Damaging Effects of the Pisco-Chincha (Peru) Earthquake on an Irregular RC Building

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ABSTRACT :

This paper presents back-analysis result of a RC building in Ica, Peru which was severely damaged during the Pisco-Chincha Earthquake on 15 Aug 2007. The building was located approximately 120 km from the epicenter. It has 12 spans in E-W direction and 3 spans in N-S direction. It consists of two stories and is separated into two modules at the center which allows independent vibration of each module. During the earthquake, many columns of the building were heavily damaged by shear-flexure-axial interaction. Masonry infill walls shortened the effective length of columns, which resulted in brittle shear failures. Part of the structure was overloaded with partition walls on second floor which might have led to the crushing of columns below. The analytical model of the building is based on the field measurements. Actual size of structural elements, non-structural elements, and exposed reinforcements were measured during field investigation. Unexposed rebars and material properties are determined considering construction practice. Ground motion recorded at a station located at 0.5 km from the building site is used as an input to the structure. To investigate the effect of infill walls on shear force demand, two analyses are conducted; with and without infill walls. The analysis results confirmed that shear force demand on columns with infill walls is significantly higher than those without infill walls. In addition the seismic demand with infill wall is larger than the shear force capacity estimated from design equation. It is anticipated that infill walls in the first floor, overload in the second floor, and inadequate stirrups of the columns resulted in the failure of columns.

KEYWORDS: Pisco Earthquake, back-analysis, RC structure

1. INTRODUCTION

On August 15 2007, a strong earthquake of magnitude 8.0 ± 0.1 hit the coast region of Central Peru, causing considerable loss of life and livelihood. The rupture mechanism was complex with two major ruptures about 60 seconds apart. The earthquake caused about 600 deaths with several hundreds injuries, destroyed over 50,000 buildings and damaged over 20,000. Extensive soil liquefaction was observed in the coastal planes where houses, utility and communication networks suffered extensive damage due to large permanent ground deformation. Major landslides and slumping was also observed in the epicentral region. The majority of structural failures were observed in stone and brick masonry structures. However, several reinforced concrete structures also suffered major damage or collapse, often due to soft storey effects and lack of vertical continuity. The lack of ductile detailing was clear and repetitive even in modern construction.

To probe the causes of damage and the features of both structural capacity and demand that may have compounded the damage, a case study of a real structure from one of the worst hit areas is studied. One of the heavily damaged reinforced buildings is chosen as a reference structure to investigate the effects of infill walls on the seismic demand and capacity of structures. The reference structure is a two-story building primarily used as class room and chemistry lab. The first story columns of the reference structure were heavily damaged during the earthquake showing failure from interaction of shear, moment, and axial load. It is suspected that infill walls negatively affected the structural capacity and seismic demand by shortening the period of the structure and reducing the effective column length. Moreover, infill walls that were not considered during design stage might have significantly increased axial load on the first floor over the design limit. This study revisits the effect of infill walls on the seismic response of reinforced concrete moment frames. For this study, nonlinear time history analysis is undertaken with record obtained from a station located at 0.5 km from the analyzed structure. Dimensions of the structure are measured during field investigation. Observed damage patterns, configuration of reference building, and analysis models and results are presented in the subsequent sections.

2. CONFIGURATION OF THE REFERNCE BUILDING

The reference structure is located at Ica, Peru, 117 km from the epicenter of Pisco-Chincha Earthquake. In Ica, many residential structures constructed with adobe and masonry were heavily damaged. Most engineered structures survived the earthquake without significant damages. However some of the engineered structures, including the one investigated in this study, suffered significant damages. The building in this study was selected as it was an engineered structure and heavily damaged during the earthquake. Furthermore, recorded ground motions were available from a nearby ground motion station (ICA2) 0.5 km away from the building. Hence the observed structural damage can be understood through a nonlinear response history analysis with the actual input ground motion. Analog accelerometer was used to record the motion and was on the first floor of a two story building similar to the reference structure. As the accelerometer was not installed on bedrock, it was anticipated that the recorded ground motions might include vibration components resulting from soil-structure interaction. Hence in the analysis in this study, soil structure interaction is not explicitly accounted as it is implicitly considered by applying recorded ground motion at the first floor of the similar building.

