Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 49, No. 1, March 2016

ASSESSMENT OF DAMAGE TO THE CEILINGS AND SPRINKLER DOWNPIPES OF AN INDUSTRIAL PLANT FROM THE 2012 7.6MW SÁMARA EARTHQUAKE IN COSTA RICA

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(Submitted *June 2015*; Reviewed *December 2015*; Accepted *January 2016*)

ABSTRACT

This paper presents field observations on the performance of ceilings and sprinkler downpipes in a manufacturing facility 30 km west of San José, Costa Rica, during the Mw 7.6 Sámara earthquake on September 5th, 2012. The ground motion intensity was MM VI at the site and IX near the epicentre, 137 km away. The structure is a typical single-storey industrial steel gable frame with a combination of braces and portal frames in the short spans, and houses injection moulding, laboratories, clean rooms, a warehouse and office facilities. There was no structural damage observed and the production facilities were operational immediately after the event, while the office area and cafeteria required repairs due to fallen ceiling tiles. Focus is on performance of the ceilings and the sprinklers downpipes in the office and cafeteria area, and the damage inflicted by sprinkler heads on ceiling tiles. It was observed that the lateral restraints used in pipe and ceiling bracing did not prevent some sprinkler heads boring into the ceilings and enlarging the original circular perforation. The enlargement was several centimetres long and it was observed in clusters rather than isolated cases. One sprinkler drop broke at the upper thread causing water damage to the cafeteria ceiling. A large proportion of the perimeter ceiling tiles and tees in the open-plan office area fell down, while little damage was observed in smaller rooms. The drawings called for closely-spaced bi-directional "V" bracing of the Tee grid with galvanised wire, but these were found during the survey to be much further apart with most hangers being fairly vertical. A comparison between as-drawn, as-built and state-of-the-art code details is undertaken, and the observed damage is compared with expected damage using state-of-the-art fragility curves. Finally, conclusions about possible improvements are made.

INTRODUCTION

At 8:42am local time on September 5th, 2012, a portion of the subduction zone between the Cocos and Caribbean plates underneath the Nicoya Peninsula on the Costa Rican Northwest ruptured, generating a Magnitude 7.6 earthquake, 18 km depth, which was felt throughout Costa Rica and other parts of Central America. Although it was one of the largest earthquakes (and the second largest within instrumental history) in Costa Rica, there were no casualties and the damage to infrastructure was remarkably low [1]. Damaged structures included 38 bridges, 56 schools, 33 health care facilities, 119 road segments, 7 potable water distribution systems, 43 public buildings and 1990 dwellings [2]. One major hospital suffered from significant non-structural damage. Twenty one municipalities went out of electric power supply due to damage to 6% of the transmission lines and 12% of the substations, but the service was almost fully restored within 24 hours. Land lines and cellular communications became also disrupted due to saturation and power outages, but were fully operational within 12 hours [3]. Most of the damage to dwellings has been attributed to non-code-compliant construction, unstable slopes, or both. This particular portion of the subduction zone had been dormant since the 1950s (which became known as the Nicoya Seismic Gap) and therefore the amount of strain energy released in this event was among the largest ever recorded in the country [1].

A manufacturing facility outside the town of Grecia, about 137 km from the epicentre, suffered from non-structural damage. The facilities operate in a campus with two main large production buildings which house the manufacturing operations, and a series of smaller support outbuildings. No structural damage was observed in any of the buildings. Only findings pertaining to suspended ceilings and sprinkler systems to one of the buildings are reported here. The building consists of a single-storey gable framed steel structure with braced bays in the perpendicular direction. The original structure was very flexible but, during subsequent expansions and retrofits, it was strengthened and stiffened to current codes. The authors of this article were part of the team in charge of post-earthquake evaluation and recommendations for this and adjacent buildings.

This offshore event generated a peak ground acceleration (PGA) of 0.57g in the recording station nearest to the epicentre and, 0.17g in the station nearest to the site (6 km) [4]. This latter intensity is roughly half the corresponding design *effective* peak ground acceleration (a_{eff}) in the current code [5], which is 0.36g on soft soil sites [5]. The spatial distribution of ground motion intensity of Figure 1, according to Barquero [6] and Barquero and Rojas [7], shows that the most affected regions are the Nicoya Peninsula and a large area subjected to significant amplification around the towns of Grecia and Naranjo (probably the structure is located in this area), both with MMI VII [8].

