11 WCEE

Copyright © 1996 Elsevier Science Ltd
Paper No. 783. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

SEISMIC EVALUATION OF BELEN CHURCH IN MERIDA, VENEZUELA

A. I. UZCATEGUI and P. J. MONTILLA

Departamento de Estructuras, Facultad de Ingeniería, Universidad de Los Andes,
Av. Tulio Febres Cordero, Mérida, Venezuela.

ABSTRACT

The present paper forms part of a research project on catholic churches of the State of Mérida, developed by the group of seismic research at the University of the Andes. In this case a structural evaluation in the inelastic range is presented and also a study of the escape routes of a typical church of the city of Mérida, considering the possible occurrence of earthquakes of different frequencies content. The longitudinal and transverse frames were modeled including masonry walls subjecting them to different levels of seismic excitation; as a result the transverse frames whose top columns do not continue to the ground, collapsed for low levels of the excitation. The other transverse frames collapsed for higher levels of excitation. In the longitudinal frames collapse occurred at the top part of the Tower and generalized damage in the top level of the Central Nave, also shear failure of a short column located in the discontinuity zone between the Nave and the Presbytery. It is concluded that the church is highly vulnerable under the effects of moderate earthquakes. The blockage of the main exits contributes to increment the risk for the occupants.

KEYWORDS

Churches; seismic evaluation; inelastic analysis; structural collapse; short column; vulnerability; escape routes.

INTRODUCTION

The city of Mérida is located on a natural terrace of the Andes Mountain Range in Venezuela, which is crossed by the Boconó geological fault, one of the most actives in the country. Historically this fault has produced catastrophic earthquakes that have destroyed a great part of the city. Specially the churches, that due to their type of construction and structural configuration, have collapsed or suffered severe damage. Due to the high occupational index of these buildings during earthquakes numerous life losses have been caused, and also the loss of historical buildings. A return period of one hundred years has been estimated for the occurrence of earthquakes of magnitude greater than seven in the region; at present this period has been surpassed, thus an earthquake of great magnitude is expected in a short term. Due to the previous circumstances a seismic vulnerability study was done of eighteen catholic churches of the State of Mérida. In this study it was determined that some of them presented problems of behavior, among them the Belén church (Uzcátegui et al., 1992). For this reason this church was selected to perform a more detailed study.

The objectives of this paper are: the evaluation of the church subjected to different seismic actions until its collapse has occurred; the determination of the non-structural elements which are vulnerable and the evaluation of the exit routes.

DESCRIPTION OF THE BUILDING

This church is constituted by a Central Nave of 10.20 meters wide and double height, 7.90 meters in the first level and 13.60 meters in the second level, both measured from the base; two lateral naves of 3.30 meters wide each and a height that varies from 5.60 to 7.90 meters; it has a total length of 53.50 meters (Fig. 1). At the front there are two Towers of 22.00 meters of height (Photos 1 and 2). The Chorus is located at the entrance, on a second level. The Presbytery and the Sacristy are at the rear of the church. The structure is made up of longitudinal and transverse frames of reinforced concrete. The roof structure is of steel trusses that rest on beams and columns of reinforced concrete. The Central Nave has two levels made up of intermediate longitudinal beams which are very deep (0.30 x 1.50 m) and therefore very stiff. These beams interrupted at the Chorus and the Presbytery create zones of discontinuity, and top beams of 0.30 x 0.40 m. At the lower level the columns are circular of 0.85 m in diameter, they change at the top level into rectangular columns with variable sections from 0.65 x 0,40 m to 0.45 x 0.40 m. In between those last ones there are other columns of 0.25 x 0.40 m that rest on the longitudinal beams at the center point of the span, and do not continue to the ground (Photo 3). The Lateral Naves have inclined beams that rest at the inner end on the circular columns or on the longitudinal beam, and at the other end rest on columns of 0.40 x 0.70 m and 0.25 x 0.30 m, alternately. The Presbytery located at the rear end of the church, is made up of beams and columns of 0.30 x 0.30 m. The Chorus and Presbytery have reinforced concrete arches that rest on circular columns of 0.85 m in diameter and square columns of 1.00 m wide, respectively (Photos 3 and 4). The roof has a false ceiling of laminated asbestos cement, very heavy, of 0.01 m of thickness. The lamps of the Central Nave suspended from the roof, are of metallic material and also very heavy.