The reference building was used as a class room and a chemistry lab. The building was a two story reinforced concrete structure and consisted of 12 bays in EW direction and 3 bays in NS direction, Figure 1. The building was constructed as two independent modules separated at the center of EW direction. Two modules had completely independent gravity load carrying system by separation of slabs and beams at their interface. Stair wall was also an independent module. Span dimension of each bay were 4.2 m x 8.0 m. At the east and west face of the building, additional columns (CW21~23 and CE21~23) were constructed on the first floor to support gravity loads of full-height infill walls on the second floor, Figure 1. The second floor of the building has similar plan with the exception of the existence of many partition walls at certain span of the floor which causes large gravity load on the first floor columns. The load was mainly carried by column CW06, CW07, CW14, and CW13 in Figure 1.

Figure 2 illustrates the west and north face elevation of the building. Exterior walls facing east and west were story-high infill walls, Figure 2 (a). Partial height infill walls were placed between most columns in north and south face of the building, Figure 2 (b). Most of them had openings for windows. Some of the infill walls had openings used for entrances. Infill walls were made of clay bricks with 175 mm thickness. The second floor slabs were extended as cantilevers. Cladding was constructed on top of the cantilevered slabs to expand usable floor area. Story heights for both floors were 4.1 m. Column dimension of the building were 350 mm x 550 mm for all columns except intermediate columns in west and east face. Eight #5 (dia. = 15.8 mm) longitudinal reinforcement bars were used for exterior columns and four #5 bars were used for interior columns. Minimal amount of stirrups were used in columns; only one #3 (dia. = 9.52 mm) stirrup was used at the end of column and thin smooth wires with 5 mm diameter were used to hold longitudinal rebars in place. Beams were 350 mm x 650 mm including 250 mm thick slab. The lack of damage to the beams and slabs resulted in theire rebar dimension not being measured since the rebars were not exposed.

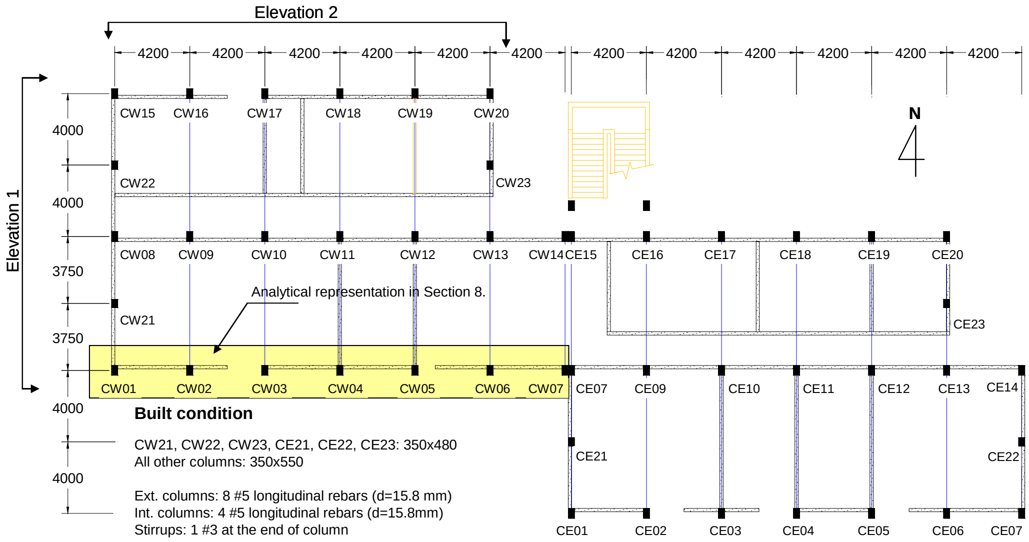
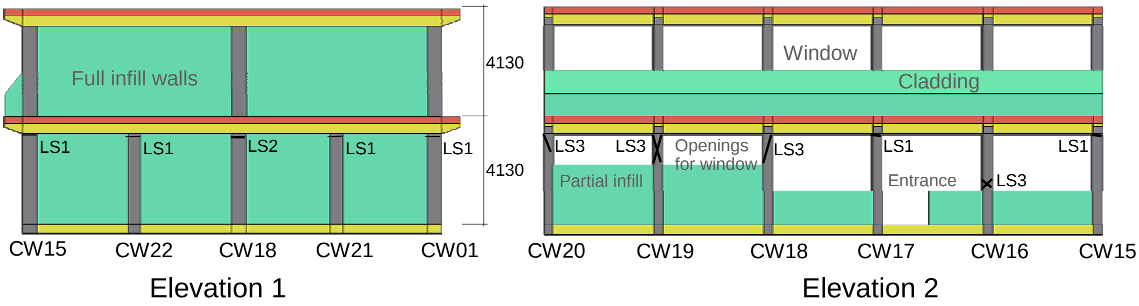