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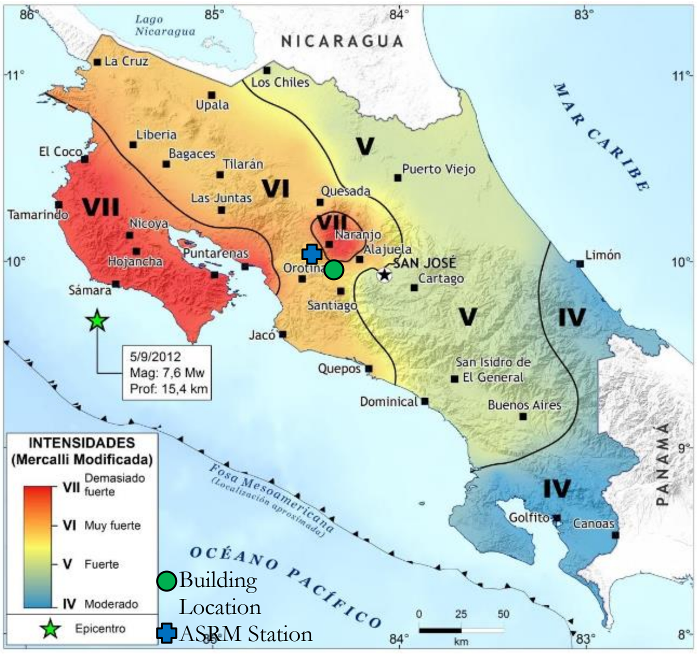


Figure 1: Mercalli intensities of the Sámara earthquake [8]. The Legend in Spanish says: "Intensities (Modified Mercalli), VII Severe, VI Very Strong, V Strong, IV Moderate, Epicentre".

COSTA RICAN CODE REQUIREMENTS FOR NON-STRUCTURAL ELEMENTS

The office and cafeteria areas had been recently remodelled at the time of the earthquake. The remodel followed the 2010 edition of the Costa Rica Seismic Code CRSC10 [5]. The provisions for non-structural elements contained in this code closely follow those of ASCE 7-10 [9]. The strength requirement is given in the following equations:

$$F_p = \frac{X_p a_{ef} I_p}{R_p}\left(1+\frac{2h_x}{h_r}\right)W_p \qquad (1)$$

$$0.75 a_{ef} I_p W_p \le F_p \le 4.0 a_{ef} I_p W_p \qquad (2)$$

A part of weight W_p, and its supports, must be able to resist a lateral force F_p given an effective ground acceleration a_{ef}. I_p is an importance factor applicable to the part. X_p is a dynamic amplification factor and R_p is a force-reduction factor which accounts for ductility of the supports, type of supports and redundancy. The dimensions h_x and h_r are the height of the part and the roof, respectively. Note that 'part' is the term used by ASCE 7-10 for any non-structural element.

For "suspended ceilings and luminaries", CRSC10 stipulates that X_p=1.0, R_p=2.5 for ceiling systems with total weight $W_p \geq$ 0.2 kN/m^2. The maximum allowed area without joints is 230 m^2, a threshold which is exceeded in both the office and cafeteria areas. No analysis is required if the ceiling length (wall to wall) is 15 m or less. Mechanical, Electrical and Piping (MEP) and lighting system should have independent anchorage to the main structure.

SURVEY OF EXITING CONDITIONS

The office and cafeteria areas are large and therefore ceiling details applicable to large ceiling areas and long wall-to-wall distances were called for in the drawings. Figure 2 shows the required ceiling details: (top) bracing with rigid bars, (centre) detachment from the wall angle along one end, (bottom) lateral bracing of the 'moving side' tee. These details, taken in conjunction, are aimed at minimising the horizontal forces on the tees which often lead to buckling and/or tearing failure of the grid followed by loss of support of the tiles.

The tiles are generally made of light acoustic material. However, in many areas around the perimeter hallways in the offices, for aesthetic effect metal pan tiles were used instead. No seismic vertical retainers or 'hold-down' clips were called for in the details.

From the site survey, it was observed that the details were in general as depicted in the construction and as-built drawings with the following exceptions:

1. The rigid stiffeners of Figure 2 (top) were used only for the heavier drywall ceilings but not generally for the lighter suspended ceilings. For the latter, wire diagonal restraints (see also Figure 2(top)) were most common, but they were often not properly executed, with many loose wires observed. This can be seen in Figure 4 (bottom). Another potential problem with the wire restraints is shown in Figure 4 as the restraints are installed at a very steep angle and thus are less effective in restraining lateral movement. Finally, local practice is to use wire of a smaller calibre than common US practice; the survey did not verify this.
2. The lateral detachment at the ceiling-to-wall angle was not followed at the interface of the suspended ceiling with a stiffer, heavier drywall ceiling as shown in Figure 4 (top), Figure 6a and Figure 6c.
3. The last tee segment at the movable support with the wall indicated in Figure 2 (bottom) was not observed.
4. In some areas, the ceiling was supported from pipes instead of from the main structure.

Pipe supports of the type depicted in Figure 5 (top and bottom) are called for in the details. These are typical of seismic applications in Costa Rica and elsewhere. A similar detail, albeit slightly more flexible, was actually used (Figure 3). The as-built drawings do not seem to have picked up the difference, but it is not clear to the authors whether this was an oversight or that simply the difference was considered immaterial. The lateral restraint of pipes was found to generally conform to the drawings and current practice.

