Codes provisions. The comparison of the seismic response is made at three levels of earthquake excitations: for a standard earthquake demand, (10% of exceedance in a 50-year period), for a maximum credible demand, (2% of exceedance in 50-year period), and for an operational level earthquake, (50% of exceedance in a 30-year period). Each of these earthquake levels is represented by a set of artificial ground motions compatibles with the corresponding design spectra. Additionally, some design aspects showing the differences among the design philosophies are presented.

Results of this study show that, in spite of the fact that American design base shear is 34% less than the corresponding Chilean-designed structure, seismic weight using American design is 22% larger than the Chilean design value due to design conditions not associated to seismic requirements. In spite of the fact that Chilean-designed structure has a larger lateral strength, American structure response seems to provide a better satisfaction of performance objectives defined for each level of earthquake demand.

Keywords: Industrial structures; Braced frames.

1. Introduction

Earthquake-resistant design philosophies behind ASCE7-10/AISC341-10, [1,4], and NCh2369.Of2003, [2], show significant differences. American provisions intend to provide a moderate lateral strength with development of high levels of ductility under severe earthquake ground motions. Consequently, they require a stringent detailed design of members and connections. On the other hand, seismic design used in Chile requires higher lateral strength associated to moderate requirements of seismic inelastic behavior under similar events. Then, the required level of seismic detailing is smaller than that used in American practice. This Chilean practice showed successful results as evidenced by the behavior of industrial structures during the February 27, 2010 earthquake. Structures designed with Chilean Code NCh2369 showed limited incursions into the inelastic behavior, very small or no structural damage and limited shutdown period of industrial facilities after the earthquake. All these characteristics meet the performance objectives of industrial facilities required by Code NCh2369 under severe earthquakes.

In Chile it is not unusual that special important projects, such as Thermoelectric Power Plants, be designed using Seismic Design Criteria based on ASCE 7 “Minimum Design Loads for Buildings and other Structures”, [1]. Additionally, seismic design must meet minimum requirements specified by Chilean Code NCh2369, [2]. In such cases, the differences in the design philosophy of both documents make it difficult to satisfy simultaneously the provisions of both documents, both from conceptual and practical points of view. This paper presents design aspects that highlight these conceptual differences between both philosophies, as well as results of inelastic time-history analyses and ductility demands for a concentrically, braced-frame industrial building, designed according to the requirements of both Chilean and American Codes.

2. Differences between ASCE7-10/AISC341-10 and NCh2369.Of2003

Design philosophies mentioned above originate different provisions and design requirements in both Chilean and American Codes. Some of them are summarized in the following:

a) ASCE 7-10, [1], is a document with a strong theoretical and conceptual background. It is mostly based on American research and has evolved from the seismic design provisions developed by SEAOC in California since 1960. However, it does not have the practical assessment of their provisions through the study of the behavior experienced by structures designed according to them, after severe earthquake ground motions. On the other hand, Chilean Code provisions are the result of seismic design criteria used in Chile since 1960 in many industrial projects. These provisions have been tested by several severe earthquakes, particularly in March 1985 and February 2010. These tests have permitted to evaluate and improve these seismic design provisions, and allow the engineers to expect a satisfactory future seismic behavior of structures and facilities designed according to the Chilean Code. It is worthwhile to note that Chile has different earthquake-resistant Codes for apartment or office buildings, (NCh433, [5]), or industrial structures and facilities, (NCh2369, [2]). In the first Code the performance objective is to prevent loss of lives under extreme earthquakes, while in the industrial Code the controlling objective is to maintain continuity of operation or limited shutdown periods under such events.

b) Lateral strength seismic design demand of ASCE 7-10 is relatively moderate as compared to that of NCh2369.Of2003, although elastic design spectra used to calculate the seismic forces are similar. The difference arises from the simultaneous effect of the following factors:

- Larger values of ASCE 7-10 response modification factor R.
- Damping ratio of ASCE 7-10 is 5% for all steel structures. NCh2369.Of2003 considers damping ratios of 2% and 3%, more representatives of industrial structures.
- Load combinations are different in both documents. LRFD amplification is 1.4 for the Chilean Code, while this factor is 1.0 in ASCE 7-10.

