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ABSTRACT

The occurrence of the November 25, 1988 Saguenay earthquake prompted a site-visit by two structural engineers and one geotechnical engineer representing the Canadian National Committee on Earthquake Engineering. This report contains some preliminary information on the ground motion records (obtained by the Geological Survey of Canada), subsoil conditions, geotechnical observations and both architectural and structural damage.

The significant role played by the soft subsoil regions in different parts of the province is clearly illustrated. The poor performance of unreinforced masonry, in many structures, is discussed. The susceptibility of embankments to stability failures was clearly demonstrated by several slope failures.

The report concludes with warnings to the engineering profession and to government agencies over the abundance of unreinforced masonry , particularly in hospitals and schools. There is a very real possibility that this brittle form of construction could lead to severe damage or collapse resulting in loss in future seismic events of equal or greater magnitude.

RÉSUMÉ

Suite au tremblement de terre du Saguenay le 25 novembre 1988, une équipe, formée de deux ingénieurs en structures et un ingénieur en géotechnique, a entrepris une visite de la région. Cette équipe représentait le comité national Canadien de génie séismique. Ce rapport présente des informations préliminaires concernant les enrégistrements séismiques (effectués par la commission géologique du Canada), les conditions des sols, les observations d'ordre géotechnique ainsi que les dommages architecturaux et structuraux qui furent causés par cet événement.

Le mouvement séismique, amplifié par le sol sous-jacent, est illustré dans plusieurs régions de la province. La piètre performance de la maçonnerie non renforcée est également éxaminée et finalement le risque d'instabilité des pentes est démontré dans plusieurs cas.

Les conclusions de ce rapport s'adressent aux ingénieurs et aux agences gouvernementales en les avertissant de la présence, en grande quantité, de murs de maçonnerie non renforcée dans les hôpitaux et dans les écoles. Il éxiste, par conséquent, une possibilité réelle que ce type de construction fragile puisse s'éfondrer et causer des pertes de vie, dans l'avenir, lors d'un séisme de magnitude égale ou supérieure à ce dernier.

ACKNOWLEDGEMENTS

The authors wish to express their appreciation to many individuals who assisted the site-visit team. The site-visit was initiated by Professor S. M. Uzumeri, the Chairman of the Canadian National Committee on Earthquake Engineering and by Dr. M. J. Berry, of the Geological Survey of Canada.

Throughout the site-visit the team was assisted by police and fire personnel, industrial organizations, government organizations, school and hospital officials, and by local residents.

The site-visit team would like to thank the following individuals for the valuable information provided in preparing this report:

- Jean-Yves Chagnon, Université Laval, Québec
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- Alain Blouin, Solivar, Québec
- René Desabrais, Consolidated Bathurst, Ville de la Baie
- Louise Heppel, Shell Canada, Montréal
- Roger Boulianne, Ecole Polyvalente Dominique Racine, Chicoutimi
- M. Truchon and J.P. Côté, Polyvalente de la Baie, Ville de la Baie
- Gilles Thibeault, Cegerco-G.C.L., Chicoutimi
- Vincent Silvestri and Georges Sarkis, Ecole Polytechnique de Montréal
- Luc Tanguay, Ministère des Transports du Québec

In addition we would like to express our gratitude to Peter Basham, Philip Monro, Robert J. Wetmiller, Maurice Lamontagne and Anne Stevens of the Geological Survey of Canada for providing useful information on the ground motion.

CHAPTER 1

INTRODUCTION

1.1 Objectives of the Site-visit

On Friday, November 25, 1988 an earthquake of magnitude 6.0 occurred in the Saguenay region approximately 36 km south of Chicoutimi. The Earthquake was felt over an extremely large area, as far south as New York City and as far west as Toronto. Despite the magnitude of the earthquake there was no loss of life directly attributed to the event and no major structural damage was observed.

The Canadian National Committee on Earthquake Engineering, under the sponsorship of the National Research Council of Canada has participated in site–visits by sending representatives of the committee to earthquake damaged areas in several countries. Because this was the largest earthquake to occur in Canada in the last 50 years it was decided to investigate the damage caused by this event.

The site–visit team consisted of three members as given below:

- Dr. Denis Mitchell – structural engineer
- Dr. René Tinawi – structural engineer
- Dr. Tim Law – geotechnical engineer

The team arrived in Chicoutimi on November 27, 1988, less than two days after the earthquake, in order to ensure adequate reporting of the damage before repairs took place.

The objectives of the site-visit were to gather information by direct observations and discussions with engineers and administrators responsible for public facilities (e.g., hospitals, schools and emergency facilities) as well as those responsible for industrial facilities and private buildings.

The team spent three days in the Chicoutimi region and two days in the Quebec City region immediately after the earthquake. The team spent additional time investigating damage in the Montreal region. Other investigative work is still being carried out by the team related to industrial facilities, hydro–electric facilities and transportation facilities.

Figure 1.1 is a map of the province of Quebec showing the location of the epicentre. Figure 1.2 shows locations referred to in this report.

1.2 Preliminary Information on Ground Motion

It is noted that this report deals mainly with damage due to the earthquake. Additional information on the strong motion data is given in a separate report published by the Geological Survey of Canada (Monro and North 1988).

The magnitude (M_s)6.0 earthquake occurred at 18:46 Eastern Standard Time on Friday, November 25, 1988. The epicentre was located 36 km south of Chicoutimi close to the northern boundary of Parc Laurentides in the province of Quebec. The focal depth was 28 km. The event was preceded by a foreshock of magnitude 4.8 on Wednesday, November 23, 1988. An aftershock of magnitude 4.1 occurred on November 25, 1988 at 22:38 Eastern Standard Time. There were more than twenty aftershocks ranging in magnitude from 1.6 to 2.9 in the three weeks following the main event.

Figure 1.3 shows the Eastern Canada Strong Motion Seismograph Network of the Geological Survey of Canada. Table 1.1 summarizes the peak horizontal and vertical accelerations and frequencies obtained from this strong motion network. These strong motion sites were all founded on bedrock except for the station located in Baie-St-Paul. Note that peak acceleration values given in Table 1.1 were read from the strong motion accelerograph film recordings and may change slightly when the digitized and "instrument corrected" data is available. Also shown in Table 1.1 are peak acceleration values measured in Maine and New York by the Lamont-Doherty network (Friberg et al 1988).

1.3 References

Friberg, P., Busby, R., Lentrichia, D., Johnson, D. Jacob, K. and Simpson, D. 1988. The M=6 Chicoutimi Earthquake of November 25, 1988, in the Province of Quebec, Canada : Preliminary NCEER strong ground motion data report. Lamont-Doherty Geological Observatory of Columbia University, Palisades N.Y., 10964. 39 p.