Figure 1. First floor plan of the reference buliding



LS1: Serviceability state (insignificant cracks)
LS2: Moderate damage state (loss of cover concrete, exposure of longitudinal bar)
LS3: Complete loss of capacity (buckling of longitudinal rebar, fracture of stirrups, loss of core concrete)

Figure 2. Elevation the west and north face of the building

3. OBSERVED DAMAGES AND ANTICIPATED CAUSES

The building was constructed as two modules separated at the center of east-west direction. The first story columns of the west side of the structure were heavily damaged. Many columns completely lost their capacities to resist gravity loads. Based on observation, the following factors are anticipated to be the causes of the structural damages.

3.1. Short Column Effects due to Infill Walls

Partial infill walls were constructed between columns in EW direction. Some of the infill walls experienced minor cracks and crushing at the corners. But most infill walls remained intact. The damage pattern of the columns showed that the infill walls shortened the effective length of columns. The shortened columns reduce structural periods, which in general increase seismic force demand. In addition, the shortened columns are subject to non-ductile shear failure rather than ductile flexural failure. Figure 3 (a) shows one example of column failure from the earthquake. The left side of the column has higher infill wall than the right side. The difference in the height of infill walls lead to non-symmetric deformation capacity of the column. Hence shear cracks developed in one direction. It can be easily noted from the marked damage patterns in Figure 2 that columns with less restraint from infill walls, CW15 and CW17, suffered less damages than columns with more infill wall restraint.

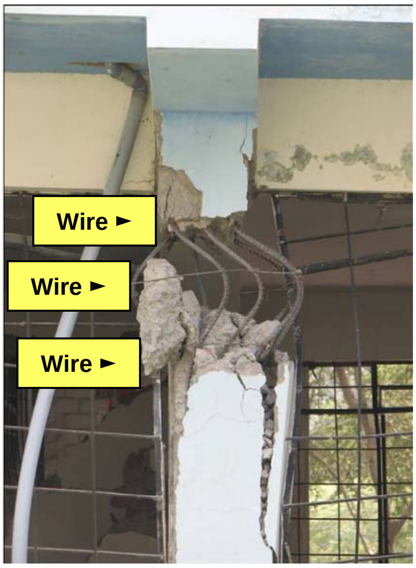
3.2 Inappropriate Stirrups

Damaged columns exposed diameters and number of longitudinal bars as well as those of stirrups in columns. Figure 3 (b) shows a close-up view of one of the damaged columns, which revealed stirrups and longitudinal reinforcements. It can be clearly seen that in most columns, smooth wires with diameters of 5 mm or less were used instead of regular deformed reinforcements. The longitudinal bars buckled as the weak stirrups couldn't provide enough confinement to core concrete. In addition, the number of stirrups was not enough to provide resistance to shear force demand. The combination of increased shear force demand from column shortening with reduced shear capacity and ductility from inappropriate stirrups are the most likely cause of the column failures.

3.3 Overloads on the Second Floor

The columns in the west module were much more severely damaged than those in the right module, even though the layout of columns and beams were almost identical. Columns CW04, CW05, CW06, and CW07 in particular were severely damaged and shortened due to loss of core concrete, Figure 3 (c). A closer look at the plans of the second floor and load carrying system revealed that the columns in the west module might have been overloaded in comparison with those in the right module. The west and east face of the building had intermediate columns between major columns to support gravity load of full-height infill walls on the second floor, Figure 1. Hence, it is expected that if there is significant dead load equivalent to the infill all of full story height, the dead load should be distributed to additional columns. From field observations, however, it was found that the span surrounded by columns, CW06, CW07, CW14, and CW13, was heavily overloaded with partition walls. In addition, there was fairly heavy cladding on the canopy of the overloaded span, Figure 3 (c). These gravity loads seem to be larger than the full-height infill walls on the east and west face. Hence the columns of this span may have carried larger gravity loads than design loads. Crushing and shortening of the columns of this span might have redistributed gravity loads over other columns resulting in subsequent failure of columns.