EVALUATION OF THE SEISMIC VULNERABILITY

For the structural evaluation an elastic 3D analysis was done initially using a computer program based on the finite element method, including the masonry walls in the model (Uzcátegui et al., 1992). At this first stage it was determined that the transverse and central longitudinal frames showed problems of behavior, thus it was decided to do a two-dimensional study of these in the inelastic range. For the inelastic analysis the DRAIN2D program was used applying two accelerograms, the Monay earthquake, registered on the local seismological net, at an epicentral distance of 121 km and the El Centro earthquake north-south component, that was registered on a soil of similar characteristics as those of the soil where the building under study is located, with the object of subjecting the structure to earthquakes of a close source and of a far away source. Both earthquakes were scaled to different values, that represented some levels of seismic actions prescribed by the seismic code. With this purpose the structure was analyzed under the action of vertical loads and earthquakes with a peak ground acceleration of 10% of gravity (g) and earthquakes that produce the collapse of the structure. In Fig. 2 the two types of transverse frames analyzed are showed, Frame A: with columns that are continuous to the ground and the Frame B: with columns that are not continuous to the ground. To take into account the effect of the stiffness of the longitudinal beams of 0.30 x 1.50 m on the transverse frames, an additional member was included in the model which represents the mentioned effect. This was done with a simplified 3D model to determine the displacement and rotations that allow the calculation of the equivalent flexure and axial stiffness. Figure 3 shows one of the longitudinal frames of the Central Nave, that includes the Tower, the longitudinal stiff beam and the Presbytery. In this frame the following structural details can be seen: the discontinuity of the longitudinal stiff beam at the Chorus and Presbytery zones, and the intermediate columns at the second level that do not continue to the ground. In the model of the longitudinal frame, at the top levels of the Tower and at the Presbytery, wall elements were included to take into account the change of stiffness produced by the masonry walls.

RESULTS OF INELASTIC STRUCTURAL ANALYSIS

Transverse frames. In table 1 results are shown of resisting moments Mr, acting moments Mac, and the relation between them (Mr/Mac) for frames A and B and earthquakes studied. Case 1 represents vertical load, case 2 the earthquakes scaled to 10% of g and case 3 the collapse of the structure.

Table 1. Results obtained for transverse frames.

