16th World Conference on Earthquake Engineering, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 2732

Registration Code: S- B1464478482

PERFORMANCE-BASED SEISMIC DESIGN OF A SCREENING BUILDING FOR THE MINING INDUSTRY

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Abstract

This paper presents important aspects of the design of a typical mining facility building using a performance-based seismic procedure. The structural system consists of heavy steel braced frame system that supports the main screening mechanisms of the dried process of a copper mine located in a very high seismic area. The building structure has been defined to the local code requirements following conventional analyses and design methods. The new procedure instead is based on a model including the nonlinearities of the structural system and its interaction with non-structural components. The model was run with a series of ground motion records that matches the basic seismic hazard defined by the local code. Results were compared to several performance objectives recommended here for mine facility buildings. The comparison showed that a code-based approach cannot reach a multi performance objective approach leading to an overdesign building. The amplitude of input motions was then systematically increased in order to find an equivalent demand that could potentially reach the performance objectives set in this document. The closest demand to reach a uniform collapse prevention limit was equivalent to a 5000-yr earthquake instead of the 475-yr earthquake adopted originally for design. However, a set of uniform demands that reaches multiple objectives could not be found from the several analyses performed in this study.

Keywords: performance-based seismic design; mining industry; Chilean code NCh2369; non-linear dynamic analysis, heavy civil.

1. Introduction

The seismic design of buildings using a performance-based approach is becoming a very popular one in structural engineering offices providing clients more competitive engineering services, a better understanding of the impact of seismic loads to buildings, a closer prediction of post-earthquake outcomes of structural and non-structural systems and a more informed decision making for preparing pre and post-earthquake plans. The seismic design of new tall buildings in the west coast of North America is perhaps the most popular application of a performance-based seismic design, herein refer to PBSD. Cities such as San Francisco, Los Angeles, San Diego and Seattle have regulated for the mandatory use of PBSD on different building typologies. Most of these new regulations are based mainly on documents prepared by practitioners, researchers and city officials [1,2].

The use of PBSD is also popular for the assessment and definition of seismic retrofit solutions for existing buildings. Countries such as Canada, US and New Zealand have been developing their own PBSD guidelines that have been quickly developed into codes or mandatory procedures for existing buildings [e.g. 3,4,5]. Engineers with expertise in both new and existing building structures have been exposed directly or indirectly to PBSD through these new trend of guidelines and many projects have been complete and tested under seismic loading. However, very few projects can be found in the mining sector that have either adopted or tried a PBSD approach [6].

The mining industry is currently experiencing a change in direction where every item or activity from the concept of the project to the production process will go through strict optimization processes. One of the main advantages of adopting a performance-based approach is the definition of systems/processes/designs that can be adapted to fulfill clients or stakeholders needs. In terms of the structural design, a performance-based seismic design (PBSD) approach can help engineers to better understand the behavior of the building under different seismic loads, to somehow better predict a post-earthquake scenario and better information for decision-makers. A code-based design, on the contrary, is only focused on safety aspects under one seismic load and do not give any reference or understanding of a post-earthquake scenario of the structure.

Based on our experience analyzing and designing several buildings using a PBSD approach, multiple structural performances under different earthquake loads are difficult to meet when a code-based approach is adopted and usually leading to costlier structural solutions. A PBSD approach, of course, will require more engineering work during the design stage of the project and the use of advanced tools and technologies. Nevertheless, many practitioners and researchers have created and validated these tools and procedures and many of them are easily available [e.g. 7,8]. This study is a first attempt to show the feasibility of adopting a PBSD process to a typical mine building and to set guidelines for future projects and work in this direction. We have first defined the basis of a PBSD for mining buildings and then adopted for assessing the performance of an existing screen building located in a high seismic area in northern Chile. The seismic demand was defined in terms of the Chilean seismic design code for industrial buildings, NCh2369 Of. 2003 [9]. The material and geometric properties were obtained from the structural drawings and specifications prepared by the engineer of record for this project. To capture the code intended performance, different earthquake demand levels were considered in this study.