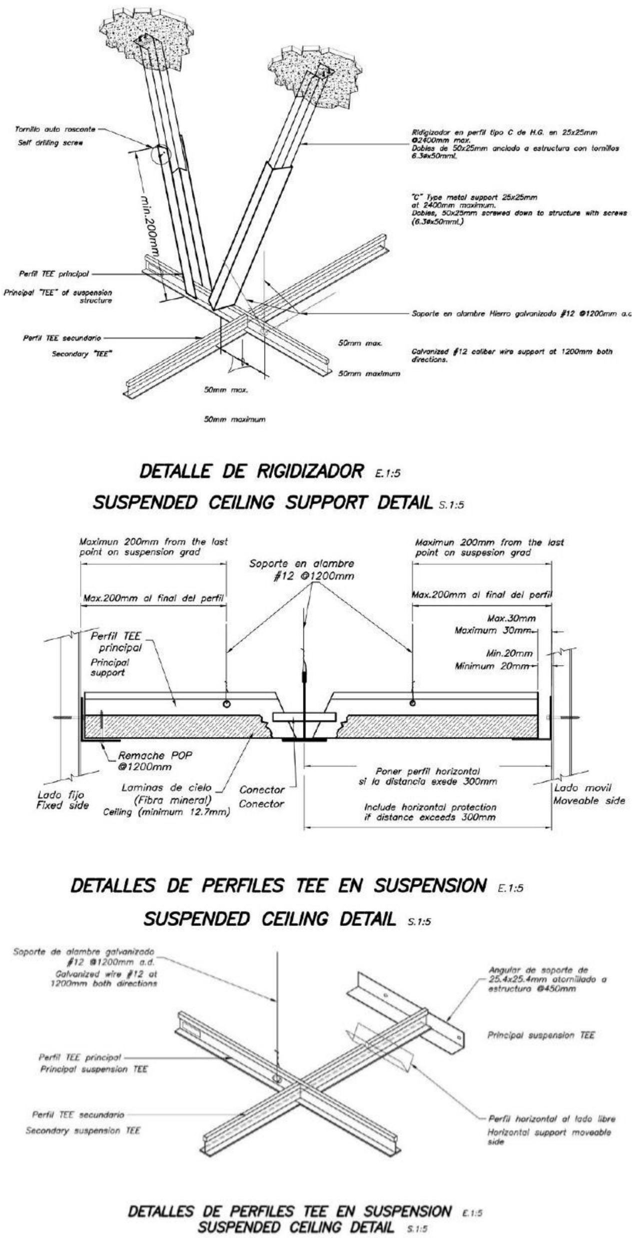


Figure 2: Ceiling grid support details found in the as-built drawings (source: FSA Ingenieria & Arquitectura S.A.).

Figure 3: Typical piping transverse brace found during survey.

Figure 4: Suspended ceiling details observed during survey.