- Minimum base shear in NCh2369.Of2003 is generally larger than minimum base shear required by ASCE 7-10.

c) The design of structural steel elements to prevent local or global buckling prior to development of inelastic behavior is also different in both Codes. ASCE 7 requires the use of AISC 360-10, [3], and AISC 341-10, [4], while NCh2369.Of2003, Chapter 8, [2], specifies less stringent requirements that have proved to be enough for the reduced ductility demand observed during past severe earthquakes. This reduced ductility demand is consistent with the larger lateral strength requirements specified by NCh2369.

d) The reasoning behind paragraph (c) is also applicable to the seismic design of connections. Although both Codes design a connection that is stronger than the connecting members, ASCE 7-10 requires the connection to transfer the expected capacity of the member, while NCh2369.Of2003 only requires transferring the nominal member capacity. In a simplified way, the difference between both levels may be represented by factor Ry, which for Chilean steel materials varies between 1.3 and 1.5.

e) Another special case is the seismic design of inverted V configuration of concentrically braced frames. Chilean design concept is to limit the ductility demand in the V braces by increasing the design reduced force by 50%, and to design the beam for the gravitational actions without considering the contribution of the V braces. On the other hand AISC 341-10 specifies the beam to have enough flexural strength to absorb the vertical force induced by one brace yielding in tension while the other has the residual post-buckling strength. Both criteria are not compatible from the practical design point of view since the satisfaction of both would result in an excessive overdesign. Requirement used in Chile is originated by recommendations included in old American UBC Codes, applicable to braced frames with limited or null inelastic incursions expected during severe earthquakes.

f) As an additional comment, it has to be considered that occurrence of earthquakes in California and in Chile is due to different tectonic mechanisms. Ground motion characteristics, such as duration of shaking, peak ground acceleration, seismic zoning, frequency content, maximum acceleration, destructive potential, Arias' intensity, among others, may induce different structural responses for ground motions of comparable severity occurring in California and in Chile.

3. Characteristics of the structure and general design requirements

Figure 1 shows a scheme of the concentrically braced frame structure that has been used in this study, which is part of an industrial structure having three identical resisting vertical planes in the earthquake direction under study. The frame used in this study corresponds to the central frame. The structure is 4 meter-high, has four 6-meter bays in the longitudinal direction and two 6-meter bays in the transverse direction. All frames in the transverse direction are moment-resisting frames. Main longitudinal platform girders at each level are separated at 2 meters, are braced against lateral torsion at 1.5 meters and are supported by main girders of the moment-resistant frames. Each story has a horizontal truss diaphragm at the platform level.

Design loads of the structure include the following:

1) Own dead weight of the main structure, which depends on the design process. However, this load is not relevant in comparison with other loadings, (10tons approximately).

2) Own dead weight of platforms, floor grating and others. It is assumed as 100kgf/m^2. This value results in a distributed load of 200kgf/m in girders for each of the three floors, and concentrated loads of 2.4tonf in each of the three central columns and 1.2tonf in each of the side columns. Total dead weight at base level is 43.2tonf.

3) Equipment weight. Equipment are assumed in the part of the frame associated to the three central columns, with a total tributary load of 100tonf for the frame under study. This results in concentrated loads of 50tonf for the central column and 25tonf for the lateral columns.

4) Live load due to heavyweight equipment in levels 1 and 2, (L_{l1} and L_{l2}), assumed as 800kgf/m^2. This load results in distributed loading of 1600kgf/m on girders, and concentrated loads of 19.2tonf in each of the three central columns and 9.6tonf in each of side columns. Total live load at base level is 230.4tonf.

5) Live load due to lightweight equipment in level 3, (L_{l3}), assumed as 400kgf/m^2. This load results in distributed loading of 800kgf/m on girders, and concentrated loads of 9.6tonf in each of the three central columns and 4.8tonf in each of side columns. Total live load at base level is 57.6tonf.

6) Seismic weight calculation. During the occurrence of the earthquake it is assumed that gravitational load acting on the frame under study corresponds to 100% of dead weight, 50% of live load in levels 1 and 2 associated to heavyweight equipment and 25% of live load in level 3 associated to lightweight equipment. It is worthwhile to note that in this case the total gravitational load acting on the central frame corresponds to approximately one half of the load acting in the building. Nevertheless, the presence of the horizontal diaphragm at each level and the identical stiffness of the three frames in the longitudinal direction make the seismic load to be equally distributed among the three frames. Consequently, the total "lateral" seismic weight for the central frame is one third of the total seismic mass of the building. This results in an approximate lateral seismic weight of 192tonf for the central frame.