2. A performance-based approach for the mining industry

Some preliminary attempts for defining guidelines for the performance-based seismic design of mining structures have been mainly focused on slight modifications to code-based design forces by simply changing the values of force reduction factors. The main intent of these changes is to “capture” different structural performances under different earthquake load levels without deviating from conventional practice. The use of different force reduction factors could be a feasible solution to get engineers, who are not familiar with PBSD, engaged. However, a procedure based on slight modifications of a code-based approach could lead to a wrong understanding of the basis and main intent of a PBSD.

Force reduction factors are applied globally to the design forces of every component of the structure. A local failure or controlled damage in a single zone could nonetheless easily define the intended performance of the building without any correlation to a global reduction factor. Furthermore, building components that are not necessarily structural can also affect the overall performance or service of the building. Many of these non-structural components are not necessarily forced controlled and, therefore, the use of a single force reduction factor does not play an important role on the definition of their performance. Finally, and most importantly, the main intent of a performance-based approach is exactly to walk away from prescriptive-code requirements, such as the use of force reduction factors, and identify the "real" behavior of structures under different earthquake load levels using sophisticated models and most advanced structural analyses.

We understand, however, that there is a need for practical guidelines for designing mining structures using a PBSD approach. These guidelines should be feasibly implemented by engineers and their impact be easily communicated to clients, stakeholders, other professionals and authorities. Fig. 1 shows a summarized flow of ideas and of our understanding of a PBSD approach defined for this study that could be useful for any other mining industry project. Our basic recommendations, as a first step towards a true PBSD, is to simplify structural modelling aspects, invest time on understanding and adopting clever energy dissipation solutions, open the access to local seismic hazard information and define reasonable performance criteria based on both safety and client-oriented needs.