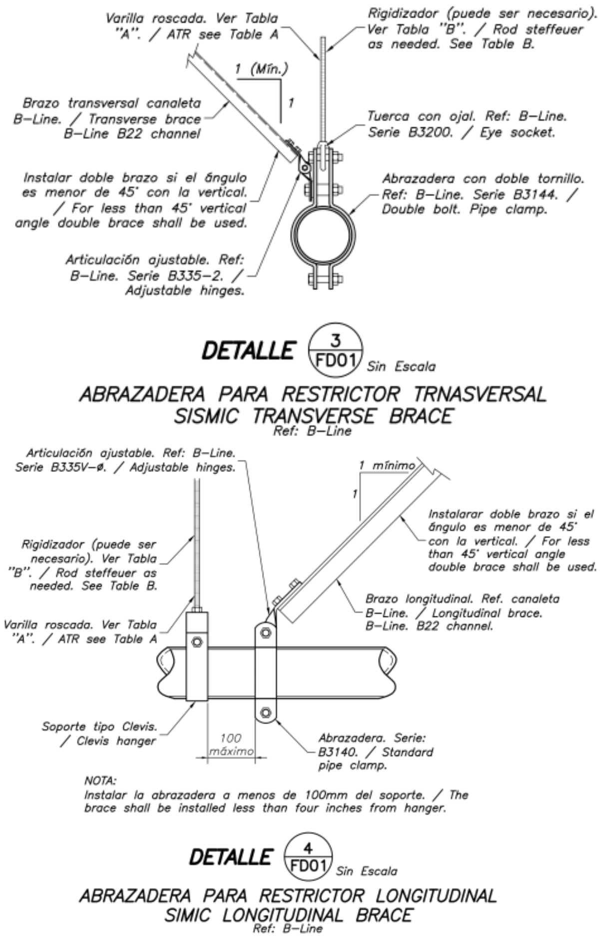


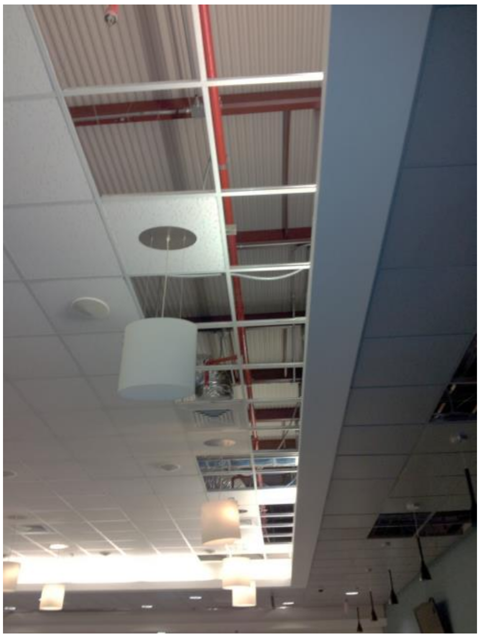
Figure 5: Pipe bracing detail in the transverse (top) and longitudinal (bottom) directions in the as-built drawings (source: FSA Ingenieria & Arquitectura S.A.).

DAMAGE IN CEILINGS AND SPRINKLER DOWNPIPES

The damage to ceilings and sprinkler system can be summarised as follows:

1. Buckling of tees around the perimeter (infrequent).
2. Falling of tiles around the perimeter (very frequent). Around 25% of total area was damaged. Fortunately, no-one was injured due to falling tiles.
3. Enlargement of the hole for sprinkler heads in ceiling tiles (frequent): during the earthquake, the sprinkler heads bore into the ceiling tiles and enlarged the originally circular perforation, rendering it oblong. The enlargement was several centimetres long, and was observed in groups rather than isolated cases, but not generalized. This leads us to conclude that the relative movement between the pipe and the ceiling is related to the local solution adopted for the pipe and ceiling bracing rather than with system behaviour.
4. Leaking downpipes (two cases): this is the last portion of pipe leading to the sprinkler head at its lower end. The relative displacement was large enough to break the connection at the top of the downpipe. A mechanical engineering consultant analysed the broken pipe and determined that a premature failure occurred as a result of an inadequate threading technique.

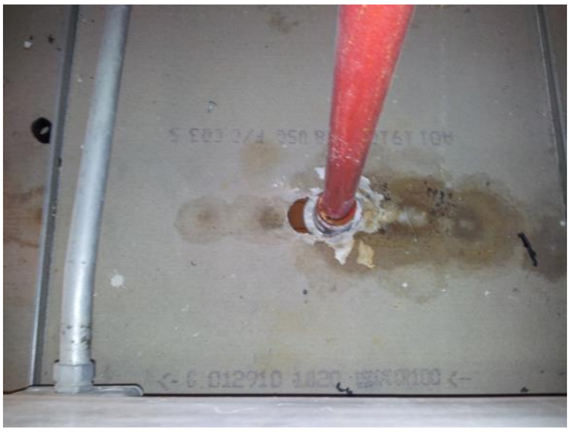
It was generally observed that the ceiling bracing was not built as required in the drawings or as shown in the as-built drawings. The enlargement of the sprinkler holes speak of a grid system strong enough to hold the tile in place while the sprinkler pipe carved its way through the tile material. The pipes and sprinklers were filled with water and thus were very heavy. This combined with a lateral bracing system for the pipes which allows some lateral movement, may have been a major contributor to this type of damage to the ceiling tiles and to the breakage of two downpipes. The owner was advised to enlarge the sprinkler holes (using the next step size cover available) and to consider eventually allowing flexible sprinkler connections in its design standards.



(a)

(b)

(c)



(d)

Figure 6: Ceiling damage. (a) Fallen tiles around the perimeter between suspended ceiling and rigid, heavier drywall soffit, and at the perimeter between metallic pan tiles and wall (cafeteria). (b) Loosened (fallen elsewhere) tiles as a result of relative movement of first tee parallel to wall and wall angle at moving support detail where the perpendicular tee is not fixed to the wall angle (offices). (c) Fallen tiles around interior drywall feature. Note that the feature is rigidly braced to the structure above. (d) Tile damaged as the downpipe supporting the sprinkler head enlarged the penetration hole (seen from above). Recent water damage can be seen as a result of a leaking connection above.

DAMAGE ANALYSIS

Given that only a few grid connection failures were observed, but that local dislocation of panels and local collapse was observed in some sections (mainly at the perimeter), it can be concluded that the ceiling grid components behaved in a satisfactory manner. The main cause of ceiling global collapse and dislocation of some panels was most likely the large areas of the ceiling system without joints. The lack of many rigid ceiling "V" shaped restraints called for in the drawings, or the use of steep angles, probably contributed to the falling of ceiling tiles. It might be possible that the tiles were smaller than the nominal size by more than 6.4 mm (i.e. undersized tiles) but these details were not examined during the visit. It is important to point out that retention clips were not called for in the drawings.