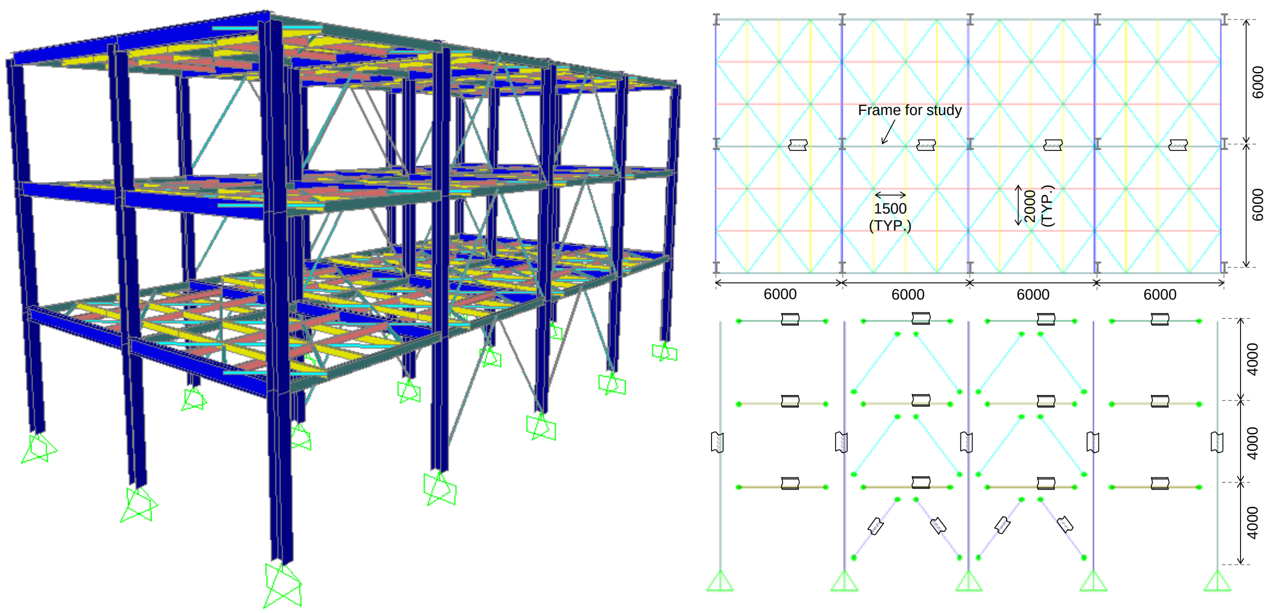

Fig. 1 – General scheme of structure used in the study

4. Design scenarios and structural designs obtained

The structure is located in the Chilean highest seismic zone, (A_0=0.4g), on hard soil, (type II according NCh2369 or type B according NCh433, [5], and DS61, [6]). It is a steel structure fabricated with steel ASTM A36, with welded shop connections, and bolted on-site connections. Design has followed state-of-the-art practice for the following scenarios:

I) Design according NCh2369.Of2003. Design follows exclusively the provisions for Category C2, (normal type of structure and equipment), and a concentrically braced type of structure.

II) Design according ASCE 7-10. Design follows exclusively the provisions of the American Code for a "nonbuilding similar to building" structure, of normal importance using concentrically braced special frames. An ASCE7-10 type of spectrum for hard soil and equivalent to Chilean highest seismic zone was used, i.e., S_{DS}=1.0g, S_{D1}=0.5g, y T_L=1.8seg. This design spectrum is similar to the spectrum corresponding to downtown San Francisco, California.

Design for both scenarios was performed using ASD Method, (AISC360-10 and AISC341-10). Basic design load combinations used in the design are shown in Table 1.

It is worthwhile to note that, according specific requirements of AISC341-10 and NCh2369.Of2003, inverted V, diagonal bracing members have to be designed for amplified seismic forces and particularly stringent boundary conditions, consistent with the expected collapse mechanism, see section 2(e).

Table 1-Basic design load combinations.

Scenario	**L.C.**	**D**	**$L_{l1\ \&\ l2}$**	**L_{l3}**	**E_h**	**E_v**
I	A1	1.000	1.000	1.000	-	-
	A2	0.750	0.375	0.188	0.750	-
II	A1	1.000	1.000	1.000	-	-
	A2	1.000	0.375	0.188	0.683	0.525

Table 2 shows a summary of parameters for both design scenarios. No correction is needed for the base shear since design value exceeds minimum value. Consequently, design should provide ductility consistent with the R-value assigned at the beginning of the design process.

Table 2-Summary of parameters for both design scenarios.

Parameter		**Scenario I (NCh2369.Of2003)**	**Scenario II (ASCE7-10)**
Importance factor	**I**	1.00	1.00
Modification factor	**R**	5	6
Damping ratio	**ξ [%]**	3	5
Overstrength factor	**Ω_o**	-	2.0
Deflection amplif. factor	**C_d**	-	5.0
Redundancy factor	**ρ**	-	1.3
Minimum seismic coef.	**$C_{mín}$**	0.100	0.044
Maximum seismic coef.	**$C_{máx}$**	0.230	0.281
Min. seismic base shear	**$V_{mín}$ [tonf]**	19.0	27.2
Max. seismic base shear	**$V_{máx}$ [tonf]**	43.8	54.1
First mode period	**T [seg]**	0.258	0.296
First mode mass	**M [%]**	90.9	90.1
Frame structural weight	**PP [tonf]**	8.5	10.4
Lateral sesimic weight	**W_s [tonf]**	190.3	192.3
Design base shear	**E_h [tonf]**	39.75	28.92
Vertical seismic force	**E_v [tonf]**	38.06	38.46