Monro, Philip,S. and North, Robert,G. 1988. The Saguenay Earthquake of November 25, 1988–Strong Motion Data. Geological Survey of Canada. Open File Report Number 1976. 17 p.

Table 1.1 Peak Accelerations at different strong motion stations, adapted from Monro and North (1988) and Friberg et al (1988)

	Station	Distance (km)	Azimuth (°)	Vertical		Horizontal*	
				Acc. (% g)	Freq. (Hz)	Acc. (% g)	Freq. (Hz)
16	Chicoutimi-Nord, Qué.	43.2	17.6	7.2	18.2	10.9	13.3
17	St-André, Qué.	63.6	291.3	3.7	12.5	10.9	20.0
20	Les Éboulements, Qué.	90.4	134.2			9.1	6.4
7	Baie-St-Paul, Qué.	91.0	145.6	11.7	10.5	15.5	4.1
8	La Malbaie, Qué.	93.0	123.4	6.9	6.7	10.3	10.0
5	Tadoussac, Qué.	109.2	88.1	3.8	16.7	2.5	16.7
1	St-Ferréol, Qué.	113.8	166.1	5.4	11.8	8.7	8.3
10	Rivière-Ouelle, Qué.	114.4	128.3	1.8	7.7	4.1	8.3
9	St-Pascal, Qué.	122.7	122.1	3.4	3.3	5.6	6.9
2	Québec, Qué.	149.3	182.5	2.2	10.0	5.0	10.5
14	Ste-Lucie-de-Beauregard, Qué.	176.8	149.6	1.0	4.0	1.4	4.0
	Dickey, Me.	198.3	125.3	2.35		9.10	
	Island Falls, Me.	322.5	135.7	0.40		0.55	
	Milo, Me.	359.2	152.1	0.37		0.45	
	Lyon Mt., N.Y.	431.1	209.9	0.40		0.36	
	Massena, N.Y.	445.6	220.2	0.26		0.69	
	Machias, Me.	470.8	141.6	0.18		0.22	
	Newcomb, N.Y.	524.2	209.5	0.17		0.28	
	Palisades, N.Y.	819.9	196.2	0.04		0.035	

* largest of the two horizontal components

* plus grandes des deux composantes horizontales

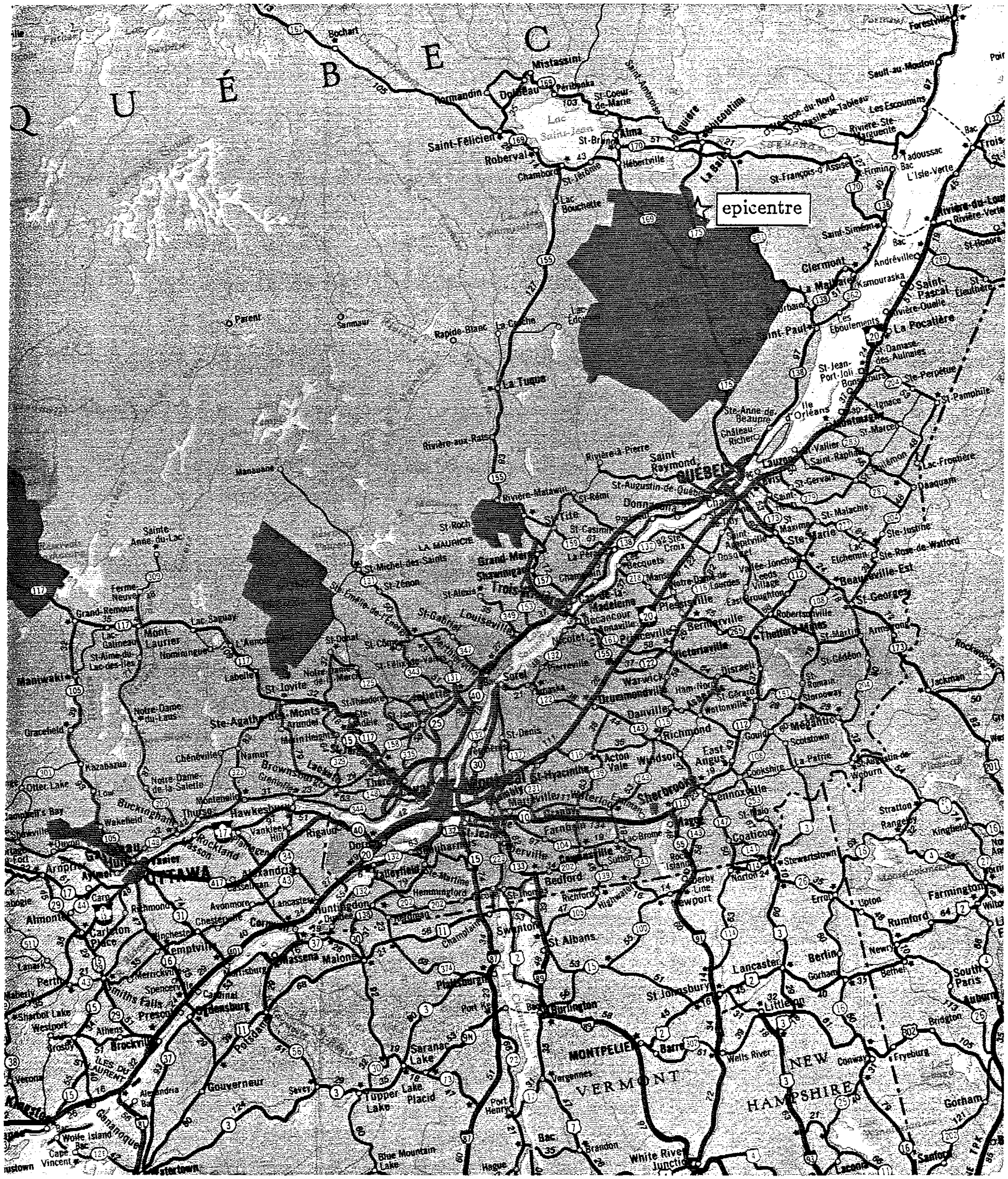


Figure 1.1 Location of the epicentre.