FEMA P-58-2 [10] provides a definition of suspended ceiling systems based on: (i) seismic design category (as per [9]), (ii) ceiling area and (iii) type of support. The structure under study can be categorized as seismic design category D (structures in areas expected to experience severe and destructive ground shaking) and as mentioned, the office and cafeteria areas are around 750 m^2 (8070 ft^2) and 660 m^2 (7100 ft^2) with no joints. As such, the ceiling system under consideration can be categorized under the fragility classification number C3032.003d from FEMA [10], that corresponds to a suspended ceiling system for seismic design category D/E with vertical and lateral support plus wider perimeter angle with a total area larger than 232 m^2 (2500 ft^2). It can be argued that the suspended ceiling under analysis does not present a wide perimeter angle but the chosen classification is the one that better matches the description of the ceiling system of interest.

FEMA P-58-3 [11] provides fragility curves (depicted in Figure 7 and described in Table 1) and defines each damage state for each fragility classification. Three different damage states for suspended ceiling systems are defined in FEMA P-58-3: DS1, implies that 5% of tiles are dislodged and have fallen; DS2, implies that 30% of tiles are dislodged and have fallen and a portion of the t-bar grid is damaged; and DS3, corresponds to total ceiling and grid collapse.

On the other hand, Badillo-Almaraz *et al*. [12] define four limit states for suspended ceilings systems. Limit state 1 (minor damage) is the loss of 1% of the tiles and should not impact the post-earthquake function of the building. Limit state 2 (moderate damage) corresponds to the "loss of 10% of the tiles from the suspension grid: damage that should not impact basic ingress/egress and life safety requirements. Replacement of dislodged and fallen tiles would be required" [12]. Limit state 3 (major damage) corresponds to "loss of 33% of the tiles from the suspension grid [and] large scale replacement of tiles and grid components would be required" [12]. Limit state 4 corresponds to failure of the grid and global ceiling collapse.

Badillo-Almaraz *et al*. [12] proposed fragility functions for suspended ceiling systems based on shake table tests. Five different configurations were tested. Configurations 1 and 4 correspond to "undersized tiles with compression post" and "normal-sized tiles with compression post" both with a total area of 24 m^2 (256 ft^2). Configuration 4 is of interest for this investigation because it fairly matches the description of the as-built drawings (although with a smaller area). Configuration 1 is of interested because it is similar to the description of FEMA's C3031.003a: seismic category D/E with vertical and lateral support plus wider perimeter angle with an area smaller than 23 m^2 (2.5% smaller than Configuration 1). The inclusion of ceilings with "undersized tiles" in the study is interesting because depending on the level of quality control of the manufacturing process, the size of the ceiling tile can differ from its nominal dimensions. Badillo-Almaraz *et al*. [12] considered the tiles to be "undersized" if the tile is 6.4 mm or smaller than the nominal dimensions. The statistical parameters of the fragility curves proposed in [12] for Configuration 1 are described in Table 1.

In accordance with the three limit states from FEMA [11] defined above, it is apparent that the suspended ceiling system of the industrial facility presented damage corresponding with DS2.

From the definition of Badillo-Almaraz *et al*. [12], the limit state that best describes the observed damage in the ceiling system under study is a damage state between LS2 and LS3. This is because, as discussed in the previous sections, even if only around 20%-25% of tiles fell, there was a disruption of operation in the office area, but according to the facility manager, the disruption of operation in the office area and cafeteria was very short (around a day for re-installation of the ceiling tiles). Badillo-Almaraz *et al*. [12] proposed that limit state 3 (major damage) might be defined as permissible damage to a ceiling system installed in low-occupancy, non essential facilities.

There is no data about acceleration in the exact location of the structure, but the Laboratory of Seismic Engineering of the University of Costa Rica (LIS-UCR) has a station located 13 km from the structure site. For that station, called ASRM

Table 1: Statistical parameters of fragility functions.

System	DS1*		$DS2_{[12]}$ *		DS2		DS3	
	PFA (g)	β	PFA (g)	β	PFA (g)	β	PFA (g)	β
CAT D,E,F <23 m^2 V&L FEMA [11]	1.0	0.4	-	-	1.8	0.4	2.5	0.4
CAT D,E,F 23 m^2<A<93 m^2 V&L FEMA [11]	0.7	0.4	-	-	1.15	0.4	1.8	0.4
CAT D,E,F 93 m^2<A<232 m^2 V&L FEMA [11]	0.45	0.4	-	-	0.7	0.4	1.0	0.4
CAT D,E,F >232 m^2 V&L FEMA [11]	0.35	0.4	-	-	0.55	0.4	0.8	0.4
Undersized tiles 24 m^2 V&L Badillo-Almaraz *et al.* [12]	0.81*	0.1*	1.01*	0.05*	1.5	0.2	2.0	0.2

**Definition of Damage States 1 and 2 from Badillo-Almaraz et al. [12] differs from FEMA [11]. See definitions in this page, above this table.*
PFA=Peak Floor Acceleration, β=log-normal standard deviation or dispersion, V&L=Vertical and Lateral support.