(a) Shear failure, CW18



(b) Inappropriate stirrups, CW06

(c) Overloads from infill walls and canopy

Figure 3. Failure modes of columns of the reference structure

4. BACK ANALYSIS OF THE REFERENCE STRUCTURE

Nonlinear analyses of the damaged structure are presented in this section. The columns and beams are modeled with fiber based section elements in Zeus-NL, Elnashai et al. 2002. Infill walls are modeled as diagonal struts with hysteretic properties determined based on methods in literatures introduced in the subsequent sections. The followings are assumed in the analytical model of the structure.

- Infill walls do not carry vertical loads. This assumption is adopted as infill walls were constructed after the construction of the frames. In addition, most of the infill walls had openings for windows, which did not allow the transfer of gravity loads from beams to infill walls.
- Infill walls can be represented by diagonal struts with horizontal resistance.
- The two modules of the building vibrate independently. Hence, only the west part of the building is modeled.

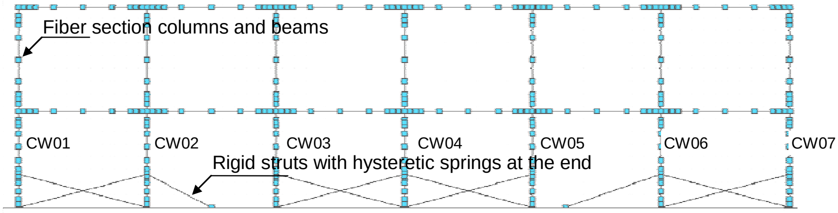
Nonlinear response history analyses are conducted to evaluate seismic demand on the 1st story columns of the building. Effects of infill walls are studied by comparison of analytical results of a frame without infill walls.

4.1 Analytical Model

The shaded frame on the west module of the building in Figure 1 is modeled for numerical analyses. The analytical model has six bays and two stories, Figure 4. Infill walls are modeled as diagonal struts whose hysteretic properties are discussed in the later part of this section. The material properties of the concrete and steel reinforcements were not obtained from the field investigation. The strengths of concrete and steel reinforcements are assumed based on typical material properties. Concrete ultimate strength is assumed to be 27 MPa and steel yield strength is assumed to be 410 MPa. Dimensions of column and beam sections were obtained from the field. Field measurements of the diameters and number of longitudinal and transverse reinforcements of columns were obtained and are shown in Figure 5. Sections of beams are modeled as T-beams to account for the effect of slabs. The effective flange width is assumed to be the smallest of 1/4 of beam length (2000 mm), 16 times the slab thickness (4000 mm), and a clear distance from web to adjacent web (4200 mm) as proposed in ACI 2002. The reinforcements of beams were not obtained from the field as none of them were exposed. Thus the reinforcements are determined from designing of the T-beam under gravity load. It can be easily noted that the section size of the beam is substantially larger than that of columns. Figure 5 illustrates layout of sections and reinforcements of beams and columns.

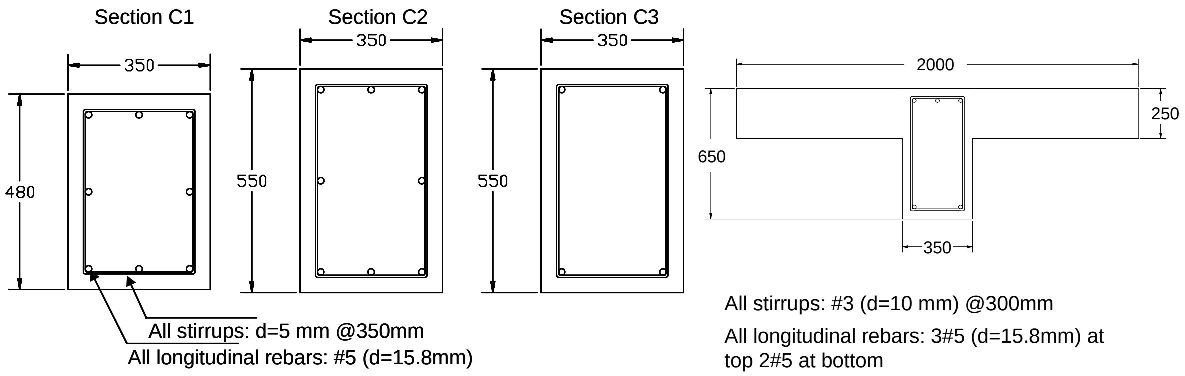
Masonry walls can be modeled with two diagonal compression struts, Madan, 1997. The strengths of the struts are determined based on the possible failure modes of infill walls. There are several potential failure modes for infill masonry walls (Paulay and Pristley, 1992) including:

- Sliding shear failure of masonry walls, horizontally
- Compression failure of diagonal strut
- Diagonal tensile cracking
- Tension failure mode (flexural).