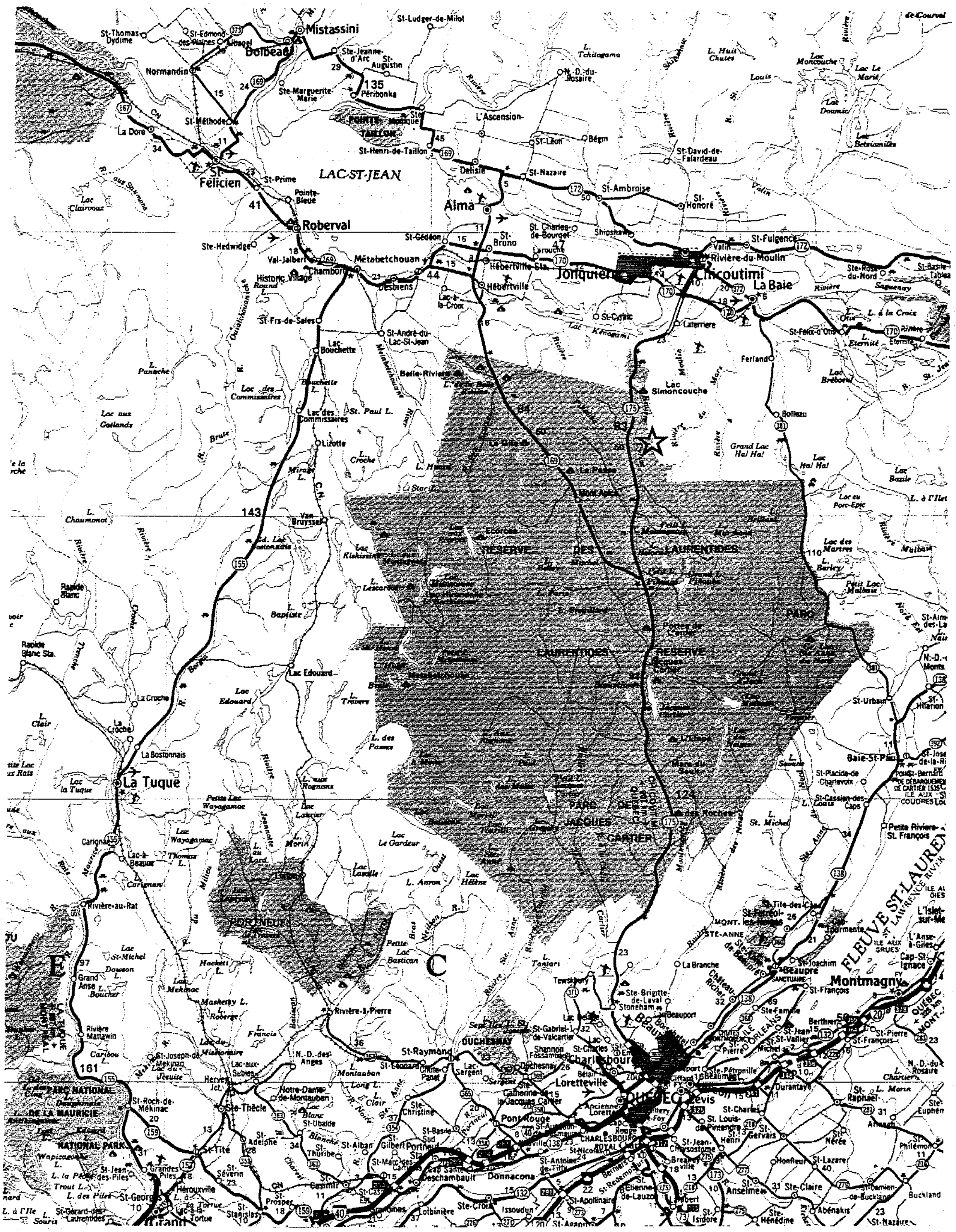
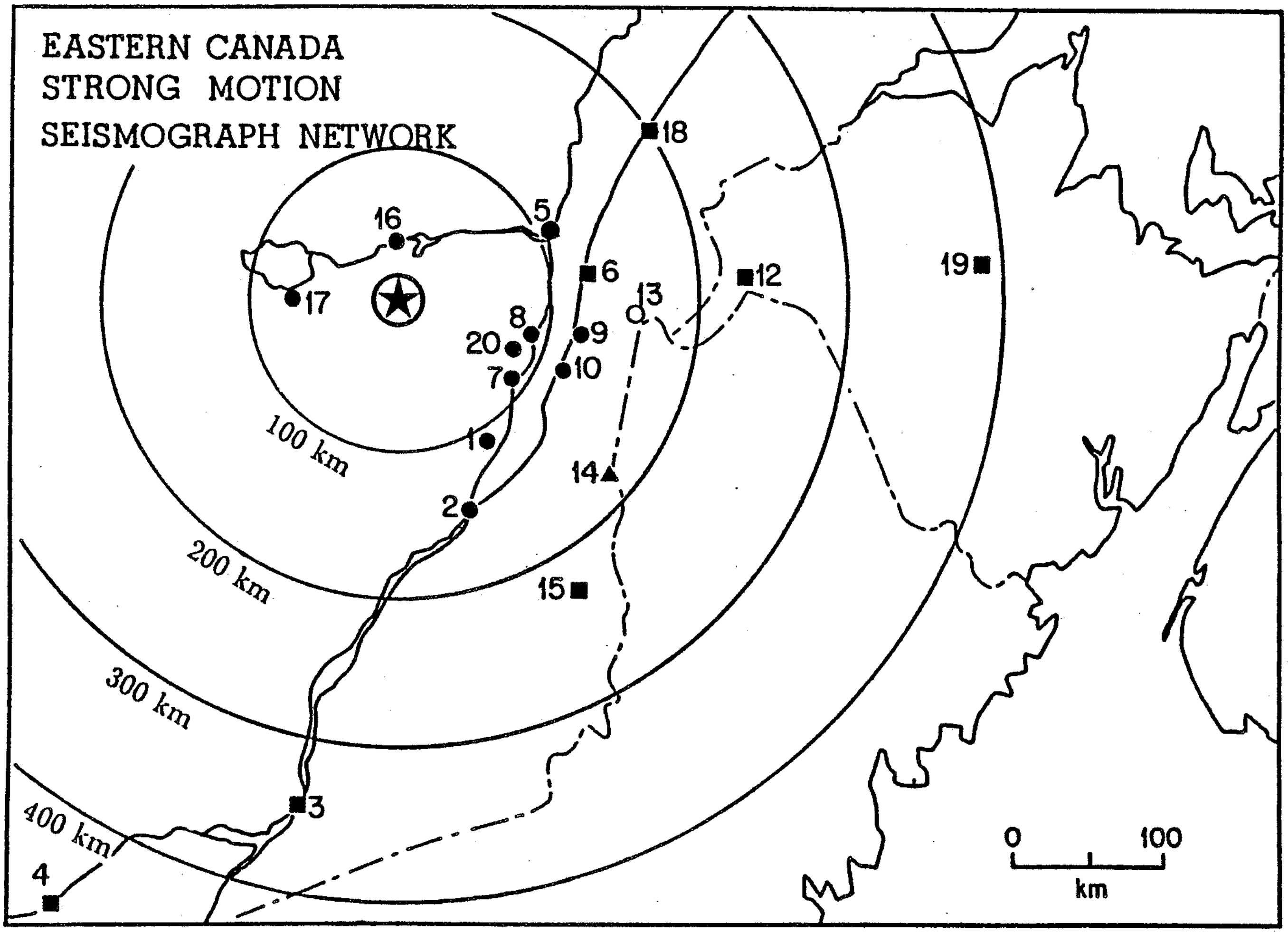


Figure 1.2 Locations visited by the site–visit team.



