

Investigation of damages on reinforced concrete buildings caused by the 1990 Philippine-Luzon earthquake

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ABSTRACT: The earthquake which occurred on Luzon island of the Philippines on July 16, 1990 caused great damage to many local buildings. The object of this paper is to investigate in detail the observed nature and the possible causes of damage to eight reinforced concrete buildings. The evaluation of the seismic capacity of these buildings are performed by applying the "Japanese Standard for Evaluation of Seismic Capacity of Existing Reinforced Concrete Buildings". It was found out that the presence of concrete hollow block walls greatly affected the seismic capacity of the buildings.

1 INTRODUCTION

An earthquake which occurred on Luzon island of the Philippines on July 16, 1990 caused great damage to many local buildings. The Architecture Institute of Japan (AIJ) sent a survey team to conduct a preliminary survey aimed at investigating the general condition of the damage caused by the earthquake and locating buildings which require more detailed investigation. Then, a second survey team was dispatched to conduct a detailed survey of (1) the damage to buildings caused

by liquefaction and other ground-related disturbance, (2) the buildings damaged by earthquake excitation and (3) the intensity of the earthquake. The authors, as members of the second survey team, were responsible for investigating item (2). The investigation was conducted mainly in the cities of Baguio and Agoo, where many buildings were damaged by the tremor.

This paper presents details of the observed nature and the possible causes of damage to eight reinforced concrete buildings which were chosen for investigation.

Table 1 Reinforced Concrete Buildings Investigated

Bldg No.	City	Type of Bldg.	No. of Storey	Structural System	Ground Condition	Footing Type	Damage Rank ¹	Main Damage
1	Baguio	Factory	3	Frame with CHB ² wall	North Bldg.: Cut Ground South Bldg.: Ground on fill	Continuous	Severe damage Collapse	Shear failure of columns and CHB walls Total collapse Fire after earthquake
2	"	Condominium	10	Frame with shear wall and CHB wall	Satisfactory Slope of a hill	Spread footings of unknown type	Severe damage	Shear failure of columns
3	"	Office	6	Frame with CHB wall	Satisfactory	"	Small damage	Small cracks in columns and girders
4	"	School	3	Frame with CHB wall	Overlying next to a cut slope	"	Severe damage	Shear failure of corner columns
5	Agoo	School	2	Frame with CHB wall	Satisfactory Flat	"	Small damage	spalling of mortar finishes
6	"	Library	2	Frame with CHB wall	Swampy	Isolated	Severe damage	Shear failure of "short columns"
7	"	Stand	2	Frame	Swampy	Isolated	Severe damage	Shear failure of "short column"
8	"	Educational facility	2	Frame with CHB wall	Satisfactory	Isolated	Slight damage	Cracks in scarcement around building

¹: damage evaluation by Murakami et al. (1988) on 1985.9 Mexico Earthquake
²: concrete hollow block

2 THE BUILDINGS INVESTIGATED

The investigation was aimed primarily at collecting data required for the examination of the seismic capacity of the buildings damaged by the earthquake. The degree of damage caused to the chosen buildings for the investigation ranged from slight to severe damage. The main activities of the investigation were to observe, to photograph, to determine the plan, the actual storey height, and the sectional areas of the columns and girders of each of these buildings. Other activities included were obtaining and consulting relevant details of the design drawings, collecting samples of reinforcing bars and concrete, and estimating the compressive strength of concrete by non-destructive tests.

Summarized in this paper are the results of the investigation of eight reinforced concrete buildings in Baguio and Agoo. Table 1 shows a list of these buildings. This table also shows the degree and condition of the damages. Most of the buildings are frame structures. One of them has reinforced concrete shear walls, but the quantity of such walls was not enough. On the other hand, walls made of concrete hollow blocks (hereafter referred to as CHB walls) are found in many of them. It may be presumed from the condition of the damage that the presence of these CHB walls greatly affected the seismic capacity of the buildings. It should also be noted that in the city of Baguio, which is located in a mountainous region, many buildings are constructed on slopes, cuts or filled-up grounds. In the city of Agoo, which is located in a coastal region, there are many buildings on swampy ground.

3 RESULTS OF INVESTIGATION

Described below are the details of the condition of the damage of four of the eight buildings listed in Table 1. The drawings of these buildings were either provided or made based on consultation with local people.