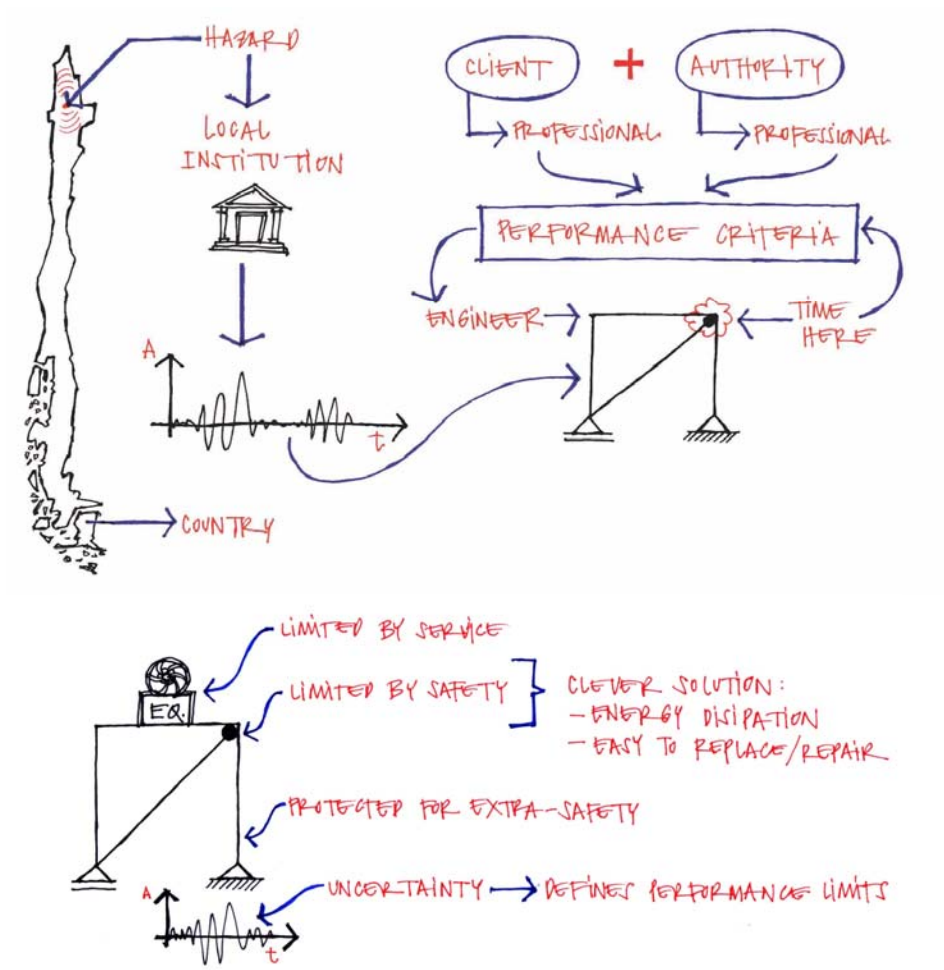

Fig. 1 – General scheme of our proposed performance-based seismic design approach for the mining industry (hand-made figures by Javiera Rolando)

3. Implementation of a PBSD

To implement the PBSD approach presented in Fig. 1, we have selected an existing mining facility building and defined both performance objectives and criteria based strictly on the operation of a particular mining process.

3.1 Crushing Operation

The selected building corresponds to the supporting structure of two screens within a copper and molybdenum production process. The plant has three identical screens buildings. The screen building is part of the wet area of the plant and serves as the intermediate process between the ore bin structure and grinding structure (ball mills). The material is carried from the ore bin through six feeders, and then goes through 6 screens. The material that passes through the screens is then stored in a 600-cubic-meter bin at the first level of building and then transported to hydro-cyclones and ball mills. Rejected material is transported back to the tertiary crushers (also known as the HPGR area).

3.2 Screening Building

There are 3 identical screening buildings supporting 2 screens each (Fig. 2). Each screening building is comprised of four levels, is 21m high and about 12m by 12m in plan. The top level (Level 4) supports the two chutes and half of loads coming from the feeders (sliding connection on screens building, vertical forces only). Level 3 supports the screens engine table and operational loads coming from maintenance. Level 2 support two screening systems (56 ton each) and the first level supports the 600m^3 pulp material. The four levels, from top to bottom, have inter-story heights of 5m, 3.2m, 8m and 4.8m. The structural system consists of steel concentrically braced frames connected to gravity steel columns through steel truss diaphragms (braces and beams). Columns are connected to reinforced concrete footings with anchor rods, steel plates and shear keys (Fig. 3).