Key

- ● P-wave trigger / déclenchement par l'onde P
- ○ No record / pas d'enregistrement
- ▲ S-wave trigger / déclenchement par l'onde S
- ■ Untriggered / non-déclenché

Figure 1.3 Eastern Canada Strong Motion Seismograph Network, Courtesy of the Geological Survey of Canada (Monro and North 1988) .

CHAPTER 2

DAMAGE IN CHICOUTIMI

2.1 Description of Chicoutimi

Chicoutimi, with a population of about 60,000, is located about 36 km north of the epicentre. As can be seen from Fig. 2.1 Chicoutimi is located in a region of rolling hills on the south shore of the Saguenay River (see Fig. 1.1). Figure 2.2 gives the locations of all of the buildings visited in Chicoutimi during the site-visit.

2.2 Place du Royaume Shopping Plaza

Place du Royaume is an indoor shopping plaza located on Boulevard Talbot. This one-storey structure has a structural system of light structural steel framing with infilled unreinforced masonry consisting of hollow concrete blocks. Figures 2.3 and 2.4 illustrates minor architectural damage in the Bay store, due to fallen ceiling tiles. As can be seen from Fig. 2.4 the ceiling tiles rested on standard inverted-tee supports which in turn were hung from the open-web steel joists of the roof framing by vertical hangers only. It is believed that the relatively long spans together with the lack of inclined hangers for the tee-bar supports resulted in differential movements of the very flexible tee-bars.

Figure 2.5 illustrates damage to the ceiling tile due to fallen masonry blocks from the topmost layer of the walls. Figure 2.6 shows an unreinforced masonry wall in which some of the remaining blocks of the top layer have been removed after the earthquake.

Similar problems were encountered in the Kébec Disque store of the same shopping plaza. Figure 2.7 shows damage to the ceiling tile due to fallen masonry blocks and Fig. 2.8 shows a lighting fixture that had fallen in the storage room due to inadequate support.

2.3 Ecole Polyvalente Dominique Racine (Secondary School)

An overall view of the "Ecole Polyvalente Dominique Racine" secondary school at 985 rue Bégin is shown in Fig. 2.9. The structural system of the original school (in foreground of Fig. 2.9) consists of reinforced concrete frames with unreinforced masonry infilled walls. The new expansion (see Fig. 2.9) consists of steel framing. Damage to the masonry concrete block walls in the original wing varied from severe shear cracks, that were in the process of being "repaired" (see Fig. 2.10) to collapse of portions of walls (see Fig. 2.11 and 2.12). All of the masonry walls in the original wing of the school were of "stack block" construction. Nearly all of the masonry walls in the classrooms, corridors and stairwells were cracked with more severe cracking in the walls in the "weaker" direction of the structure. An expansion joint in the original concrete frame structure, which was originally 1/2 in. had opened to a 1 in. gap as shown in Fig. 2.13.

Figures 2.14 and 2.15 show the top row of unreinforced masonry blocks which had been placed standing on end. Some of these blocks had fallen through the ceiling during the earthquake. As can be seen from Fig. 2.16 an inspection carried out by an engineer after the earthquake, enabled the identification of masonry blocks that had to be removed for safety reasons.

Figure 2.17 shows severe damage to the ceiling above the corridor and the seating area of the swimming pool located in the new wing. It is apparent from Fig. 2.18 that the damage to the ceiling was caused by falling masonry blocks which occurred over a length of 16 feet. Figures 2.19 and 2.20 show that the five top rows of 4 in. x 4 in. x 16 in. long hollow blocks had fallen. The unreinforced masonry was of "stack- block" construction and had no visible connection to the structural framing.

2.4 Pavillon Sagamie

The Sagamie pavilion (shown in Fig. 2.21), located at 930 rue Jacques Cartier Est, formerly part of the Université du Québec à Chicoutimi, presently houses a number of private and provincial government agencies such as Les Affaires Culturelles, Agriculture Pêcheries et Alimentation, Office du Crédit Agricole and Les Archives Nationales.

As can be seen from Figure 2.22 the pavilion consists of two separate wings. The older wing (on the right of the photograph) is a five storey masonry building with pile foundations that was constructed in 1932. The new wing is an eight storey reinforced concrete frame structure with masonry cladding constructed in 1962. The subsoil consists of clay 30 to 40 feet deep and the new wing is supported on a foundation mat.

Figure 2.23 shows the corridors connecting the two different wings. The stiff axis of the new wing is perpendicular to the stiff axis of the older wing. This, together with different foundation systems as well as different heights of the wings, contributed to minor damage in the interconnecting corridor along the joint between the corridor and the older wing. The interior masonry wall was also cracked on the inside of the joint as shown in Fig. 2.24. The stairwell shown on the right of Fig. 2.23 also had cracking in all of its plaster walls.

The south end of the newer wing had a staircase with a glass curtain wall as can be seen on the left side of Fig. 2.22. As can be seen from Fig. 2.25 and 2.26 there was considerable damage to the masonry wall perpendicular to the glass curtain wall resulting in a 4 in. separation and loss of ceiling tiles. This wall failed in flexure about its weak axis as the plane of the wall was perpendicular to the earthquake direction. Due to lack of connection between the walls, as shown in Fig. 2.26, there was no lateral support for that wall. Figure 2.27 shows the flexural failure crack with a major crack extending from the ceiling through the blocks and through the opening in the wall created by the electrical box.

2.5 Christ-Roi Church

Christ-Roi Church is located on 366 Avenue, St-Anne, close to the Saguenay River. Figure 2.28 shows an overall view of the church and its nine-storey bell tower constructed in the 1950's. The earthquake occurred during mass. The falling of 65 crystal prisms from the chandeliers (see Fig. 2.29) together with the falling of decorative marble panels inside the nave of the church (see Fig. 2.30) contributed to the general panic of the congregation. Figure 2.31 shows the bell tower which experienced cracking in the masonry at the top of the windows (see Fig. 2.32).

The structural system of the tower is made up of a steel frame with tension only steel cross-bracing with masonry brick walls on the outside and hollow unreinforced masonry blocks on the inside (see Fig. 2.33). The steel frame consisted of double 8 in. x 8 in. I-shaped columns in the corners of the structure and two 6 in. x 6 in. columns on each side between the windows. The cross bracing consisted of steel plates which varied from 4 in. x 5/16 in. thick near the base of tower to 3 in. x 5/16 in. at the top. Figure 2.34 shows the significant buckling of a tension cross-braced member with approximately 1 in. of residual out-of-plane displacement at the centre of the brace.