3.1 Building No.1

Outline of the building : Figure 1 shows a rough plan of the building. Figure 2 shows a standard detail of section for the columns on the first floor. The building is a three-storey structure and was constructed in 1984. The floor and girders are prestressed. The building has 3 spans (@ 9.0 meters) in the transverse direction and 18 spans (@ 6.0 meters) in the longitudinal direction. The storey height is about 3.6 meters. The sectional area of each column is 46 cm square. The main reinforcement is 8-D32 and the hoop is D10-@300 for the side columns and 16-D32

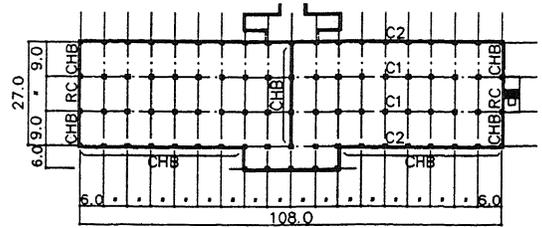


Fig. 1 Plan of building No. 1

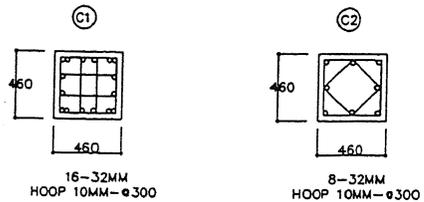


Fig. 2 Details of columns

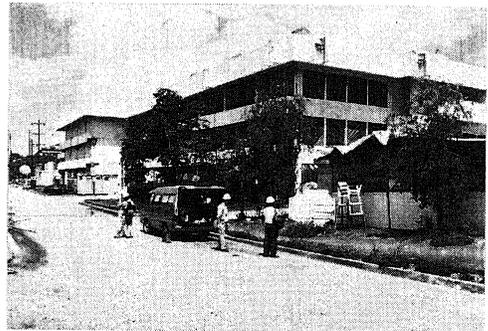


Fig. 3 Overall view of building No. 1

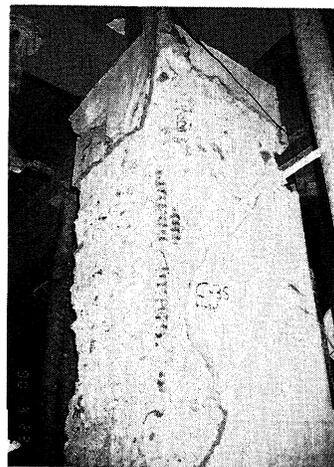


Fig. 4 Bond-slip failure in column

(main) and D10-@300 (hoop) for the interior columns.

Condition of the damage : Of the two buildings that were built with the same specifications, one was destroyed and the other was severely damaged. Figure 3 shows the severely damaged building. There were shear cracks and bond slip failures in almost all of its columns (Figure 4). Its side columns, in the longitudinal direction in particular, had shear cracks and had been transformed into short columns due to the presence of oblong windows on top of the CHB walls and right under the girders. The residual relative rotation measured on the columns on the first floor ranged from 1/200 to 1/100. It is assumed that there was such a difference in the degree of damage between the two buildings, in part, because one (which was destroyed) was built on a filled-up ground while the other (which was severely damaged) was built on a cut ground.

3.2 Building No.2

Outline of the building : The building is a 10-story reinforced concrete condominium built on a southern slope. It was constructed in 1973. Figure 5 shows the typical floor plan of the building. The building is made up of three structures connected to each other with expansion joints. Found in the central part of the building is a reinforced concrete core wall, which provides spaces for elevators and staircases.

Condition of the damage : Figure 6 shows an overall view of the building. There were bending shear failures in the columns in the southeastern part of the first floor, and the main reinforcing bars buckled. There were shear failures in the capitals of the columns near the entrance, which were caused by the columns' transformation into short columns due to the presence of CHB walls (Figure 7).