(Sede UCR-Occidente, Lat: 10.0867, Lon: -84.4784), the peak ground acceleration of the earthquake was 0.26g (north-south component), while the effective peak ground acceleration for the seismic zone and soil type where the structure is located is 0.36g [5]. In that station, the maximum vertical acceleration was 0.1g and the peak ground acceleration of the other horizontal component (east-west) was 0.23g [13].

Figure 8 shows the CSCR10 horizontal design spectra for 2% and 5% elastic damping at the ASRM station site (same seismic zone as the structure). Also shown in the figure, are the response spectra computed for 2% elastic damping for three components of the record. The spectra were computed following Newmark's method with $\gamma={}^{1}/_{2}$ and $\beta={}^{1}/_{4}$ with a time step of 0.005 s.

According to the designer of the structure [14], the building has an elastic period in the braced direction of 0.35+/- 0.05 s. This period range coincides with the plateau of the design spectrum of the site illustrated in Figure 8. The elastic period in the steel frame direction is 0.8 +/- 0.1 s [14].

Since the structure is a 1-storey industrial building, the spectral acceleration from a response spectrum similar to the one shown in Figure 8 represents, in an approximate manner, the floor (roof) acceleration and thus the peak floor acceleration can be estimated from the response spectrum from Figure 8. As such, we estimated that the maximum horizontal spectral acceleration in the building site for the period ranges of the structure (0.3-0.4 s and 0.7-0.9 s) was between 0.6g and 1.0g.

It is important to point out the fact that, due to some site amplification as inferred from the Modified Mercalli spatial distribution map in Figure 1, the ground accelerations at the structure site were probably higher than those at the ARSM station. This hypothesis is also supported by LIS [13] based on maximum accelerations recorded from different stations and H/V spectral analysis, and by Rollins *et al*. [15] based on specific shear wave velocity profiles measured by a GEER team (Geotechnical Extreme Events Reconnaissance Association) during a geophysical survey with MASW (Multi-channel Analysis of Surfaces Waves) equipment performed near the zone, after the earthquake. The soil type for the station is S3 as per **CSCR10** definition.

Following the above discussion, and assuming that the roof acceleration was a value between 0.6g and 1.0g, it is apparent that the suspended ceiling system responded as expected, for a ceiling system with the geometry, area and characteristics found *in-situ*. Moreover, as shown in Figure 7 (bottom right), if the roof experienced acceleration values near 1.0g, the ceiling system would have had a 70% probability of exceeding damage state 3. This corresponds to total collapse. Clearly this was not the case, suggesting that the actual acceleration in the roof was less than 0.8g (median value for DS3 for a similar ceiling system according to FEMA [11]).

