Seismic Performance Assessment of a Typical Peruvian Public-School Building

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Abstract. Several Peruvian public school buildings have not performed well during earthquakes, with the level of damage varying from cracks in structural elements to the complete collapse of the building. The reasons for the unsatisfactory seismic performance of the buildings include the design codes followed at the time these schools were built, and the lack of professional engineering supervision during the design and construction phases of these schools. This study focuses on a typical school building, selected after performing a survey to public schools in the district of San Juan de Miraflores in the city of Lima, in Peru. The seismic response of the selected building to various ground motions is analysed with nonlinear static and time history analyses. The results indicate that the building does not meet the minimum seismic performance level required by the current Peruvian seismic design provisions.

1. Introduction

In all areas located at highly active seismic zones, it is important to guarantee a sufficient seismic resistant behaviour of the educational infrastructure. In the 90s, the Peruvian Government built new schools throughout the country, according to the so-called Construction Plan [1]. A recent qualitative study carried out about the seismic risk of public educational infrastructure [2], indicates that the “780 Pre” building is a representative building that can be found in many cities of Peru, many of them at high seismic zones.

The “780 Pre” is a two-story building with plan dimensions of 7.80m x 23.5m and 3.35m height per story [3]. It was built before 1997, typically not following current design and construction practice [2]. Moreover, after earthquakes events serious structural damage has been observed in this type of building, including shear failures on columns due to the short column effect introduced by the infill walls and the short wide windows at the top introduced to allow light into the classrooms [4]. The short column effect and typical distribution of the “780 Pre” building are shown in Figure 1. In this article, the seismic performance of such buildings is investigated with static and dynamic non-linear analyses.
2. Analysis Procedures

2.1. Computational Model

The building consists in confined masonry in the transverse axis and reinforced concrete frames with masonry infills in the longitudinal axis. Concrete column sizes are 25cm x 45cm with 6\(\frac{3}{4}\)” bars, while the masonry walls are 25cm thick when parallel to the transverse direction of the building and 15 cm thick when aligned with the longitudinal direction [3]. It is modeled using the software SAP2000. In the transverse direction, the masonry walls were modelled as equivalent struts as proposed by Bazan & Meli [5]. Material strength for concrete and masonry in compression, and steel in tension are \(f'_c\) of 17MPa, \(f'_m\) of 4.4MPa, and \(f_y\) 420MPa, according to original drawings [3].

In the longitudinal axis, infill masonry walls were initially modelled as shell elements with elastic behaviour because there was no significant damage seen on these walls after previous seismic events. These infill walls restrain the lateral displacements of columns up to a certain height, causing excessive shear stresses in the remaining free height. For comparison purposes, the infill masonry walls are also modelled as equivalent struts using the same methodology as the transverse masonry walls. Figure 2 shows the type of models per case.

Plastic hinges are defined at the ends of beams and columns. For concrete columns the moment hinge represents a flexure-controlled failure of the element, meaning that the reinforcement steel, concrete strength or element dimensions are not sufficient to resist the moments that are developed in the location of those hinges. The shear hinge is used to capture the short column failure observed in these schools after different seismic events [4]. For masonry elements an axial hinge is applied in the equivalent struts and represents the failure in tension/compression of the masonry and the infill walls, in the transverse and longitudinal directions, respectively.

2.2. Capacity Curves from Non Linear Static Procedures NLSP
A monotonically increasing base shear force is applied to the structure until certain displacement in a control joint is reached. As result, capacity curves are obtained per each orthogonal direction and they are shown in Figures 3 and 4.

![Capacity curve](image1)

**(a) Capacity curve**

![Shear hinges](image2)

**(b) Shear hinges**

**Figure. 3** Pushover analysis results along the longitudinal direction

![Capacity curve](image3)

**(a) Capacity curve**

![Damage pattern](image4)

**(b) Damage pattern**

**Figure. 4** Pushover analysis results along the transverse direction

2.3. **Capacity Curves from Non Linear Time-History Analysis NLTHA**

Non Linear Time-History Analysis (NLTHA) is used to assess the behaviour of the structure when it is subjected to cyclic loads due to ground motion records of a real-life earthquake of 6ML magnitude [6].