Selected Earthquake and Transv. Frame	Element	Joint	Case 1 Mr (kg-m)	Case 1 Mac (kg-m)	Case 1 Mr/Mac	Case 2 Mr (kg-m)	Case 2 Mac (kg-m)	Case 2 Mr/Mac	Case 3 Mr (kg-m)	Case 3 Mac (kg-m)	Case 3 Mr/Mac
	1.-Column	1	-37365	-302	123.7	-37365	-951	39.3	-37365	-2413	15.5
El Centro		5	-37365	-447	83.8	-37365	-840	45.5	-37365	-1675	22.3
	2.-Column	4	37365	302	123.7	-37365	-386	96.8	-37365	-1894	19.7
with		6	37365	447	83.8	37365	38	977.9	-37365	-841	44.4
circular	3.-Column	2	34000	139	244.6	-34000	-3074	11.1	-34000	-10569	3.2
columns		7	34000	31	1096.8	-34000	-4143	8.2	-34000	-13949	2.4
Frame A	4.-Column	3	-34000	-139	244.6	-34000	-3494	9.7	-34000	-11039	3.1
(Fig. 2. (a))		8	-34000	-31	1096.8	-34000	-4313	7.9	-34000	-14076	2.4
	5.-Column	7	-13700	-640	21.4	13700	3692	3.7	13700	13697	* 1.0
Collapse		9	-13700	0	——	13700	0	——	13700	0	——
at	6.-Column	8	13700	640	21.4	13700	4977	2.8	13700	14601	* 0.9
t = 1.0 sec		12	13700	0	——	13700	0	——	13700	0	——
	7.-Beam	5	30032	447	67.3	30032	854	35.2	30032	1869	16.1
		7	30032	609	49.3	30032	319	94.2	-44676	-361	123.8
	8.-Beam	6	-44676	-447	100.1	30032	33	913.7	30032	1079	27.8
		8	-44676	-609	73.4	-44676	-849	52.7	-44676	-1466	30.5
	1.-Column	1	-37365	-302	123.7	37365	1132	33.0	37365	3536	10.6
Monay		5	-37365	-447	83.8	37365	861	43.4	37365	3084	12.1
	2.-Column	4	37365	302	123.7	37365	1840	20.3	37365	3881	9.6
with		6	37365	447	83.8	37365	1845	20.3	37365	3734	10.0
circular	3.-Column	2	34000	139	244.6	34000	943	36.1	34000	2214	15.4
columns		7	34000	31	1096.8	-34000	-3343	10.2	-34000	-9065	3.8
Frame A	4.-Column	3	-34000	-139	244.6	34000	947	35.9	34000	1825	18.6
(Fig. 2. (a))		8	-34000	-31	1096.8	-34000	-3155	10.8	-34000	-8460	4.0
	5.-Column	7	-13700	-640	21.4	13700	5101	2.7	13700	14715	* 0.9
Collapse		9	-13700	0	——	13700	0	——	13700	0	——
at	6.-Column	8	13700	640	21.4	13700	6369	2.2	13700	14827	* 0.9
t = 5.1 sec		12	13700	0	——	13700	0	——	13700	0	——
	7.-Beam	5	30032	447	67.3	-44676	-879	50.8	-44676	-3023	14.8
		7	30032	609	49.3	-44676	-2027	22.0	-44676	-6406	7.0
	8.-Beam	6	-44676	-447	100.1	-44676	-1819	24.6	-44676	-3684	12.1
		8	-44676	-609	73.4	-44676	-3284	13.6	-44676	-7138	6.3
	1.-Column	1	-2435	-10	251.0	-2435	-299	8.1	-2435	-325	7.5
El Centro		5	-2435	-6	425.7	-2435	-148	16.4	-2435	-161	15.1
	2.-Column	4	2435	10	251.0	-2435	-280	8.7	-2435	-306	8.0
without		6	2435	6	427.2	-2435	-136	17.9	-2435	-150	16.3
circular	5.-Column	7	3258	72	45.4	3258	2990	* 1.1	3258	3258	* 1.0
columns		9	3258	0	——	3258	0	——	3258	0	——
Frame B	6.-Column	8	3258	72	45.4	3258	3133	* 1.0	3258	3267	* 1.0
(Fig. 2. (b))		12	3258	0	——	3258	0	——	3258	0	——
	7.-Beam	5	3900	6	684.2	3900	144	27.2	3900	156	25.0
Collapse		7	3900	6	696.2	3900	76	51.3	3900	82	47.3
at	8.-Beam	6	-3900	-6	684.2	3900	132	29.5	3900	145	26.9
t = 1.6 sec		8	-3900	-6	696.2	3900	65	60.2	3900	73	53.4
	1.-Column	1	-2435	-10	251.0	2435	90	27.00	2435	152	16.0
Monay		5	-2435	-6	425.7	2435	57	42.42	2435	96	25.3
	2.-Column	4	2435	10	251.0	2435	108	22.57	2435	169	14.4
without		6	2435	6	427.2	2435	68	35.91	2435	106	23.1
circular	5.-Column	7	3258	72	45.4	3258	2030	1.61	3258	3259	* 1.0
columns		9	3258	0	——	3258	0	——	3258	0	——
Frame B	6.-Column	8	3258	72	45.4	3258	2178	1.50	3258	3270	* 1.0
(Fig. 2. (b))		12	3258	0	——	3258	0	——	3258	0	——
	7.-Beam	5	3900	6	684.2	3900	59	66.67	-3900	-98	39.75
Collapse		7	3900	6	696.2	3900	74	52.49	-3900	-124	31.54
at	8.-Beam	6	3900	-6	684.2	3900	69	56.69	-3900	-107	36.38
t = 3.9 sec		8	3900	-6	696.2	3900	85	45.99	-3900	-131	29.83