Note: Analytical model represents shaded frame in Figure 1.

Figure 4. Analytical Model of a Reference Frame



Section C1: CW21, CW22, CW23, CE21, CE22, CE23
Section C2: CW01~08, CW14~20, CE01~08, CE14~20
Section C3: CW09~13, CE09~13
Note: The column sections and beam section are in different scale.

Figure 5. Section dimension of beam and columns

Among the above failure modes, the first and the second failure modes are the most common. In this study shear strengths for the first and the second modes are evaluated for each infill wall, and the minimum of the two is considered to be the ultimate strength of infill walls. Compression strength of masonry prism is a key parameter in the estimation of the properties of the diagonal struts. Paulay and Pristley (1992) proposed an equation for the estimation of the compression strength of masonry prism, $f_m^{'}$.

$$f_m^{'} = f_y = \frac{f_{cb}^{'}\left(f_{tb}^{'} + \alpha f_j^{'}\right)}{U_u\left(f_{tb}^{'} + \alpha f_{cb}^{'}\right)} \tag{1}$$

Due to limitation in space, a reference is made to Paulay and Pristley (1992) for details about each parameter. The above equation needs material parameters of brick and mortar, both of which are not available for the reference building. Loaiza and Blondet (2002) reported that the strength of masonry prism of typical masonry walls in Peru is approximately 13~16 MN/m^2. In this study, an average value of 14.5 MN/m^2 was assumed for the compression strength of masonry prism.

Shear strength for sliding shear failure mode can be defined as below, following Mohr-Coulomb failure criteria:

$$\tau_f = \tau_o + \mu\sigma_N \tag{2}$$

where, τ_o is cohesive capacity of the mortar beds, μ is the sliding friction coefficient along the bed joint, and σ_N is vertical compression stress in the infill wall. Typical ranges for these parameters are $0.1 \le \tau_o \le 1.5$ MPa and $0.3 \le \mu \le 1.2$, Mostafaei and Kabeyasawa (2004). For evaluation analysis purposes, it may be assumed that $\tau_o = 0.04 f_m^{'} = 0.04(14.5) = 0.58$ MPa (Paulay and Pristley, 1992). Based on experiments, Chen (2003) reports that the frictional coefficient, μ, can be defined as below:

$$\mu = 0.654 + 0.000515 f_j^{'} \tag{3}$$

where $f_j^{'}$ is mortar strength in kgf/cm^2. Assuming $f_j^{'}$=50 kgf/cm^2, frictional coefficient can be calculated as $\mu = 0.68$. Compression failure of infill walls occurs due to the compression failure of the equivalent diagonal strut. The horizontal component of the diagonal strut capacity (shear force) is,

$$V_c = ztf_m^{'} \cos\theta \tag{4}$$

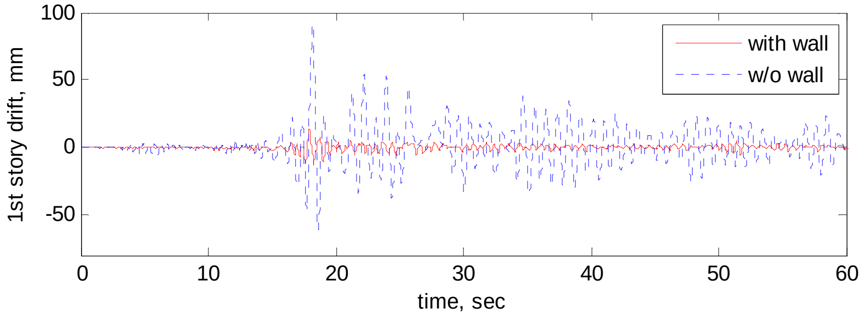
where *z* is the equivalent strut width, *t* is thickness of the infill panel and equivalent strut, and θ is the angle whose tangent is the infill height to infill wall length. The shear strengths obtained from the sliding shear failure and the diagonal compression failure may not exceed 8.3 kgf/cm^2 as recommended by ACI 530-88. The two diagonal struts of infill walls provide resistance against lateral load. In this study, it is assumed that diagonal struts behave as tri-linear in compression and have zero forces in tension. Ideally, the resistance of the infill walls after failure should be smaller than maximum resistance, strength and stiffness degradation may occur for repeated cycles, and sliding may occur. As the field observation showed that infill walls were rarely damaged, tri-linear hysteretic curves are adopted for infill walls' hysteretic model. Further details on infill wall modeling will be available at the earthquake reconnaissance report, Elnashai et al. 2008.