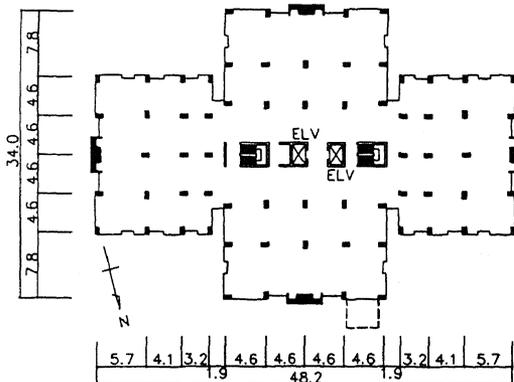


Fig.5 Plan of building No.2

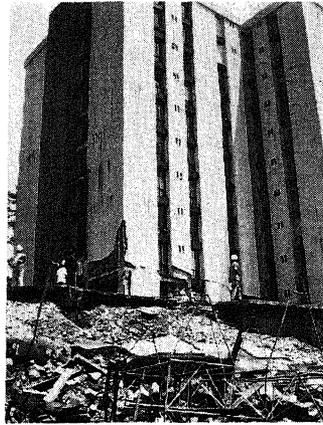


Fig.6 Overall view of building No.2

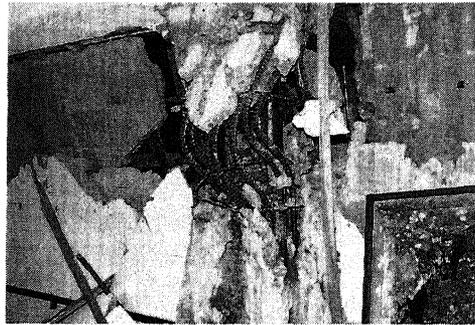


Fig.7 Shear failure in CHB wall and column

3.3 Building No.6

Outline of the building : This is a two-story building with reinforced concrete column, and prestressed girders and floors. The building was constructed in 1987. It is 5 spans long in the longitudinal direction and is 3 spans long in the transverse direction. There is a 4 span-long extension in the longitudinal direction. It was still under construction and had concrete columns as high as the floor of the second storey when the earthquake occurred. Figure 8 shows a rough structural plan and a standard detail of section for the columns on the first floor. The central span in the transverse direction is 16 meters long, and prestressed girders were used (sectional area : 30 cm x 75 cm). The foundation consists of isolated footings with no tie beams. Short CHB walls with cased openings on top were found connecting the columns along rows B and E. In general, CHB walls were used as partitions. The roof was made of wooden truss covered with corrugated GI sheets.

Condition of the damage : Figure 9 shows an overall view of this building. Shear failures

in the reinforced concrete columns, which were transformed into short columns due to the presence of CHB walls in rows B and E, were the most noteworthy damage (see Figure 10). Many internal and external CHB walls collapsed and there were many cracks in them. It should be noted that no damage was caused to the girders and floors with long spans, to the columns at the second floor, neither to the extension of the existing building.

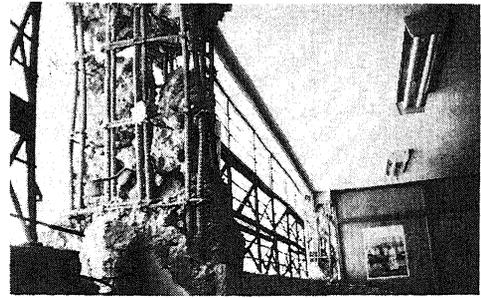
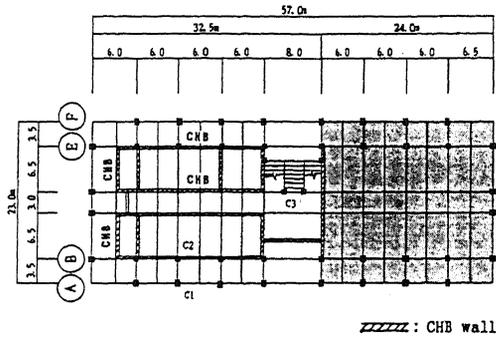
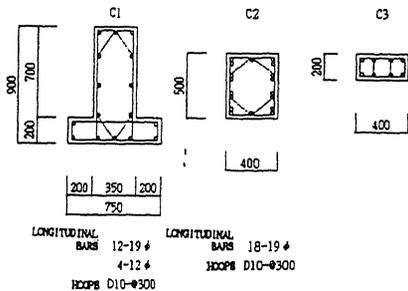


Fig.10 Shear failure in Column with CHB wall



a) structural plan



b) details of columns

Fig.8 Structural plan and details of columns

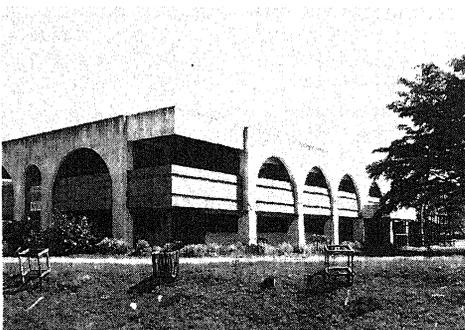


Fig.9 Overall View of Building No.6

3.4 Building No.8

Outline of the building : This is a two-story reinforced concrete building. Figure 11 shows the structural plan of the first floor of the building. Figure 12 shows the detail of section for the typical columns, girders and foundation beams. The height of the first floor is 3.4 meters. The roof is a wooden truss covered with corrugated GI sheets. The footing is an isolated footing with tie beams. The ground condition is satisfactory.