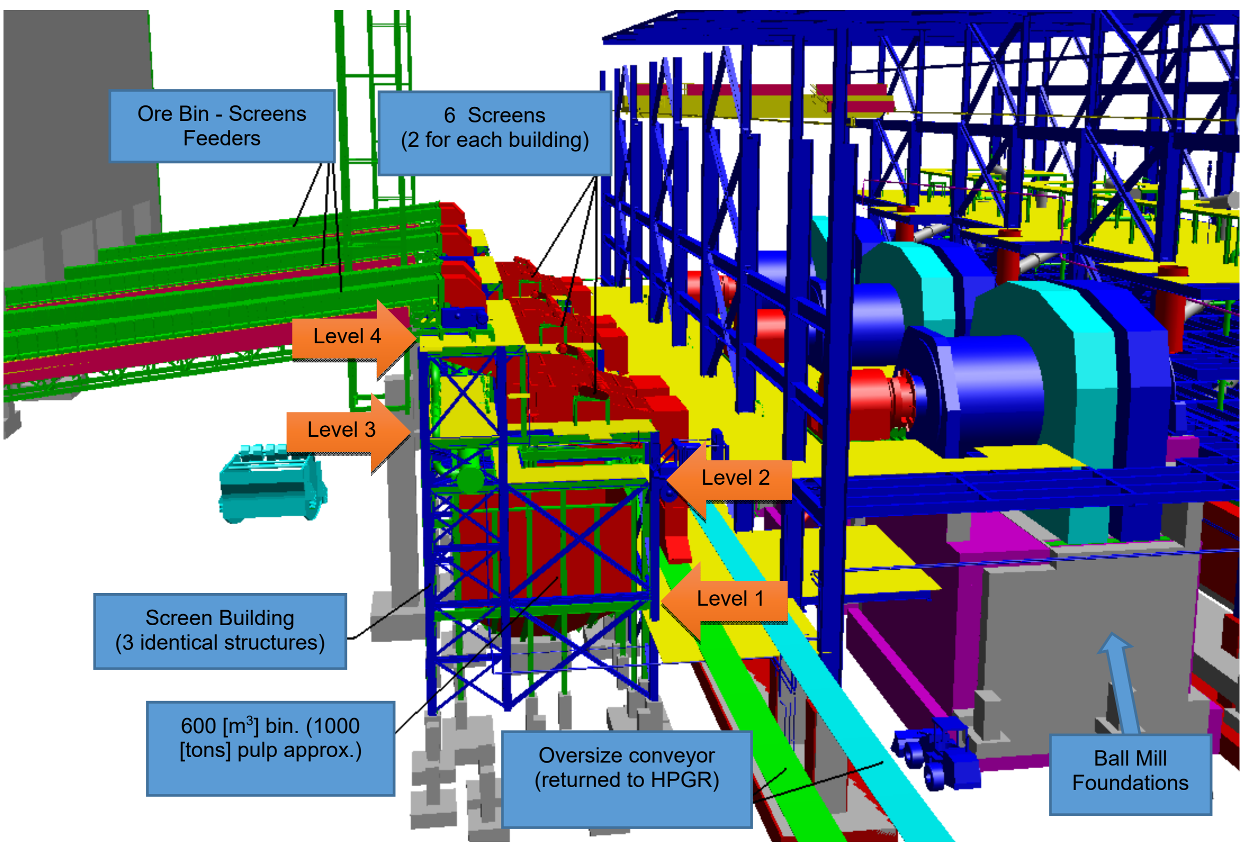

Fig. 2 – Schematic rendering of a screen building in a copper concentrator plant. Boxes are showing some main structural and non-structural components. (rendering by Raul Lobos)

Fig. 3 – Selected pictures of a screening building taken during construction and operation: (a) front elevation, (b) screen in operation, (c) back elevation during construction, (d) screen before installation (e) typical brace-column-foundation connection (f) foundation under construction. (Pictures by Tomas Nunez).

3.2 Seismic Demand

The structure is massive (approximately 2000 tons) and stiff with measured first translational periods of 0.42s and 0.38s. The base shear of this building was computed with a force reduction factor of 3 for the 475-yr earthquake spectrum shown in Figure 4 as per the local code [9]. The period values indicate that the adopted lateral force in design was close to the maximum code-based force (see the plateau for the 475-yr spectrum in Fig. 4). The local code [9] defines design criteria to meet two performance objectives: life safety and continued operation. However, the code only defines one seismic level equivalent to a 475-yr return period earthquake. The criteria defined to meet these two objectives can be associated to those presented in Table 2 for Controlled Damage (CD) using a Linear Dynamic (LD) analysis.

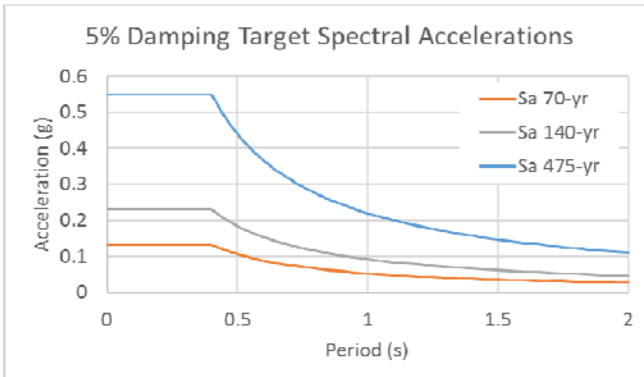

Fig. 4 – Design Spectrum adopted for seismic design based on NCh2369 [9] with R =1.0