Designed obtained for both structures is summarized in Table 3. It is necessary to point out that design has been decided according to realistic Chilean practice for industrial structures. This should lead to a structure representative of Chilean practice. Members have not been designed at their full capacity, to account for eventual future upgrade process during lifetime of the facility. Likewise, minimum member dimensions, a reasonable standardization of sections and connection design to guarantee a proper construction have been considered. Finally, note that design of members is controlled by standard operational conditions, except for the case of seismic bracing and for the girder of braced panel according to scenario II.

Table 3-Summary of designed structures.

Member	Scenario I (NCh2369.Of2003)			Scenario II (ASCE7-10)		
	Section	UR A1	UR A2	Section	UR A1	UR A2
Central girder Level 1	IN30x44.6*	0.79	0.13	IN60x98.5**	0.13	0.92
Central girder Level 2	IN30x44.6*	0.79	0.08	IN60x90.9**	0.04	0.91
Central girder Level 3	IN25x32.6*	0.72	0.03	IN60x90.9**	0.20	0.91
Lateral girder Levels 1 and 2	IN30x44.6*	0.79	0.35	IN30x44.6*	0.79	0.39
Lateral girder Level 3	IN25x32.6*	0.72	0.23	IN25x32.6*	0.72	0.29
Central columns	HN30x92.2	0.85	0.49	IN30x102**	0.77	0.54***
Lateral columns	IN20x45.6	0.84	0.36	IN20x45.6	0.84	0.40***
Bracing member Level 1* **	ϕ160x6	0.19	0.75	ϕ140x5	0.35	0.92
Bracing member Levels 2 and 3* **	ϕ140x5	0.29	0.70	ϕ120x5	0.47	0.79

(*) Design controlled by vertical deflection. (**) Design controlled by ductility requirements Sect. F2.3(ii) in AISC341-10. (***) UR A2 corresponds to load combination with amplified seismic load.

Modal shapes shown in Figure 2 and associated periods of vibrations are almost identical for both scenarios, due to the fact that lateral stiffness is essentially provided by diagonal bracing members.

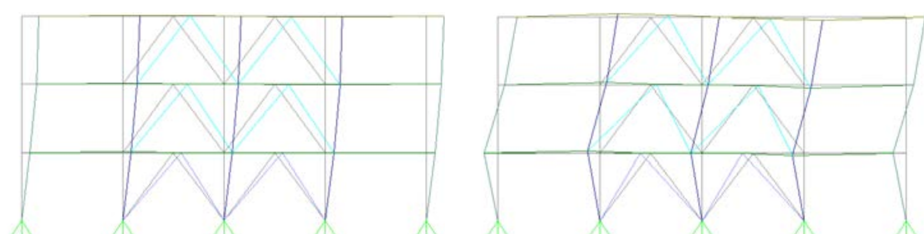

Fig. 2 –First two modal shapes.

5. Maximum demand expected during earthquake events

After design of the structures was completed according to both scenarios, time-history inelastic analyses were carried out for three levels of seismic demands defined as follows:

1) Operating Level Earthquake (OLE). Corresponds to a level of seismic demand for which it is expected the industrial facility shall remain 100% operative after its occurrence. This demand level has been chosen to have a 50% probability of exceedance in a 30-year period, (average return period of 43 years).

2) Design Level Earthquake (DLE). Corresponds to a level of seismic demand for which it is expected the industrial facility shall present limited inelastic incursions and damage, so that eventual repairs will be null o very minor, with a very short shutdown period after such event. This demand level has been chosen to have a 10% probability of exceedance in a 50-year period, (average return period of 475 years).

3) Maximum Considered Earthquake (MCE). Corresponds to a level of seismic demand for which it is expected the industrial facility shall not collapse. This demand level has been chosen to have a 2% probability of exceedance in a 50-year period, (average return period of 2475 years).

In order to define the spectra associated to demand levels above, a basic DLE spectrum has been chosen as indicated next, and spectra for levels MCE and OLE have been obtained by multiplying the basic spectrum by 1.5 and 0.6, respectively. These factors have been obtained from seismic risk curves derived for the Chilean highest seismic zone. Each demand level, for each design scenario, has been represented by three artificial records, compatible with the corresponding spectrum:

I) Design NCh2369.Of2003. Artificial earthquake records are constructed from three "seed" actual Chilean records: Rec.1: Llolleo 2010 SMA-1 Channel 3, Rec. 2: Talca 2010 SMA-1 Channel 1, Rec. 3: Constitucion 2010 SMA-1 Channel 3, [7]. These records are representative of Chilean seismic zone and type of soil considered for this study. The resulting artificial records show good adjustment to spectra defined in DS61 to verify displacements for seismic zone 3 and soil type B, as shown in Figure 3.