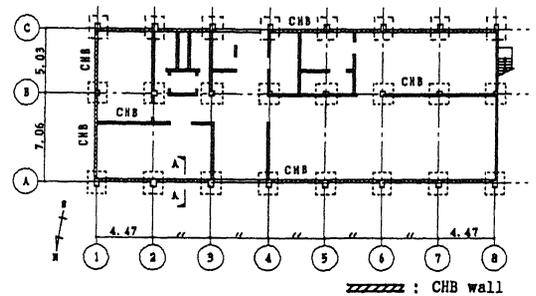
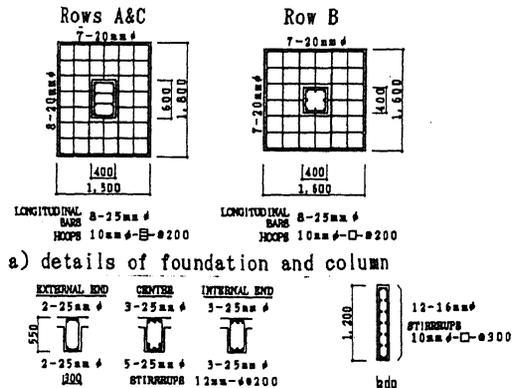


Fig.11 Structural plan of building No.8



a) details of foundation and column b) details of girder c) details of tie beam

Fig.12 Details of columns and girders

Condition of the damage : At a first glance the building does not appear to have been damaged by the earthquake except for the cracks in the scarcement around the building. It may be conjectured that the CHB walls with adequate numbers reinforcing bars minimized the impact of the earthquake.

3.5 Material characteristics

Table 2 shows the tension test results of the reinforcing bars used in Building No.1 and No.6. All these bars were used as main reinforcing bars. The yield strength of one test piece was less than 300 MPa, but higher values were obtained for all the other test pieces. Although in some buildings damaged by the earthquake, reinforcing bars broke due to their brittleness. The results of this test indicate that most of the reinforcing bars have satisfactory elongation percentages. These reinforcing bars are considered to fall into the category of SD295 or SD345 in the Japanese standard.

Pieces of concrete taken from Building No.1 which was being demolished were transformed into square pillar-shaped test pieces for compression test. Table 3 shows the results of the compression test. Table 4 shows the results of a non-destructive test (the Schmidt hammer method) conducted to estimate the compressive strength of the concrete used in Buildings No.2, No.3 and No.4. Some of the values of the compressive strength shown in Table 3 are very low. In light of the fact that the test pieces were prepared using pieces of damaged concrete, however, it can be said that the concrete used in these buildings was satisfactory in terms of compressive strength.

4 OBSERVED CAUSES OF THE DAMAGE

Concrete CHB walls are used in large numbers not only in the above-mentioned eight buildings but also in most of the other buildings in the Philippines. The investigation has also revealed that there are two types of concrete CHB walls. One type is sufficient in terms of seismic capacity while the other type is not. In Buildings No.1, No.2 and No.6, most CHB walls were built under cased openings, which adversely affected the unity of the CHB walls and other structural members, causing shear failures in the CHB walls. To examine quantitatively the correlation between the degree of damage and the quantity of CHB walls, the average shear stress and the seismic capacity index (Is) were calculated. The average shear stress is obtained by dividing the value of the weight of the building by the combined total sectional area of the columns, walls and CHB walls. While the index Is is calculated in

Table 2 Material Test Results of Steel Bars

Building No.	Average Diameter (mm)	Area (mm ²)	Elongation (%)	Yield Strength (MPa)	Ultimate Strength (MPa)
1	32.20	814.33	26.97	356.5	550.4
6	22.10	383.60	21.10	296.6	460.1
	19.80	307.91	22.86	367.9	560.6
	22.65	402.93	25.21	275.0	425.9

Table 3 Compressive strength of concrete test pieces taken from Building No.1.