3.3 Modelling and Structural Analysis

A full 3-D model of the building was created with computer program SAP2000 [7] including nonlinearities following popular guidelines for the nonlinear analysis of steel structures and buildings [10, 11]. Only the main structural components have been modeled, such as beams, columns and braces. The mass and gravity loads of equipment and other non-structural components have been assigned at each floor based on the information provided by vendors. The lateral load resisting system consists of tension/compression concentrically braced frames and the nonlinear behavior have been assigned to braces only following a yielding backbone curve of the tension/compression steel braces defined in Table 9-3 of the ASCE 41-13 [4]. To improve speed of the several analyses run during this study, we have adopted the fast-nonlinear method in Sap2000.

We have modeled only braces with nonlinearities following the code intent of protecting every component and expecting most damage (if any) on these elements. Our recommendation, however, is to model nonlinearities throughout the building in column/beam connections, column/foundation connections, brace-to-frame connections and some components of the horizontal diaphragms – as per recommendations of NIST 2013 [10]. We also recommend the proper modeling of heavy equipment and connections to capture a better interaction under dynamic loading between structural and non-structural components, especially for heavy equipment such as the screens and the bin. Results presented below are based on this simplified approach for modeling an expected nonlinear behavior in the structure. Nevertheless, we continue working on this model for future research projects and studies on this topic.

4. Performance Objectives

For conventional residential and commercial buildings, a set of two performance objectives are normally defined in a performance-based design process. A Service Earthquake Level performance objective is normally defined to assure the functionality of the building under a very frequent type of earthquake that is normally quantified as 1 every 475 years or as an event with 10% probability of exceeding the demand in the next 50 years (the life-span of a building). The second objective corresponds to collapse prevention under a major event that occurs every 2500 years or normally quantified as one with a 2% probability of exceeding the demand in the next 50 years. Certainly these two objectives can be modified or extrapolated to other types of structures but they will certainly differ depending on the operation or service of the building, importance factors or any other particular consideration.

The performance objectives for a mining facility should be different than those described above where the owner will probably be more inclined to set the functionality of its operation to the 1 in 475-year earthquake or even to a larger one, perhaps accepting certain operations to be stopped under a 2475-year earthquake or 2% probability of being exceeded in the next 50 years. The problem here is to set the performance criteria for any of these two or three earthquake levels and within the typical time the building or facility is under operation. From a practical point of view and based on experience dealing with mine owners and operators, it's better to set three objectives within a 25-yr life-span as follows:

Operational Objective 1 (OO1): Fully operational under earthquake motions with 30% probability of being exceeded in the next 25 years (approximately 70-year earthquake return period);

Operational Objective 2 (OO2): Continuing operation after a short downtime (max 10 days) under earthquake motions with a 15% probability of being exceeded in the next 25 years (approximately 140-year earthquake return period); and

Controlled Damage (CD): Continuing operation after a longer downtime (between 10 to 30 days) under earthquake motions with a 5% probability of being exceeded in the next 25 years (approximately 475-year earthquake return period).

The limits that define the performance of each of these three objectives are presented in the Performance Criteria section and will depend on the building component and type of analysis conducted for seismic design.

4.1 Selected Ground Motions for non-linear dynamic analysis (NLDA).

To assess the CD performance objective, we performed a series of nonlinear dynamic analyses of the existing structure using a set of 7 local earthquake ground motions recorded from the Mw 7.8 1985 Valparaiso earthquake and the Mw 8.8 2010 El Maule earthquake (see Table 1).