Given that the longitudinal axis is the structural weaker direction according to the typical collapse mechanism seen in these buildings in previous seismic events, it is the only axis that is analyzed through the NLTHA. The envelopes in Figure 5 represent the capacity curves of the structure modelled with different approaches, including shell elements and diagonal struts for the infill walls.
2.4. Performance Based Seismic Design

After performing the non-linear analyses, the performance point assessment is found. This method is proposed by the ATC-40 [7], therefore the ADSR conversion equations were used.

Four different soil conditions from the Peruvian seismic design provisions [8] are assessed in this study: S0, S1, S2 and S3 with S0 soil representing a stiff rock soil and the S3 a flexible weak soil. The spectrum responses according to different soil conditions are converted to seismic demand curves. Then, these curves are plotted with the capacity curves found in the previous section for both elastic and inelastic demand curves. The performance levels are also plotted along the other curves, as shown in Figures 6 and Figure 7.

Figure 5 NLTHA results for longitudinal direction using equivalent struts (a), and shell elements (b)

Figure 6 Seismic performance assessment for the longitudinal direction with moment frames
3. Discussion

The NLSP in the longitudinal direction indicates that the structure fails by excessive shear stresses in the columns caused by the infill masonry walls. The highest base shear force that the structure is able to resist in the longitudinal direction is 73.9t with a lateral drift of 1.79‰, while the drift before collapse is 3.43‰.

In the transverse direction, the failure mechanism initiates with the cracking of the masonry walls, which end up failing in compression. The points after the peak of the capacity curve represent the influence of the reinforced concrete frame that is supposed to only serve as confinement of the masonry wall. At the end, the structure collapses due to the excessive moments in the plastic hinges defined in the frame elements of the reinforced concrete confinement frame. The capacity curve in this direction indicates that the structure can resist a base shear force of 156t associated to a 0.45‰ drift. The structure would collapse in this direction if the lateral drift reaches 2.99‰ as the resistance drops to 41t.

With the NLTHA, two capacity curves are obtained by drawing the envelopes over the hysteresis curves of both models, with shell elements and equivalent struts. These capacity curves are also compared in one graphic with the capacity curves from NLSP results, for the two modelling variations, as shown in Figure 8. The difference between these curves shows that the model is sensitive when the infill walls are modelled with an elastic or inelastic response, and when there is a monotonic NLSP or dynamic NLTHA load analysis, as expected.

Figure 7 Seismic performance assessment for the transverse direction with confined masonry walls

(a) Elastic

(b) Inelastic
The seismic performance assessment of the longitudinal direction shows that when the structure is expected to behave elastically, it is not able to intercept any of the mentioned curves. This means that it would not be able to resist the base shear force even if it were built upon a S0 soil, the best soil considered in this study. On the other hand, when the structure is subjected to inelastic seismic demands, it does intercept these curves. Hence, the structure’s performance point for the S0 soil is immediate occupancy, for the S1 and S2 soils is damage control, and for the S3 soil it is beyond the damage control limit.

In the transverse direction, it is found that the structure does not intercept any of the elastic seismic demand curves, indicating that it would not resist the base shear forces if it were built upon any soil condition. Compared to the inelastic demand curves, the structure only intercepts the S0 and S1 soil. For the S0 soil, it still has capacity to resist slightly higher seismic accelerations, but for the S1 soil the structure reaches its inelastic behavior. For both S0 and S1 soils, the seismic performance level is of damage control.

4. Summary and Conclusions
Both static and dynamic non-linear analyses are performed to the “780 Pre” typical public-school building of Peru, using different approaches when modelling the infill masonry walls. The seismic behaviour is also assessed by plotting its capacity curve in each direction and comparing against the spectrum responses estimated for the four different soil conditions, which were converted in compatible plots of base shear force and lateral drifts.

It is found that the typical school building does not guarantee its immediate occupancy use as a refuge infrastructure after a severe earthquake in most cases. The only exception is if the structure is built upon a site with S0 soil with a seismic acceleration in the longitudinal direction.

Analyzing effective methods for the structural strengthening of these buildings is highly recommended for all “780 Pre” buildings found throughout the country.

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