For vertical loads the safety factors are high in both transverse frames. For 10% of g it can be seen that in frame B members 5 and 6 are close to failure with factors very close to the critical condition (Mr/Mac $\cong$ 1). This shows that under moderate earthquakes the building will have problems in those frames whose central columns do not reach the ground. In Fig. 4 the collapse mechanisms are shown for transverse frames under the selected earthquakes. For both earthquakes failure is produced by the formation of a lateral mechanism at the top level. In Frame A collapse is produced under the El Centro scaled at 30.3% of g at time t = 1.0 sec, and under the Monay scaled at 31.1% of g at time t = 5.1 sec. In Frame B collapse is produced for El Centro scaled at 11.5 of g and t = 1.6 sec, and for Monay scaled at 16.2% of g and t = 3.9 sec. It can be seen that the most probable failure mechanism occurs in Frame B, with non continuous columns, which indicates that this frame is the most vulnerable of the structure; the church will have behavioral problems in these frames for moderate levels of seismic actions and in general, El Centro earthquake is more destructive than Monay. This is due to the fact that although both earthquakes were registered on hard soil, their frequencies content is different. Table 2 shows the interstorey drift obtained in Frame B for both scaled earthquakes. According to the Venezuelan Seismic Code (Norma Venezolana, 1982), for severe earthquake the allowable drift is 0.015, and it is considered in this study that for moderate earthquakes with an assumed ductility of 2 for the structure, the lateral drift is of 0.0075. Comparing these values with those obtained in cases 2 and 3 of the Table 2, it is concluded that they are not surpassed; therefore the lateral drift is not a determinant factor in the seismic behavior of the frame. The values of lateral drift for Frame A are smaller than those of Frame B.

Table 2. Study of lateral drift in transverse frames.

Selected Earthquake and Transv. Frame	Element	Joint	Case 1 Δi (cm)	Case 1 Drift αi	Case 2 Δi (cm)	Case 2 Drift αi	Case 3 Δi (cm)	Case 3 Drift αi
El Centro	1	1	0.0000		0.0000		0.0000	
		5	0.0060	0.000011	0.1962	0.000350	0.2132	0.000381
without	2	4	0.0000		0.0000		0.0000	
		6	0.0060	0.000011	0.1841	0.000329	0.2010	0.000359
circular	5	7	0.0059		0.1938		0.2105	
columns		9	0.0384	0.000057	0.9435	0.001995	1.0309	0.002178
Frame B	6	8	0.0059		0.1819		0.1987	
(Fig. 2. (b))		12	0.0384	0.000057	1.0202	0.002109	1.1077	0.002292
Monay	1	1	0.0000		0.0000		0.0000	
		5	0.0060	0.000011	0.0536	0.000096	0.0906	0.000162
without	2	4	0.0000		0.0000		0.0000	
		6	0.0060	0.000011	0.0645	0.000115	0.1014	0.000181
circular	5	7	0.0059		0.0548		0.0925	
columns		9	0.0384	0.000057	0.9194	0.000726	1.5102	0.002487
Frame B	6	8	0.0059		0.0656		0.1031	
(Fig. 2. (b))		12	0.0384	0.000057	0.9964	0.001633	1.5874	0.002604

Longitudinal Frames. The collapse of the last level of the Tower occurs under the action of El Centro scaled to 10% of g. The most severe damages in these frames occur for the same earthquake without scaling, at time t = 1.2 sec, represented in Fig. 5, characterized by the collapse of the last level of the Tower and severe damage at the top of the Chorus, the Central Nave and the Presbytery. It can be seen that the lateral stability of the structure is maintained by the contribution of the columns continuous to the ground of the Central Nave. For Monay scaled to 16.2 % of g, at time t = 1.5 sec, the collapse of the last level of the Tower was observed, with no more structural damages. Scaling this earthquake to 30% of g, the maximum ground acceleration expected at the site according to the Venezuelan Seismic Code, the previous behavior is maintained. It was found that for both earthquakes scaled to levels of accelerations above 30% of g, the acting shear surpassed the shear strength in the short column located in the Presbytery, failing it by shear (Fig. 5). The shear ratio history for El Centro earthquake is shown in Fig. 6. The shear strength was calculated using the equations obtained for R. C. short columns by Umehara (Umehara et al.,1984). In these frames, the values of lateral drift are less than the allowable values. In the 3D analysis done previously, it was observed that the arches of the Presbytery and of the Chorus, because they are not braced, contribute to create zones that are vulnerable and could cause the collapse of that part of the building. Also, the structure of the Chorus is very stiff (Photo 3), and attracts a great part of the inertial forces, localizing the damage in the surrounding zones (Fig. 5).