Table 1 – Selected records for nonlinear dynamic analyses

Record	**Earthquake**		**Station**	**Record**		
No	**Year**	**Mw**	**Name**	**PGA (g)**	**PGV (cm/sec)**	**PGD (cm)**
1NS	1985	7.8	VINA2247	0.32	32.6	6.4
1EW	1985	7.8	VINA 2247	0.21	12.6	0.8
2NS	2001	8.4	aricacostanera01006231	0.33	13.5	1.9
2EW	2001	8.4	aricacostanera01006231	0.27	11	1.8
3NS	2007	7.7	mejillones0711141	0.37	14.1	2.6
3EW	2007	7.7	mejillones0711141	0.27	11.6	1.4
4NS	2007	7.7	tocopillapuerto0711141	0.27	6.3	0.7
4EW	2007	7.7	tocopillapuerto0711141	0.13	3.5	0.4
5NS	2010	8.8	concepcion1002271	0.41	32.2	5.4
5EW	2010	8.8	concepcion1002271	0.28	26.8	3.9
6NS	2010	8.8	constitucion1002271	0.52	21.6	2.4
6EW	2010	8.8	constitucion1002271	0.62	31.8	3.6
7NS	2014	8.2	PB11	0.44	13.1	1.4
7EW	2014	8.2	PB11	0.38	13	0.8

5. Performance Criteria

Performance of mining facility buildings should be first set in terms of the overall operation of the mining process and then based upon priorities within each activity. These priorities can be defined with a single importance factor that amplifies the level of demands on the structural and non-structural components. For the sake of defining an initial set of criteria, we will concentrate on a single building with an importance factor of one. The criteria for checking the performance of the structure and its content have been set in terms of the particular operation and use of a screening building in a mine.

5.1 Criteria for Building Components

Before defining the performance criteria or limits, we need to identify the type of component, its intended use and importance. There are two types of components: Structural (S) and Non-structural (NS), two types of actions or intended use: Force-controlled (F) or Displacement-controlled (D) and two types or levels of importance: Critical (C) or Non-critical (NC). The definition of these terms are similar to those defined in [2] for the performance-based seismic design of new buildings in California. Table 2 and Fig. 5 show a summary of these components and classification and their performance criteria. Factors presented in Table 2 corresponds to the Capacity over Demand ratio (C/D). For LD analyses, Capacity is based on nominal code-based values and the Demand is the maximum linear dynamic response. For NLD analyses, Capacity is based on probable material properties and the Demand is the average of the maximum values obtained from the several nonlinear dynamic analyses. Also note that for NLD analyses the criteria for Displacement-controlled components is based on limits recommended in [4] for steel braced frames (see Table 3).

Table 2 - Capacity over Demand values for different performance objectives and for different building components and importance levels. Note that the limits set for Collapse-prevention using a NLD analyses are set for elements intended to be yielding and are defined in Table 3.

Component	**C/D for Performance Objective**					
Type, Importance	**OO1**		**OO2**		**CD**	
	LD* (R=1)	**NLD* (Opt)**	**LD* (R=1)**	**NLD* (Opt)**	**LD*** (R=3)**	**NLD* (Opt**)**
Structural (S)						
*Force-controlled **(F)***						
Critical (**C**)	1.5	1.5	1.3	1.0	1.1	1.0
Non-critical (**NC**)	1.0	1.0	1.0	1.0	1.1	1.0
*Displacement-controlled **(D)***						
Critical (**C**)	1.2	<50% CDC limits	1.2	OO1> & <CDC limits	1.0	See CDC Limits
Non-critical (**NC**)	1.0	<50% CDC limits	0.8	OO1> & <CDC limits	1.0	See CDNC Limits
Non-structural						
*Force sensitive **(F)***						
Critical (**C**)	1.5	1.5	1.2		1.4	0.9
Non-critical (**NC**)	1.2	1.2	1.2		1.2	0.9
*Displacement sensitive **(D)***						
Critical (**C**)	1.3	1.3	1.2		1.5	0.9
Non-critical (**NC**)	1.1	1.1	1.2		1.0	0.9

*LD: Linear Dynamic Analysis, NLD: Non-linear Dynamic Analysis; **For this study we are recommending the use of NLD for the CD Performance, but in general a NLD should be mandatory for assessing any performance beyond the yielding point of any building component; *** These C/D values are the ones normally adopted for the design of new industrial buildings and are computed with code-based nominal capacities.