II) Design ASCE7-10. Artificial earthquake records are constructed from three "seed" actual Californian records: Rec.1: Northridge 1994, Rec. 2: Loma Prieta 1989, Rec. 3: Sylmar 1971. These records are representative of American seismic zone and type of soil considered for this study. The resulting artificial records show good adjustment to design spectra defined for scenario II, as shown in Figure 4.

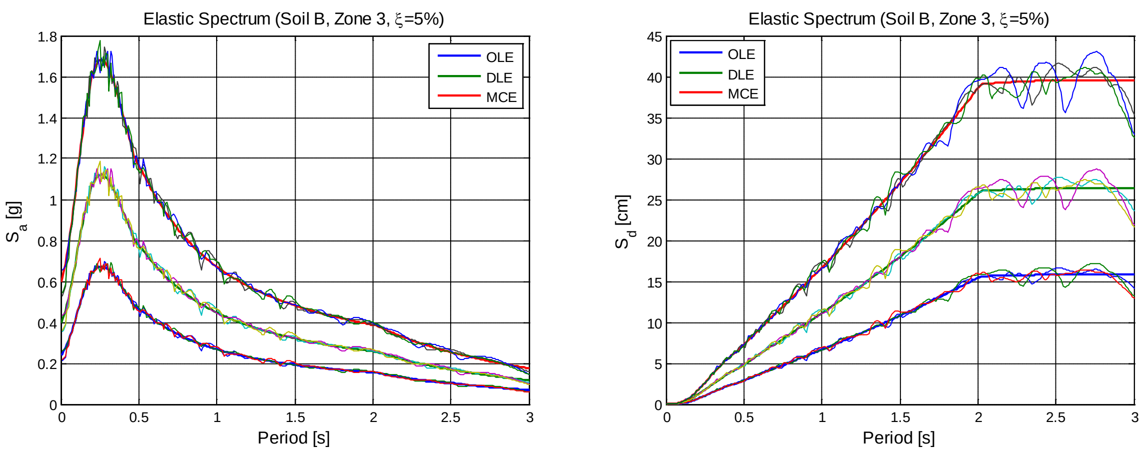

Fig. 3 – Seismic demand and adjustment of Chilean records.

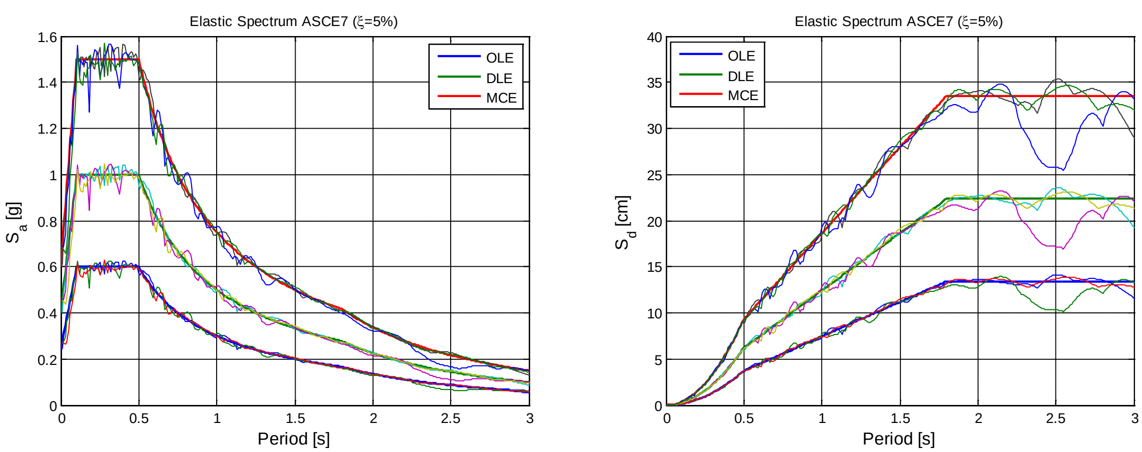

Fig. 4 – Seismic demand levels and adjustment of American records.

6. Idealized inelastic behavior

Due to inverted V-bracing structural configuration, it is expected that inelastic behavior will be concentrated in the bracing and in the center span of the girder. In the definition of their idealized constitutive relationship an amplification factor R_y=1.4 over the nominal yield stress of ASTM A36 steel is assumed. The stiffness beyond yielding is assumed as 3% of the elastic stiffness. In the case of diagonal bracing, a hysteretic model to take into account the asymmetry of the axial behavior due compression buckling is assumed. Such hysteretic model depends mainly of the global slenderness of each element, (Ductile Design of Steel Structures, [8]). For SAP2000 model, a plastic multilinear constitutive relation of pivot type is assumed. Additionally, the models include the possibility of developing plastic hinges in the columns under flexure-compression interaction after the bracing members have buckled.