VULNERABILITY OF NON-STRUCTURAL ELEMENTS

The false ceiling is made of laminated asbestos cement, is very heavy and simply supported from the roof trusses by flexible braces (Photo 3). These elements could deform considerably and cause the lamina to fall down representing a great hazard for the occupants. The metallic lamps of the Central Nave are heavy and are hanging from the roof trusses with chains welded to them (Photo 3); they are hexagonal in form and have sharp edges, that could represent a hazard to the occupants if they fell. The longitudinal beam of the Central Nave is decorated with a concrete covering of 0.06 m of thickness, with great outcrops at the top and bottom of the beam (Photo 4). The seismic experience obtained from other similar buildings (Montilla et al.,1996), has shown that this type of decoration can fall down easily, being very dangerous for the users of the building. The religious images are simply placed on wooden pedestals without bracing and could overturn.

EVALUATION OF THE ESCAPE ROUTES

The church has three exit routes: the main door and two lateral doors. These last two are permanently closed with padlocks and the keys are normally kept in a distant place, which impedes the exit of people through these doors. The main door is of 2.60 m width, and is delimited internally by a wooden closure which has three doors, two lateral ones of 0.90 m width and a central one of 2.60 m that remain closed (Fig. 1 and Photo 4). The final exit routes are limited only to the two small lateral doors of the wooden closure; which makes it very difficult to evacuate quickly the occupants during an earthquake, creating a great risk to them.

CONCLUSIONS

In this paper a seismic evaluation of a catholic church of traditional construction in Venezuela is done. From the Study it is concluded that the transverse frames of the church whose central columns do not continue to the ground level are very vulnerable, collapsing for an earthquake with low level of accelerations, close to 11% of gravity. Also the longitudinal frames present collapse in the Tower for the same level of accelerations, probably due to the effect of the sudden change of stiffness between the structure of the Central Nave and the Tower. It was observed that the El Centro earthquake produces more structural damages than the Monay one. This is possibly due to the fact that an earthquake close to the site, with high content of high frequencies results more dangerous for this building with a short period than a far away earthquake with longer periods. The great stiffness and lack of continuity of longitudinal beams of the Central Nave in the Presbytery produce a high concentration of stress that cause severe damage in that area. This has occurred in similar churches that have supported earthquakes of considerable magnitude. The placing of masonry walls in the Presbytery and Chorus in a discontinuous form, contribute to create stress concentration in those zones. The arches without bracing contribute to create vulnerable zones in the building. The elimination and closing of escape routes impedes the quick evacuation of people, makes panic greater, increments seismic risk and could cause the loss of lives. The placing of a heavy false ceiling with inadequate tying, heavy coverings with insufficient anchorage, heavy and sharp lamps located at considerable heights and religious images without adequate fastening are highly dangerous for building occupants.

REFERENCES

Norma Venezolana para Edificaciones Antisismicas (1982). COVENIN-MINDUR. Caracas, Venezuela.

Montilla, P. J., A. I. Uzcátegui and S. M. Hernandez (1996). Damage occurred to churches during the earthquake of February 8th, 1995 in Pereira, Colombia. 11WCEE, Acapulco, Mexico.

Umehara, H. and J. O. Jirsa (1984). Behavior and design of short R. C. columns. Vol. 5, pp. 877-884, 8WCEE, San Francisco, USA.

Uzcátegui, A. I., P. J. Montilla, S. M. Hernandez, R. Torres and A. DeBarcia (1992). Evaluación sísmica de la iglesia de Belén. Facultad de Ingeniería, Universidad de Los Andes, Mérida, Venezuela.