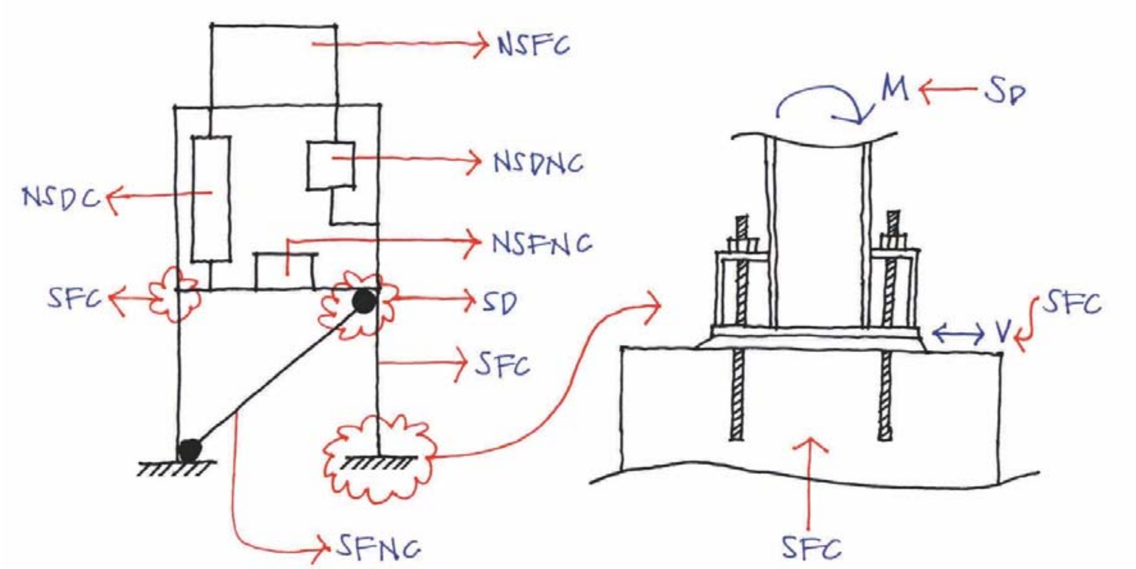

Fig. 5 - Identification of Type of element and intended performance on a building (hand-made picture by Javiera Rolando)

5.1 Controlled Damage Performance Criteria

Different limits are recommended for elements that will incur nonlinear deformations mainly based on the level of dissipation or damage that is accepted in the structure. The importance level, Critical and Non-Critical is set mainly by the owner which can be also part of a post-earthquake program defined at the design stage. A Critical importance will certainly drive much costly repairs than a non-critical one. For the purpose of this study and to simplify the assessment process, we will recommend for Non-Critical components the use of 1.5 the limits defined for Critical Components. The recommended limit values for each component at the CD level are listed in Table 3.

Table 3 – Recommended limits for the CD performance objective of this project

Component	Response	Recommended Limit	Reference
Structural			
Braces	Deformation - Tension	6 Δ_T	80% ASCE 41
	Deformation - Compression	0.8 Δ_C	80% ASCE 41
Brace to Frame Connections	Deformation over Length	1.2 x CD limits for braces*	
Frames			
- Beams	C/D (Axial)	Compression – 1.0 Tension – 0.6	
	Axial - Tensile	4 Δ_T**	80% ASCE 41
- Columns	C/D (Axial)	Compression – 1.0 Tension – 0.6	
-	Axial - Tensile	4 Δ_T **	80% ASCE 41
Diaphragms	C/D (Shear, Moment, Collectors)	0.8 (Chords) 1.0 (shear and collectors)	
Foundations			
- Connection	Rotation	Max. Drift of 0.5%	
	C/D (Shear)	1.0	
- Bearing	C/D	1.0	

* Only for this study and not recommended for new buildings; ** If modeled nonlinear; ΔT and ΔC are the deformations at the yielding and buckling force capacity of the brace, respectively.