It was interesting to note that on the 6th floor a cross brace member was omitted during construction as shown in Fig. 2.35. This missing brace probably contributed to the damage.

Each floor of the tower, which is 14 ft. x 14 ft. in plan, is supported by two steel I-Beams in one direction and is reinforced by two diagonal steel tension braces in order to provide diaphragm resistance as shown in Fig. 2.36. Figure 2.37 shows over 4 in. of differential vertical movement between the two horizontal diagonal members. This example clearly illustrates the inadequacy of tension-only cross bracing for earthquake design.

2.6 Geotechnical Observations

Several cases of settlement of fill supporting house foundations were reported by residents in the Chicoutimi area. On the other hand, well compacted fills and natural soil deposits in Chicoutimi have not suffered any damage during this earthquake. No evidence of movement or distress could be found in a well compacted sand fill approach embankment (see Fig. 2.38) being constructed near Boulevard Talbot. In addition, no signs of movement or soil distress were observed in the reinforced-earth retaining wall shown in

Fig. 2.39. The generally small geotechnical damage in Chicoutimi, despite the proximity of the epicentre, is probably due to the bedrock being very near the ground surface. There are generally very few thick layers of soft surface soils to amplify the ground motion.

Several small boulders had rolled down a slope on Route 172 on the north shore of the Saguenay River close to Ste. Rose du Nord. On the other hand a sand pit on Route 172 was inspected and no sign of movement was observed for this pit which had steep slopes of about 4 m in height.

Figure 2.40 shows a natural slope landslide in Parc des Laurentides just east of Route 175 close to the epicentral region.

Figure 2.1 View of Chicoutimi on the Saguenay River showing the two bridges crossing the river, the rolling hills on the north shore of the Saguenay and Christ-Roi Church with its nine storey bell tower.

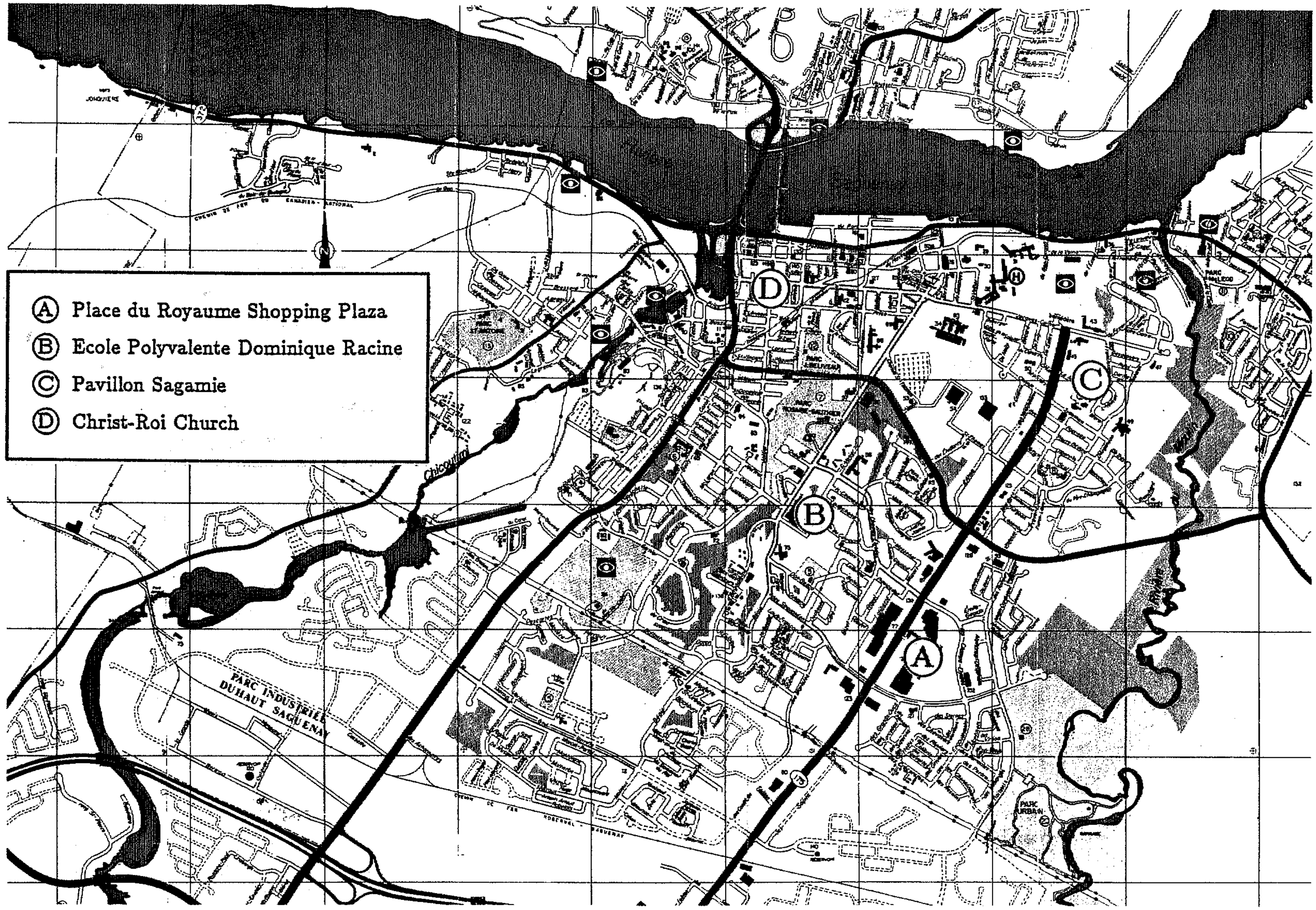


Figure 2.2 Map of the City of Chicoutimi showing locations of buildings that were inspected.

Figure 2.3 Damage to ceiling tiles in The Bay Store at Place du Royaume Shopping Plaza.