Inelastic time-history analyses were run under direct integration, without considering second order effects. Rayleigh damping ratio of 2% for periods associated to first two modes of vibration was used.

7. Inelastic response from time-history analyses

Figures 5 through 9 show different inelastic responses for each of the two scenarios and the three demand levels considered. Likewise, Table 4 shows a summary of representative parameters of average behavior for the same scenarios and demand levels.

Table 4-Summary of average parameter values from set of records. Important values are highlighted.

Parameter		**Scenario I (NCh2369)**			**Scenario II (ASCE7)**		
		O.L.E.	**D.L.E.**	**M.C.E.**	**O.L.E.**	**D.L.E.**	**M.C.E.**
Roof displacement (cm)		2.38	3.12	6.96	2.25	3.75	8.40
Base shear (tonf)		144.5	143.8	144.6	84.1	88.1	105.5
Interstory drift (cm)	**Story 1**	*1.61*	2.78	6.33	*0.98*	3.03	5.28
	Story 2	0.83	0.83	0.90	1.26	1.12	2.89
	Story 3	0.34	0.34	0.36	0.31	0.31	0.35
Interstory drift (%)	**Story 1**	*0.40*	0.69	1.58	*0.24*	0.76	1.32
	Story 2	0.21	0.21	0.22	0.31	0.28	0.72
	Story 3	0.08	0.09	0.09	0.08	0.08	0.09
Bracing compression axial deformation (cm)	**Story 1**	1.75	3.15	*7.32*	0.82	2.97	*4.76*
	Story 2	0.55	0.55	0.74	1.16	1.06	2.76
	Story 3	0.20	0.20	0.21	0.20	0.21	0.25
Bracing compression axial deformation (%)	**Story 1**	0.35	0.63	*1.46*	0.16	0.59	*0.95*
	Story 2	0.11	0.11	0.15	0.23	0.21	0.55
	Story 3	0.04	0.04	0.04	0.04	0.04	0.05
Bracing tension axial deformation (cm)	**Story 1**	0.51	0.51	*0.53*	0.40	0.65	*1.59*
	Story 2	0.37	0.37	0.38	0.36	0.33	0.74
	Story 3	0.13	0.13	0.14	0.14	0.14	0.18
Bracing tension axial deformation (%)	**Story 1**	0.10	0.10	*0.11*	0.08	0.13	*0.32*
	Story 2	0.07	0.07	0.08	0.07	0.07	0.15
	Story 3	0.03	0.03	0.03	0.03	0.03	0.04
Central girder M_{max}/M_{pe}	**Story 1**	0.17	0.38	*0.94*	0.08	0.56	*0.86*
	Story 2	0.05	0.05	*0.07*	0.17	0.16	*0.47*
	Story 3	0.02	0.02	0.02	0.01	0.01	0.01
Exterior column* M_{max}/M_{pe}	**Story 1**	0.05	0.08	0.17	0.03	0.07	0.09
	Story 2	0.02	0.02	0.04	0.04	0.03	0.06
Interior column * M_{max}/M_{pe}	**Story 1**	0.10	0.16	*0.37*	0.05	0.12	*0.16*
	Story 2	0.03	0.04	0.08	0.06	0.05	0.10
Central column * M_{max}/M_{pe}	**Story 1**	0.11	0.17	*0.38*	0.05	0.13	*0.17*
	Story 2	0.03	0.04	0.08	0.06	0.05	0.09
Story displacement (cm)	**Story 1**	1.61	2.78	6.33	0.98	3.03	5.28
	Story 2	2.04	2.99	6.77	2.10	3.63	8.08
	Story 3	2.38	3.12	6.96	2.25	3.75	8.40

(*) Considers M_{pe} (expected plastic moment) corrected for the expected maximum axial force. It does not necessarily coincides with M_{max}. This indicator is only used in stories 1 and 2, since no flexure is developed in column of story 3.

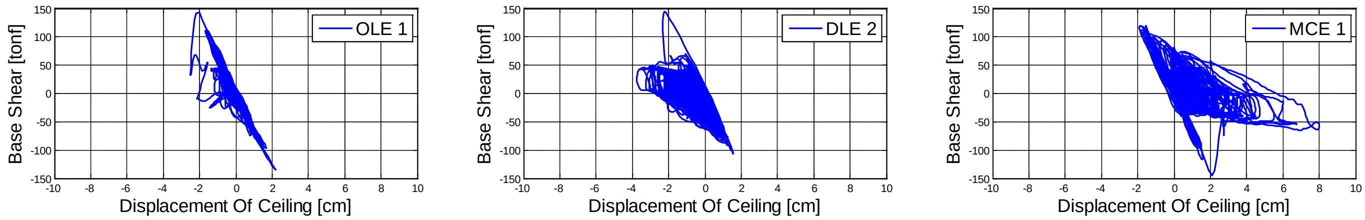

Fig. 5 – Base shear v/s Roof displacement – Scenario I (NCh2369.Of2003). Response history is shown for most demanding record in each level.