4.2 Nonlinear Response History Analyses

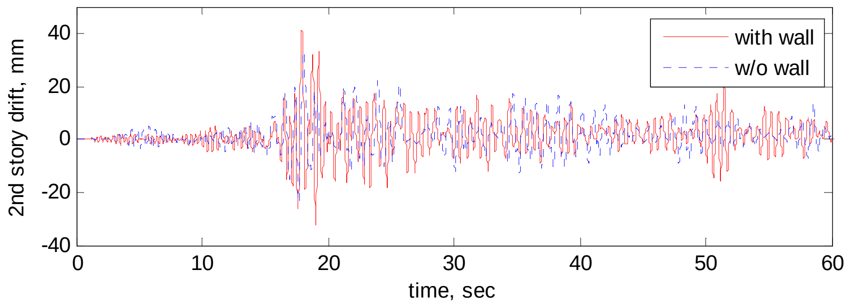
The fundamental periods of the two frames, one without infill walls and the other with infill walls, are 0.26 sec and 0.37 sec from Eigen value analysis. Nonlinear response history analyses are conducted with both structures. The E-W component of the recorded ground motion at ICA2 station, is applied to the frame. The ground motion is not scaled. The frame without infill wall experiences much larger interstory drift at the first floor than the interstory drift of frame with infill wall, Figure 6. Shear force demands on the first story columns are compared in Table 1. Note that shear force demands on the frame with infill wall is up to 50% higher than the demands on the frame without infill wall. The shear capacity of the column is calculated following ACI design guidelines, ACI 318-02. In the shear capacity calculation, the contribution of stirrups is ignored as the stirrups were very widely spaced and smooth wires with diameters of 5 mm or less are used in the construction. Shear capacity provided by concrete for the Column CW06 is based on following equation in design code.

$$V_c = 2\left(1+\frac{N_u}{2000A_g}\right)\sqrt{f_c^{'}}b_w d \tag{5}$$

From Table 1, it can be noted that the shear force demands to columns of the frame with infill walls are close to 50% larger than those to columns without infill walls. Also the seismic demand is very close to the shear capacity, 149 kN, calculated with design shear equation.



(a) First story interstory drift



(b) Second story interstory drift

Figure 6. Interstory Drifts From Nonlinear Response History Analysis

Table 1. Shear force demand on the 1st story columns

	CW01	CW02	CW03	CW04	CW05	CW06	CW07
w/o wall	86	97	100	98	99	104	100
w/ wall	109	135	140	142	143	156	146

Note: Units are kN. Shear force capacity of CW06 is 149 kN. The shear capacities of other columns are of the same magnitude.

5. CONCLUSION

Damage from the Pisco-Chincha earthquake has been more severe than the recorded peak ground accelerations suggest. To probe the causes of damage and the features of both structural capacity and demand that may have compounded the damage, a case study of a real structure from one of the worst hit areas is studied. In this study, a heavily damaged reinforced concrete structure is chosen as a reference structure. The structure experienced damage to the first story columns. Detailed observations and possible causes of the observed failure are discussed. The observed damage may have resulted from column shortening due to the construction of infill walls, low quality confining stirrups in columns, and overload on the second floor due to inferior architectural layout. The effect of infill walls on the shear force demand of columns is investigated through nonlinear response history analysis, carried out using detailed frame analysis procedure using recorded ground motion at a nearby station. The results clearly show that the shear force demand on a frame with an infill wall is much higher than the demand on a frame without infill walls. In addition, the shear force demand on a frame with an infill wall is close to or larger than the shear force capacity approximately calculated with ACI design guideline. The study employs investigation tools, from modeling assumptions to strong-motion selection, that are of general applicability to the forensic study of damaged structures in earthquake regions.

ACKNOWLEDGEMENT
The field mission to Peru was sponsored by the Mid-America Earthquake Center. The MAE Center is an Engineering Research Center funded by the National Science Foundation under cooperative agreement reference EEC 97-01785. CISMID (Center for Peruvian-Japanese Seismic Research and Disaster Mitigation), represented by Professor Carlos Zavala, and the Faculty of Civil Engineering at the National University of Peru, represented by Professor Jorge Lava, provided logistical support and valuable information.

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