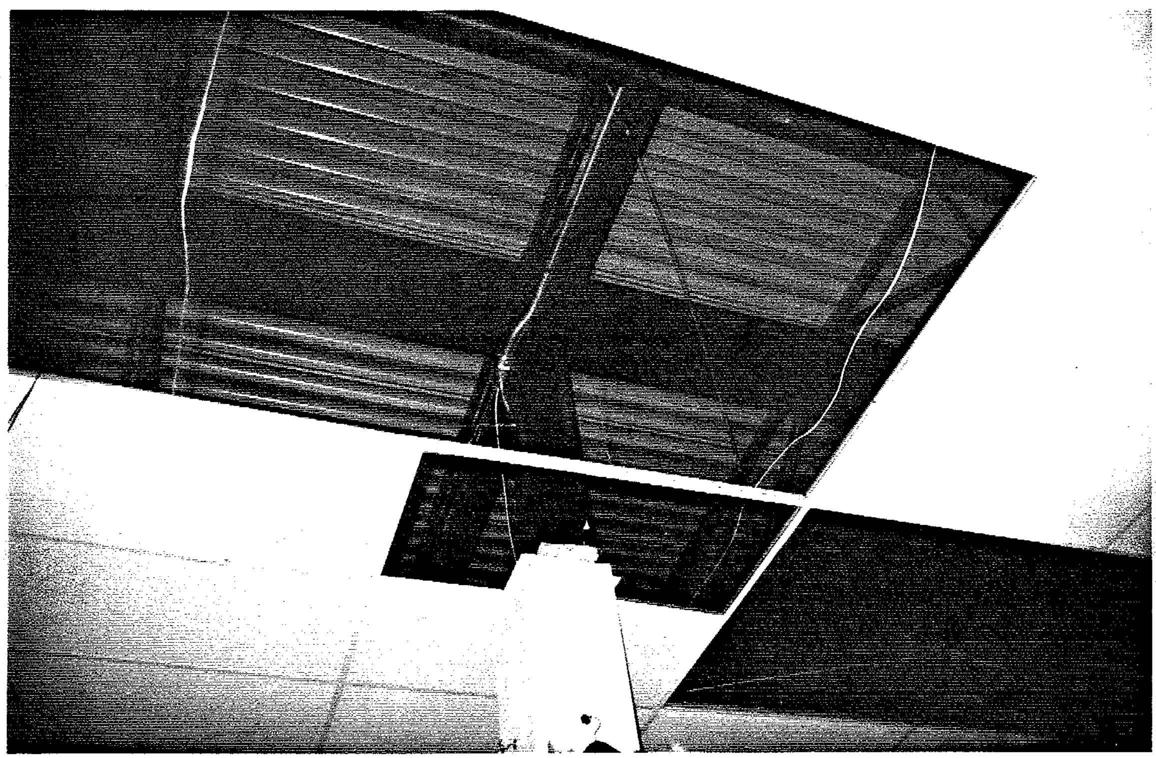


Figure 2.4 Close-up of supports for ceiling tiles.

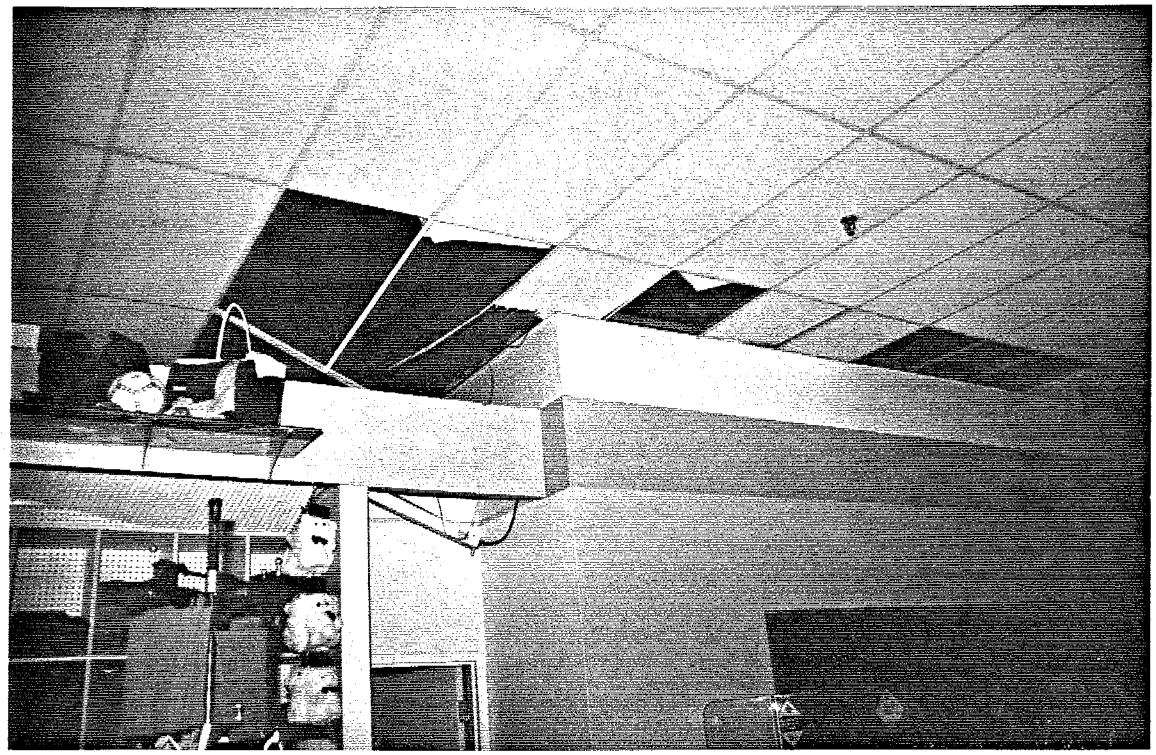


Figure 2.5 Damaged ceiling tiles due to fallen masonry blocks in The Bay store.