Specimen No.	Width a (mm)	Length L (mm)	a/L ratio	Area a ² (mm ²)	Compressive Strength	
					Measured (MPa)	Estimated (MPa)
1	50.19	47.50	1.06	2519.04	40.5	33.3
2	47.72	47.85	1.00	2277.20	35.1	28.5
3	47.19	44.15	1.07	2228.90	47.3	39.8
4	45.71	90.58	0.50	2069.40	18.8	14.2
5	71.42	141.95	0.50	5100.80	23.8	18.4

* Estimated compressive strength (f_{cy}) of cylinder by Neville(1981)

$$f_{cy} = (0.78 + 0.2 \log_{10} f_{cu}/199.67) f_{cu} \quad f_{cu} : \text{compressive strength of cube}$$

Table 4 Compressive strength estimated by non-destructive test

Bldg. No.	Structural member tested	Compressive strength (MPa)
2	1st floor column	37.3
	1st floor shear wall	27.5
3	column	28.4
4	column	29.4
	column	24.5

accordance with the Japanese Standard for Evaluation of Seismic Capacity of Existing Reinforced Concrete Buildings (refer to Okada et al. (1986)). The index Is can be expressed as follows.

$$I_s = E_o \times S_D \times T$$

where E_o : Basic structural index
S_D : Structural design index
T : Time index

The value of Is can be calculated in three types of screening procedure (first level through third level screening procedure). The higher the level of the screening procedure, the greater is the reliability of the results of calculation. The Japanese standard does not specify any method for handling CHB walls. In the examination, the value of Is was assessed on the assumption that only those CHB walls which were built in the framework can be considered and that the average shear stress on CHB walls is 0.49 MPa (refer to Ryo, et al.(1963)). In the case of

shear failures on columns due to insufficient unity of the CHB walls and the framework, only the short columns can be considered.

Table 5 shows a list of the calculated values of the average shear stress (τ) and those of I_s obtained from the result of the first level screening procedure. Generally, the greater is the average shear stress and the smaller is the value of I_s , the greater is the degree of damage. This applies to Buildings No.1 and No.2. In the case of Buildings No.5 and No.8 which have a relatively high values of I_s , the degrees of damage range from small damage to slight damage. This is also consistent with the above generalization.

In the case of Building No.3, the degree of damage was small despite a low value of I_s . The presence of CHB walls, properly arranged and relatively in large number outside of the framework, presumably worked to minimize the damage to this building. Conversely, in the case of Buildings No.4, No.6 and No.7, the degree of damage was severe despite relatively high values of I_s . It appears that the part of Building No.4 which borders the slope was severely damaged due to ground movement. Building No.6, which had isolated footings without tie beams, seems to have been severely damaged because it was built on a swampy ground and because the stress concentrated on the CHB walls which had been transformed into short columns. Building No.7, an outdoor basketball bench stand was damaged due to the presence of short columns in front of the stand. Furthermore, Buildings No.1 and No.6 went through the second level screening procedure. The resulting value of I_s was 0.28 in the X direction and 0.67 in the Y direction for Building No.1, and 0.69 in the X

direction and 0.61 in the Y-direction for Building No.6. This substantiates the results of the first level screening procedure.

5 CONCLUSION

The conclusions that may be derived from the earthquake damage investigation can be summarized as follows:

1. The presence of concrete hollow block walls greatly affected the seismic capacity of the building.
2. The strength and deformation properties of CHB infill wall have to be clarified to accurately determine the seismic capacity of the buildings with CHB walls by applying the "Japanese Standard for Evaluation of Seismic Capacity of Existing Reinforced Concrete Buildings".

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Table 5 Average shear stresses (τ) and I_s -indices by 1st level screening

Bldg. No.	Total floor area A_f (m ²)*1	Total weight W (x10 ³ KN)	Total section area of column A_c (cm ²)	Total section area of wall A_w (cm ²)	Total section area of CHB A_b (cm ²)	Average shear stresses τ (MPa) *2	I_s indices
1	5,830	51.0	160,800	X 0	0	3.17	0.18
				Y 25,600	119,500	1.67	0.34
2-1*3	6,640	53.9	63,700	X 96,500	18,400	3.02	0.54
				Y 51,500	18,700	4.03	0.32
2-2*4	2,620	22.8	43,200	X 0	24,600	3.37	0.13
				Y 22,500	14,500	2.85	0.37
3	2,070	20.3	53,824	X 0	39,000	2.19	0.35
				Y 0	0	3.78	0.26
4	1,010	9.9	59,700	X 0	40,500	0.99	0.79
				Y 0	73,500	0.75	0.95
5	3,530	26.4	294,000	X 0	178,000	0.58	0.42
				Y 0	90,000	0.89	0.36
6	2,050	12.7	126,800	X 0	12,000	0.92	0.69
				Y 0	92,250	0.58	0.70
7	650	5.1	82,600	X 0	0	0.62	1.08
				Y 0	0	0.62	1.34
8	760	6.1	51,200	X 0	20,100	0.84	0.54
				Y 0	62,500	0.53	0.65

*1 : excluding first floor area

*2 : $\tau = W / (A_c + A_w + A_b)$

*3 : central structure of Bldg. No.2

*4 : side structure of Bldg. No.2