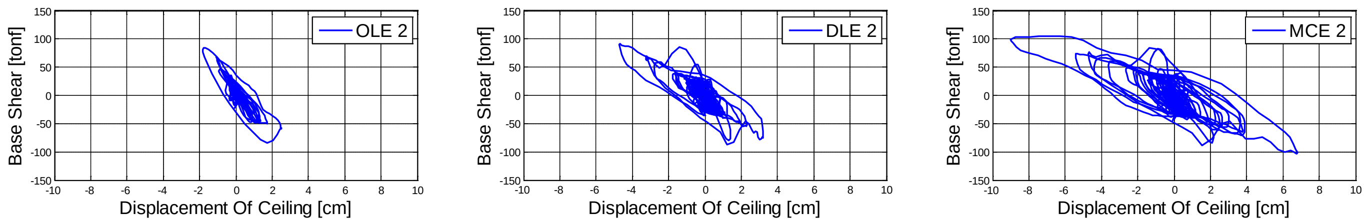

Fig. 6 – Base shear v/s Roof displacement – Scenario II (ASCE7-10). Response history is shown for most demanding record in each level.

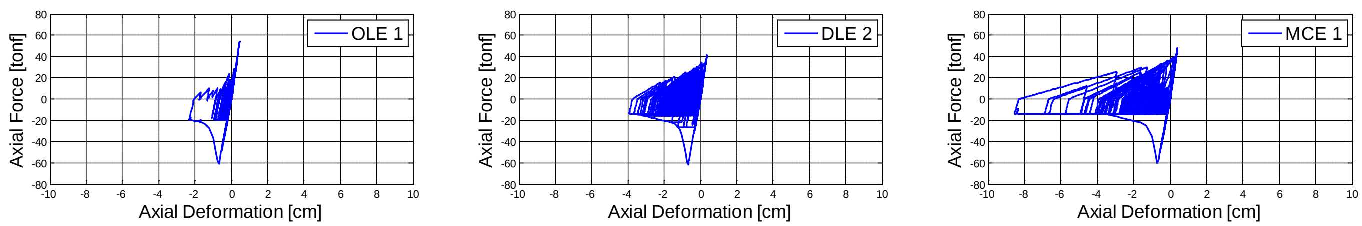

Fig. 7 – Axial force v/s Bracing deformation first story– Scenario I (NCh2369.Of2003). Response history is shown for most demanding record in each level.

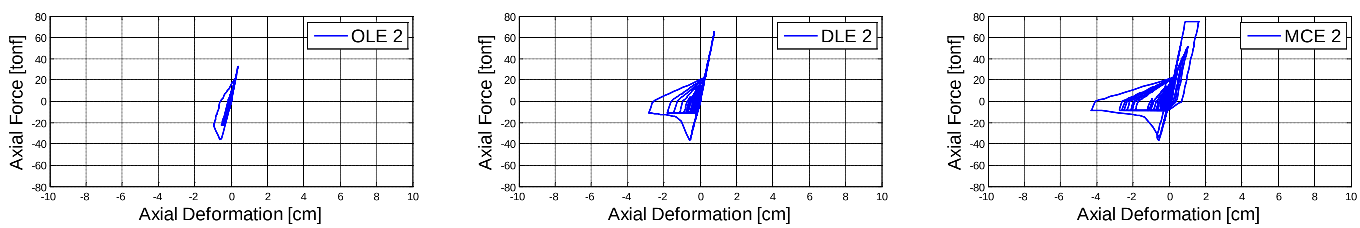

Fig. 8 – Axial force v/s Bracing deformation first story– Scenario II (ASCE7-10). Response history is shown for most demanding record in each level.