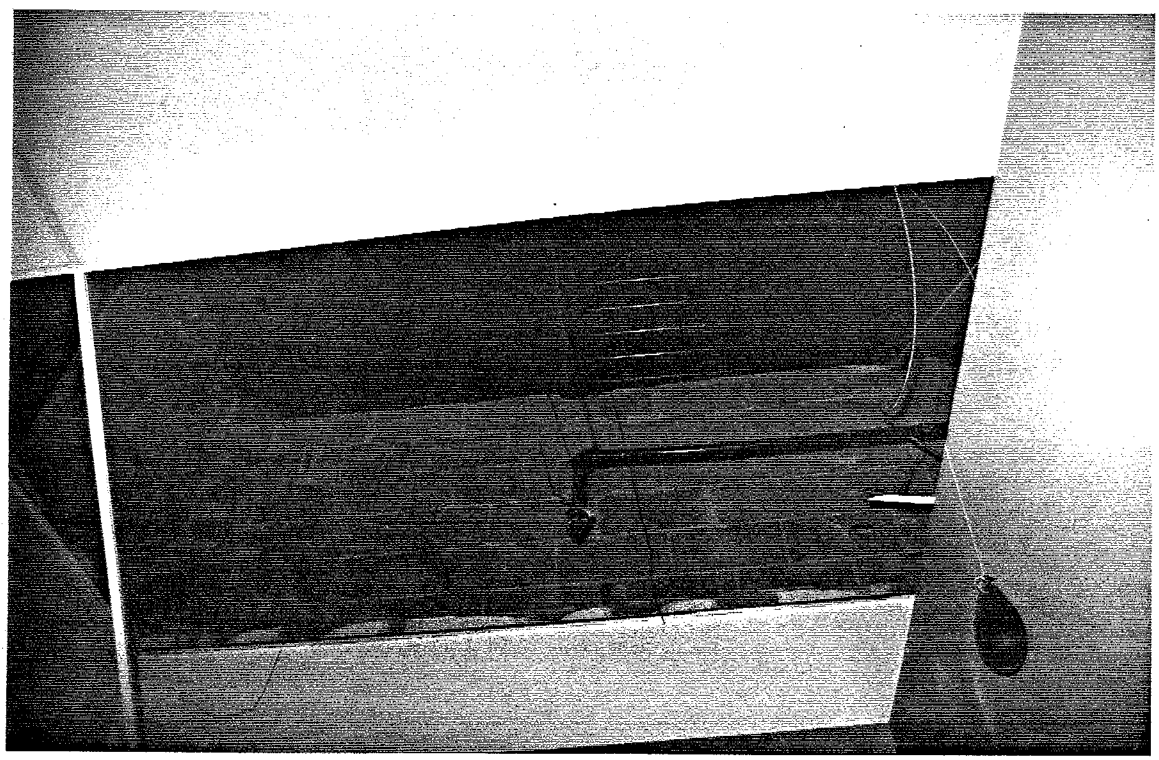


Figure 2.6 Masonry blocks with the top layer removed after the earthquake.

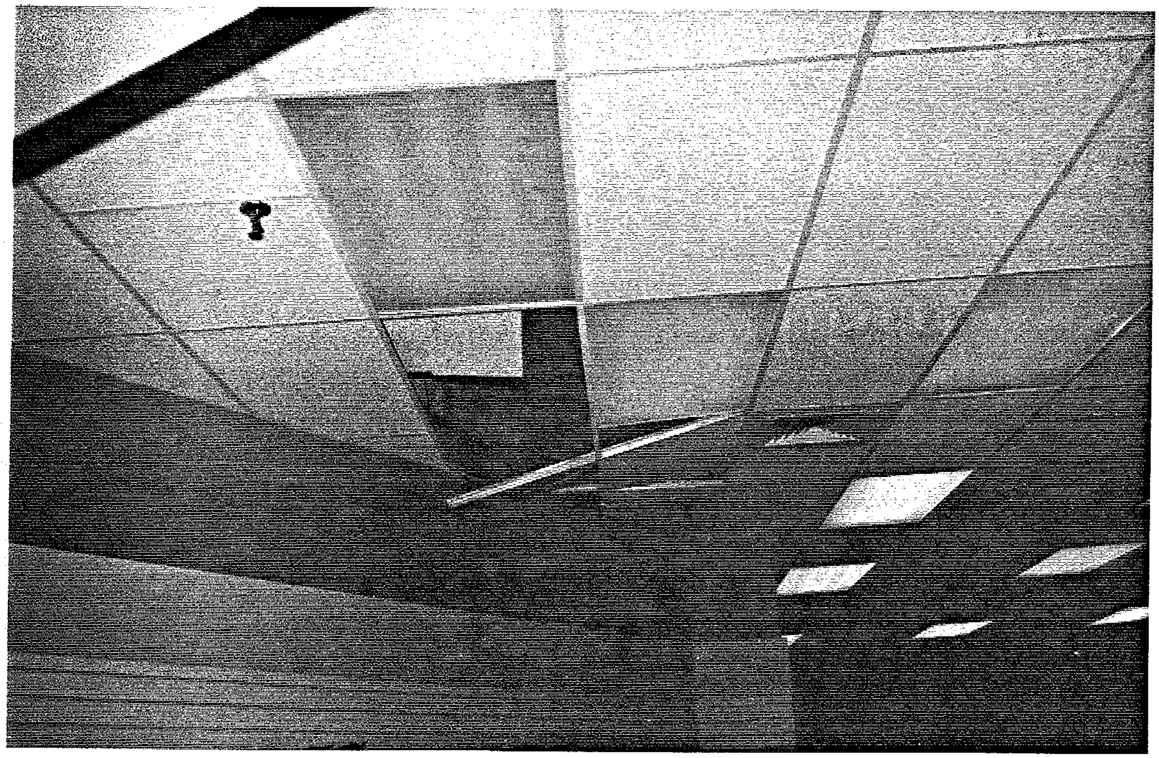


Figure 2.7 Damaged ceiling tiles due to fallen masonry blocks at Kébec Disque store in Place du Royaume Shopping Plaza.

Figure 2.8 Damage to light fixture in Kébec Disque store due to loss of support.

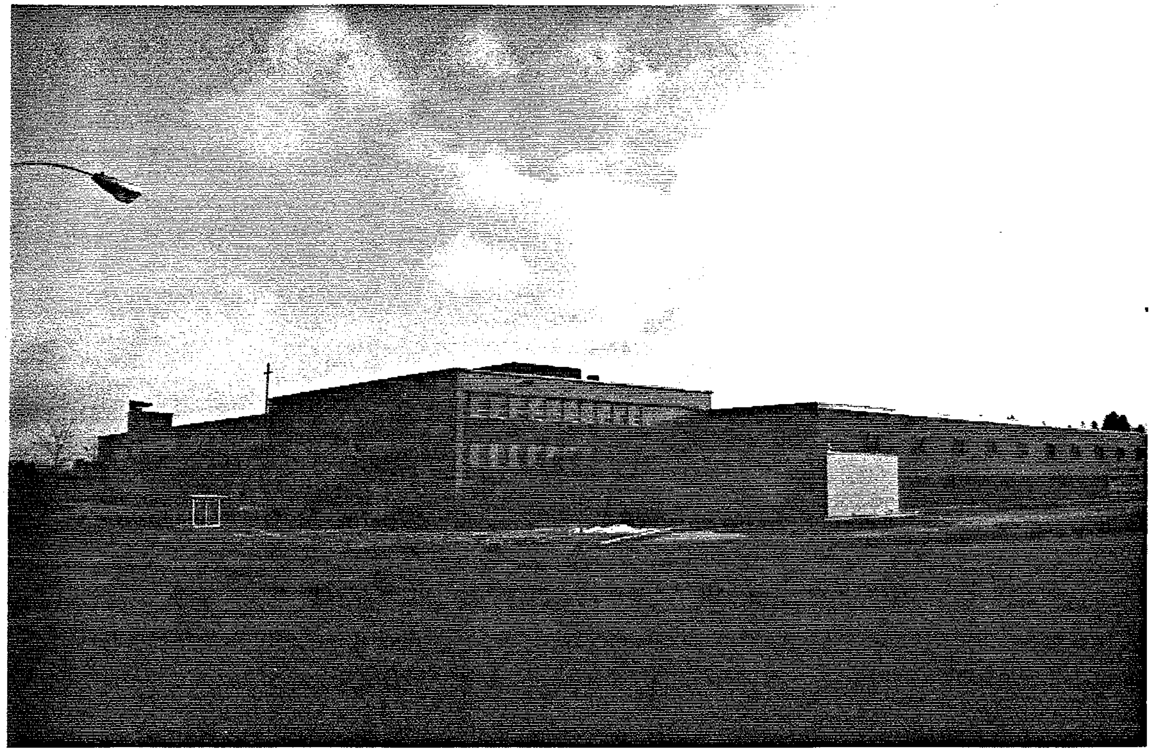


Figure 2.9 Ecole Polyvalente Dominique Racine in Chicoutimi.