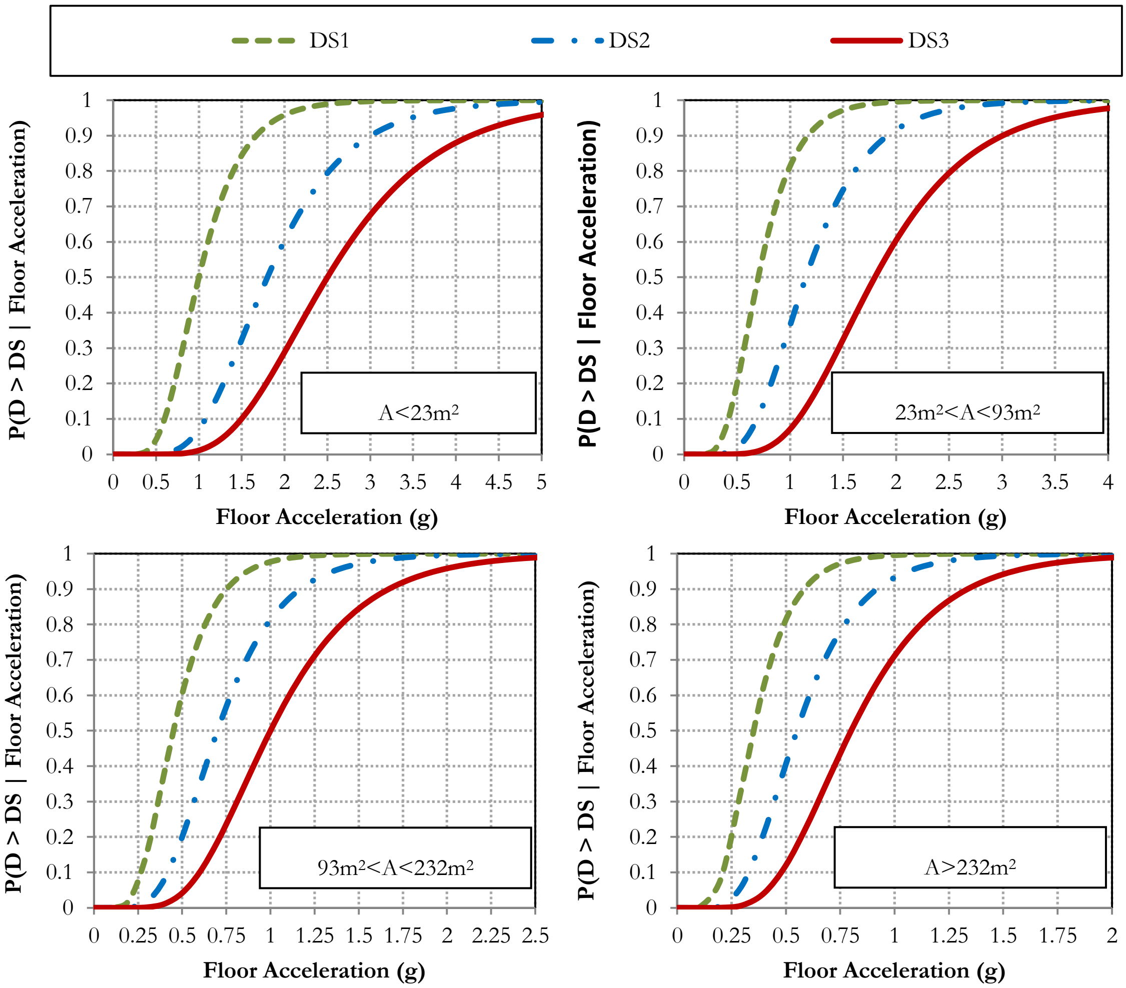


Figure 7: Fragility curves from FEMA [11] for: ceiling systems under seismic category D/E with lateral and vertical supports for: A <23 m^2 (top left), 23 m^2<A<93 m^2 (top right), 93 m^2<A<232 m^2 (bottom left), and A >232 m^2 (bottom right). A=Area.

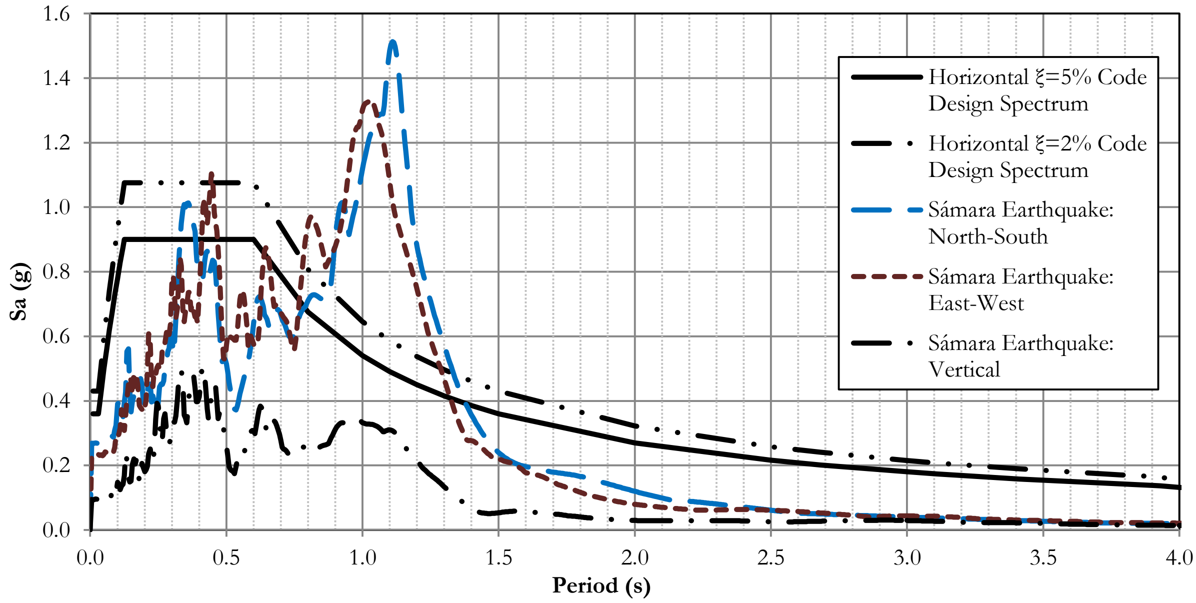


Figure 8: Comparison of 2% damped elastic spectra (both vertical and horizontal components) in a seismic station located 13km off the structure with the CSCR10 2% and 5% damped design spectra.

Moreover, the fragility curves from FEMA [11], shown in Figure 7 and Table 1, suggest that if the CSCR10 recommendation of maximum area without joints of 232 m^2 had been followed, the damage inflicted to the ceiling system would have been smaller provided that the roof acceleration was close to 0.7g.

Another important measure to improve ceiling system behaviour that has been studied in the past is the use of retainer clips. Figure 9 shows the fragility curves proposed by Badillo-Almaraz *et al*. [12] for suspended ceiling systems of 24 m^2 with compression posts with and without retainer clips.