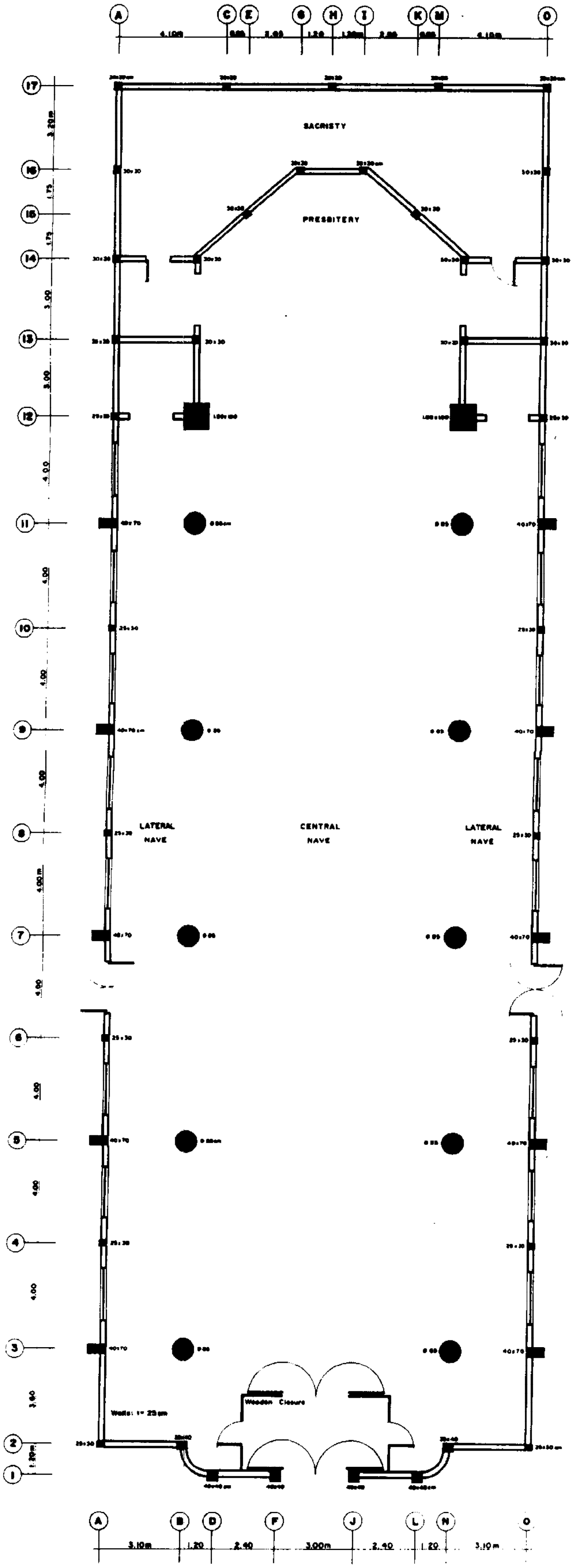
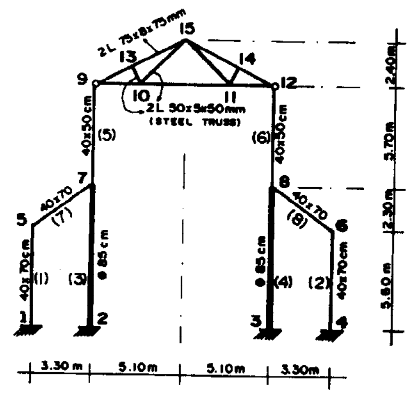
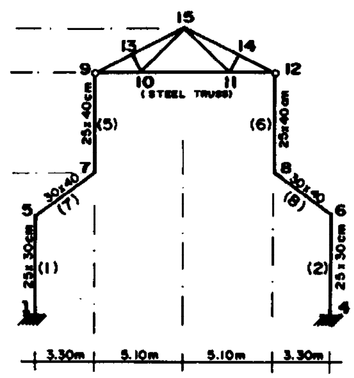


Fig. 1. Plan view of Belen Church.



(a). Frame A. With central columns continued to ground level.



(b). Frame B. Without central columns continued to ground level.

Fig. 2. Transverse Frames.