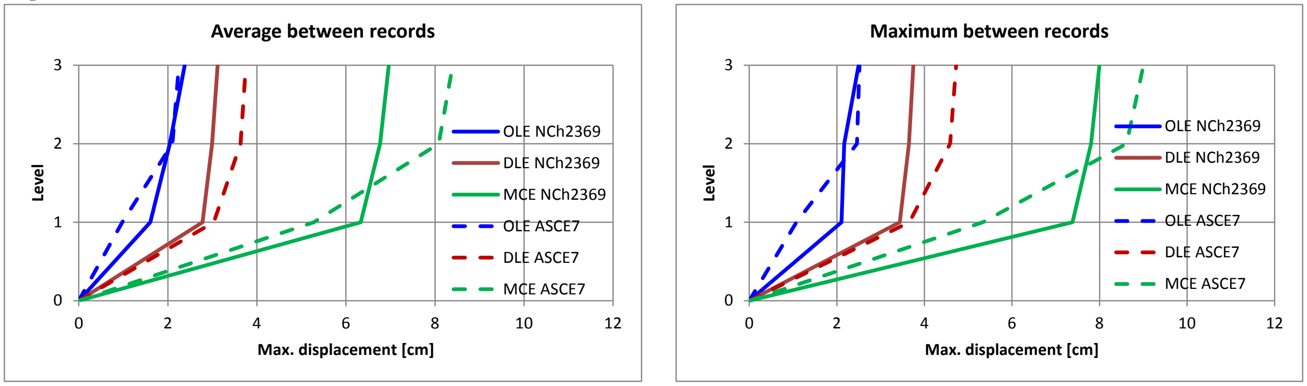

Fig. 9 – Story displacements– Scenarios I and II. Average and maximum values for the records are shown. Maximum values do not necessary occur for the same record.

8. Conclusions

Following conclusions may be derived for the specific structure and the design scenarios chosen for this study.

1. The frame structural weight, not including detailing of connections that was not evaluated in this study, was 22% higher in Scenario II (see Table 2) than in Scenario I. However, the base shear for the ASD method was 34% smaller in Scenario II (see Tables 1 and 2) than in Scenario I. In principle, this result might seem inconsistent or be interpreted as a consequence of inadequate decisions in the choice of structural members for each case. Nonetheless, as shown in Table 3, members were selected realistically. For this reason, members were not always chosen by seismic strength requirements, but for ductility, (local compactness) serviceability, (limits of deformation, minimum plate thickness), or even standard operational strength requirements, (not against severe events). Complementarily, selection of members was done from a database (catalog) of sections whose commercial availability be guaranteed (discrete set). This fact leads to a solution that not necessarily meets the design requirements in an "adjusted" way. Consequently, diagonal bracing are the only structural members whose work is essentially seismic, and its choice truly represents the base shear of each design scenario.

2. In the case of the Chilean structure, for the three levels of demand, there is a concentration of drifts in the lower stories, (soft first floor), as shown by Table 4 and Figure 9, while upper floors remain essentially elastic. In the case of the American structure, the seismic response extends to the upper floors thus involving a larger number of elements as compared to the Chilean structure.

3. Displacements of the first floor (most critical zone) are smaller in the case of the American structure, both for the OLE and MCE levels. Chilean structure displacement values are slightly smaller for the DLE level.

4. As far as column response is concerned, (see Table 4), although interstory drifts induced considerable flexure in the lower part of the building, in none of the cases response was close to plastification of members. However, columns of Chilean structure had by far more seismic requirement than the American structure.

5. In all cases studied, first floor diagonals in compression, (see Table 4 and Figures 7-8) showed higher axial deformations, (due to global buckling), in the Chilean structure. For the OLE level, Chilean diagonals showed larger inelastic incursions (buckling), as compared to slight incursions of the American structure. For the MCE level, Chilean diagonals had axial deformations of about 1.46%, (vs. 0.95% in the American case), a value that is not clear the selected members can effectively tolerate with the width/thickness ratio required by Chilean Code (less stringent than AISC341-10).

6. First floor diagonals in tension, (see Table 4 and Figures 7-8), reached yielding for the American structure and the MCE level. This condition was not reached in the Chilean structure. Complementarily, the girder receiving the diagonals did not reach yielding in the American structure, while in the Chilean structure yielding was reached during one of the records but not for the average response. It is worthwhile to note that girder of the American structure should not reach yielding since this design restriction is explicitly imposed. On the other

hand, Chilean design allows this flexural yielding to occur since no direct requirements are established to avoid this situation.

7. Following final conclusion may be stated from the response parameters shown in Table 4 and figures in Section 7 of this paper, and from the performance objectives defined for industrial structures, (see "1) -2) -3)" in Section 5): generally speaking, the American structure has shown a seismic behavior that seems to be more effective than the Chilean structure in meeting the performance objectives established for the three levels of seismic demand. Even though the expected performance for a DLE level of demand is similar for both structures, the differences are more noticeable for OLE and MCE levels.

As an additional commentary, we must say that exist a sister paper in this conference (N° 4629) that study the performance of steel moment frames. That document gives some different conclusions about the problem, because the behavior of industrial steel moment frames has not result be the same that steel braced frames. The main reason of that is the different relative elastic stiffness between both structural systems that produces different inelastic incursions and dynamic responses.

9. References

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