From Figure 9, it is clear that by adding retainer clips, the floor acceleration at which the minor damage state has a 50% probability of being exceeded increases by more than 1.0g and, by all means, is higher than 1.4g even for undersized tiles. Notably, the fragility curve from Figure 9 was developed from shake table experiments for a ceiling system of 24 m^2, but the fact that the retainer clip highly improves the behaviour for minor damage state is evident. The presence of retention clips can change the response of a 24 m^2 ceiling system from "permissible damage" to "minor damage."

LESSONS LEARNED

It is apparent that the recommendation in the CSCR10 of maximum ceiling area without joints of 235 m^2 can help to improve the overall behaviour, but according to FEMA [11] fragility curves, such a ceiling system is still is very vulnerable to a floor acceleration less than 0.7g (probability of exceedance of 90% and 60% for damage state 1 and 2 respectively).

As explained, no retention clips were called for in the drawings and we did not observe any during the visit. After investigating the behaviour of the ceiling system and the study of results from shake table tests [12], it can be concluded that the presence of the retention clips could have helped to drastically reduce the loss of tiles during the Samara earthquake. It is recommended that similar details are specified in future versions of the CSCR.

This building case illustrates that, although the structure did not suffer damage, operation of the facility was interrupted due to failure of non-structural elements, underlying the importance of the engineered designs and solutions for non-structural elements.

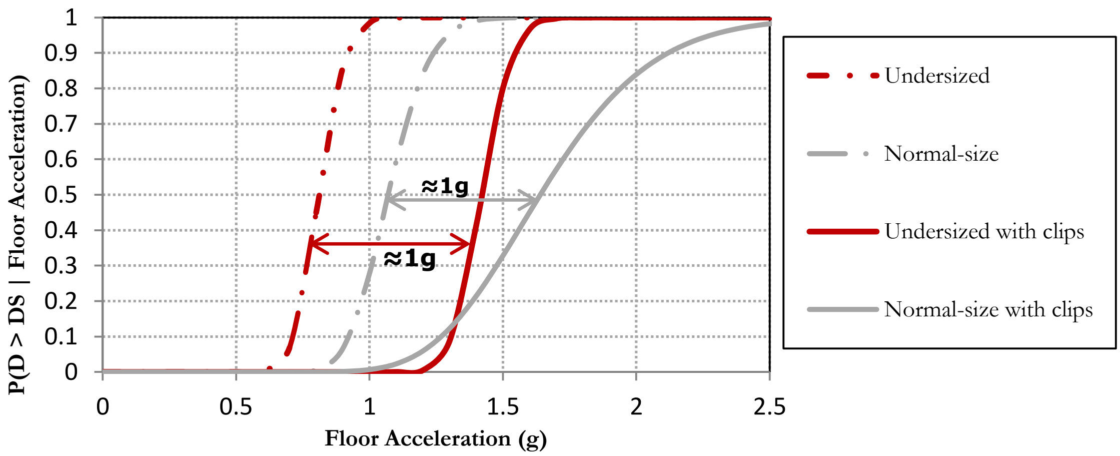


Figure 9: Fragility curves from Badillo-Almaraz et al. [12] for" minor damage" for ceiling systems of 24 m^2 with compression posts with normal-size tiles, undersized tiles, normal-size tiles with clips and undersized tiles with clips.

RESEARCH NEEDS

Although the Costa Rican code prescribes forces for the seismic design of ceiling systems (as described in Section 2) and a maximum ceiling area that can be constructed without joints, there is no construction standard that prescribes the seismic detailing or minimum installation requirements. Specific construction standard for seismic detailing and installation in the Costa Rican Seismic Code will help improve the response of ceiling systems in the country and will help engineers and architects improve the design and inspection procedures.

Apparently, retainer systems for tiles (such as the retainer clip) are not in common use in Costa Rica. One important disadvantages of relying on retainer clips for seismic performance is that clips are removed for maintenance of MEP systems. These clips are often not replaced due to the difficult but fundamental practical requirement of ready replacement of an individual tile and holding down mechanism following service access to the ceiling space [16]. Another important disadvantage of the use of retainer clips is the fact that their use might increase the inertial loads on the grid, resulting in grid damage [12]. A simple solution based on Costa Rican engineering practice can be investigated, with a focus on less expensive and easier-to-replace systems. A possible solution that can be studied is the use of double-sided tape, but this proposal should be accompanied by analytical and experimental tests.

The use of suspended ceilings systems in office and cafeteria areas in single-storey gable framed steel structures with braced bays in the perpendicular direction is very common in Costa Rican practice (and arguably, around the world). Given the single-storey characteristics, a simplified method for assessment and computation of the maximum acceleration at roof level and the ceiling system can be developed based on the expected spectral acceleration, as has been preliminarily investigated in this article.

A method to improve seismic performance of common ceiling systems used in Costa Rica needs to be developed.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the European Centre for Training and Research in Earthquake Engineering (EUCENTRE) for funding the research leading to the results presented in this paper. The help and contribution from FSA Ingeniería y Arquitectura S.A. with drawings and personal communication is also greatly appreciated.

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