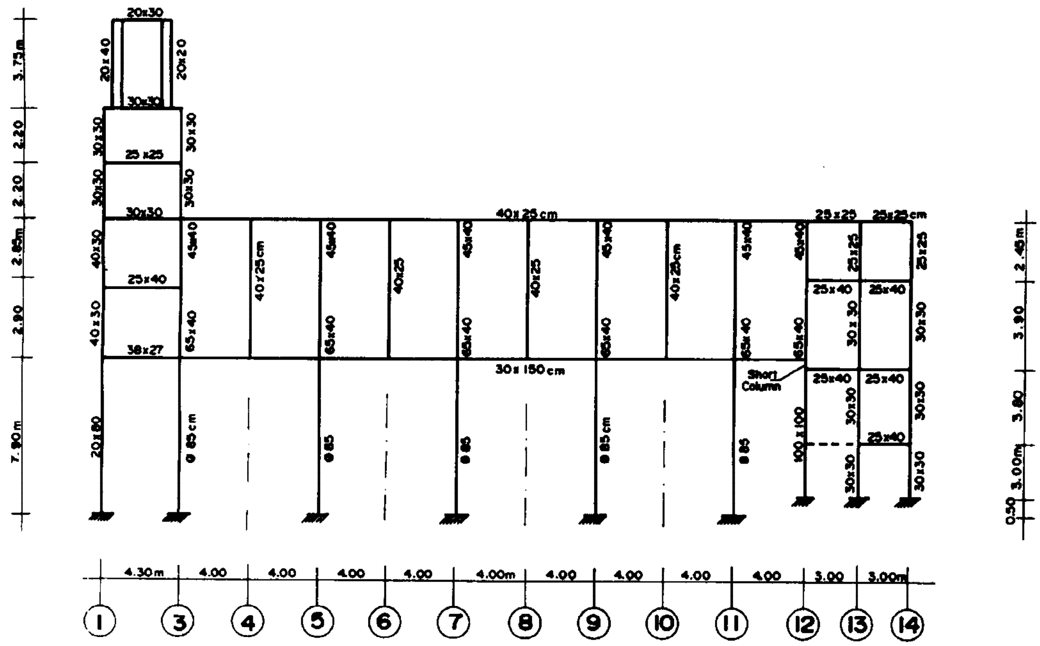
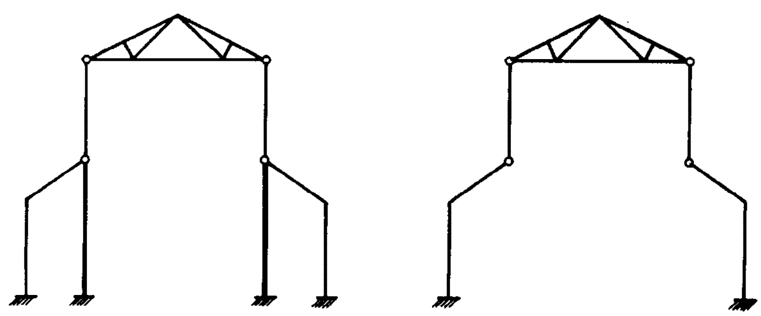


Fig. 3. Longitudinal Frames of Belen Church.



(a). Collapse for Frame A. (b). Collapse for Frame B.

Fig. 4. Collapse mechanism for transverse frames.

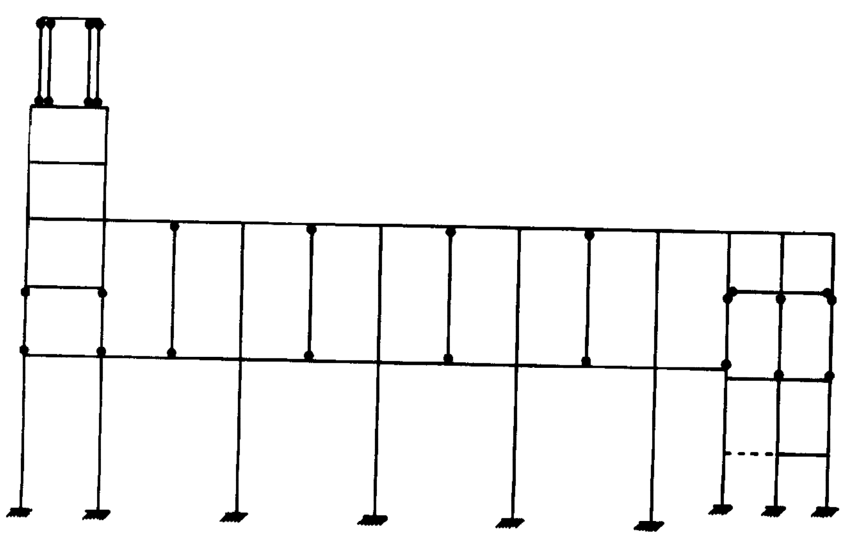


Fig. 5. Partial collapse mechanism in longitudinal frames.