The performance assessment of non-structural components such as the screening system and the bin can be done by checking the force adopted to design the connections versus the maximum average force obtained at the corresponding floor from the nonlinear dynamic analyses. If the ASCE 7-10 [12] formula for non-structural components is adopted the performance assessment can be done as the ratio of the floor acceleration versus the amplification factor (a_p) times the design acceleration (SD_S), the height modification (function of z) and the equipment modification factor (R/I). For this study, we have checked the engine of the screens located at Levels 2 and 3 with a maximum drift of 0.5% and the screens located at Level 3 with a maximum floor acceleration of 1.0g (this value was estimated from the mechanical/isolated system that connects the screen to the structure).

6. Comparing Code-based versus Performance-based

After several runs of the existing structure model using a CD earthquake level, we could not observe any performance to be near the limits set for this performance for the OOP1, OOP2 nor CD. We then increased the dynamic loads by single amplification factors of the input motions and trying to find an equivalent earthquake level for the performance limits that were closed enough to the ones defined for the CD objective in Table 3. We are showing in Table 4 for selected components the responses that were closely matching any of the performance objectives defined in Table 2.

Table 4 – Equivalent performance objective of each building component under different earthquake hazard levels

Performance-based analyses Result	**Code-Based Design C/D (R=3)**	CD -475yr Earthquake	1.5xCD – App. 1000yr Earthquake	2.5xCD – App. 2500yr Earthquake	3.0xCD – App. 5000yr Earthquake	4.5xCD – App. 10000yr Earthquake
Structural						
Braces	**1.0**	OO2	OO2	OO2	OO2	CD
Brace to Frame Connections	**1.1**	OO1	OO1	OO2	OO2	OO2
Frames						
- Beams	**1.2**	OO1	OO1	OO1	OO2	CD
- Columns	**1.2**	OO1	OO1	OO2	OO2	CD
Foundations						
- Connection	**1.2**	OO1	OO1	OO2	CD	CD
- Bearing	**1.2**	OO1	OO1	OO1	OO2	CD
Diaphragms	**1.0**	OO1	OO1	OO2	CD	>CD
Non-Structural						
Heavy Equipment	**1.5**	OO1	OO1	CD	CD	>CD
Light Equipment	**1.5**	OO2	OO2	>CD	>CD	>CD

7. Comments and future work

The main intent of this study was to define a performance-based seismic design approach for mine facility buildings in seismic areas. Performance objectives and criteria have been defined here and applied to a typical mine facility building. The building structure consisted of a steel frame building supporting the screening system of the mine. The building was originally designed to code requirements. Results from several dynamic analyses showed that a code-based seismic design of the building has clearly overdesigned both structural and non-structural components when compared to performance criteria defined in this document that are similar in intent to those defined by the design code adopted. We have also found that by increasing the earthquake demand we could not find a single event level that could reach the intended performance of the code, which is operational and safety at the same time. A performance-based seismic design will directly achieve the intended performance of the code without following prescriptive requirements of the demand and minimum design requirements.

This is a first attempt to define a feasible guideline for applying a performance-based seismic design of mine infrastructure. We have demonstrated here that a code-based design of mining industrial building cannot meet different levels of structural and non-structural performance under different earthquake levels. A second stage of this project is trying to identify the flaws of the design and to accommodate better energy dissipation mechanisms to achieve the intended performance of the building to a multi-level earthquake hazard.

8. Acknowledgements

We would like to acknowledge Mrs. Javiera Rolando for her great effort and quality of the hand-made figures of this paper, Mr. Raul Lobos for its wonderful rendering of mine buildings, Mr. Antonio Iruretagoyena and Mr. Victor Contreras for their great assistance on the selection and data processing of ground motion records and to the Centro Sismologico Nacional (www.sismologia.cl) and RENADIC – Universidad de Chile (www.renadic.cl) for their work and generosity on providing seismic hazard information in Chile and access to seismic records.

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