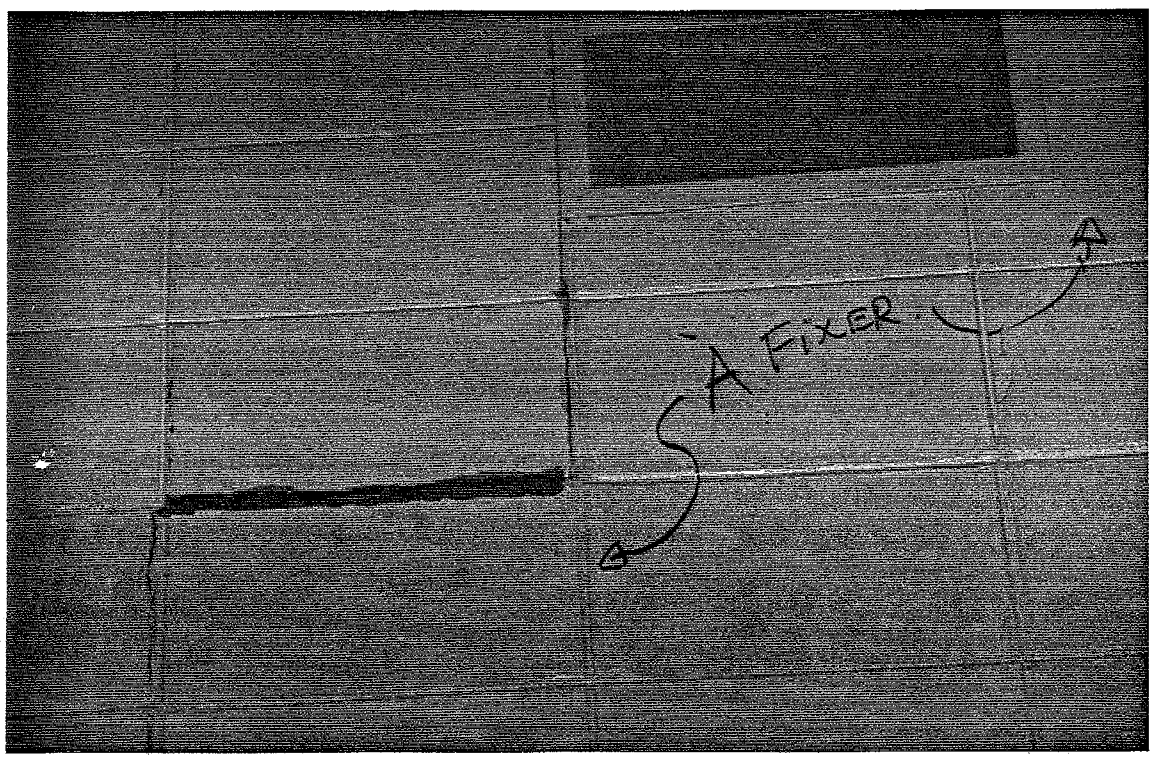


Figure 2.10 Cracking of joints in masonry blocks in Dominique Racine school.

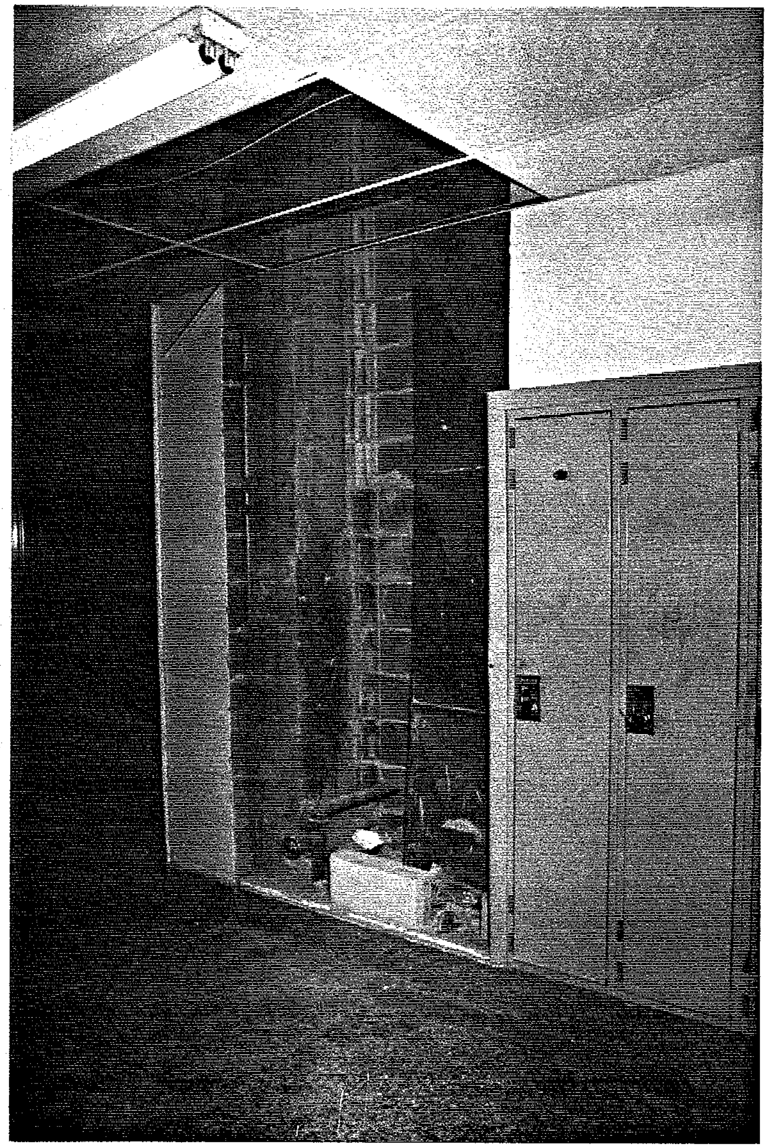


Figure 2.11 Collapse of masonry blocks in corridor of Dominique Racine school.

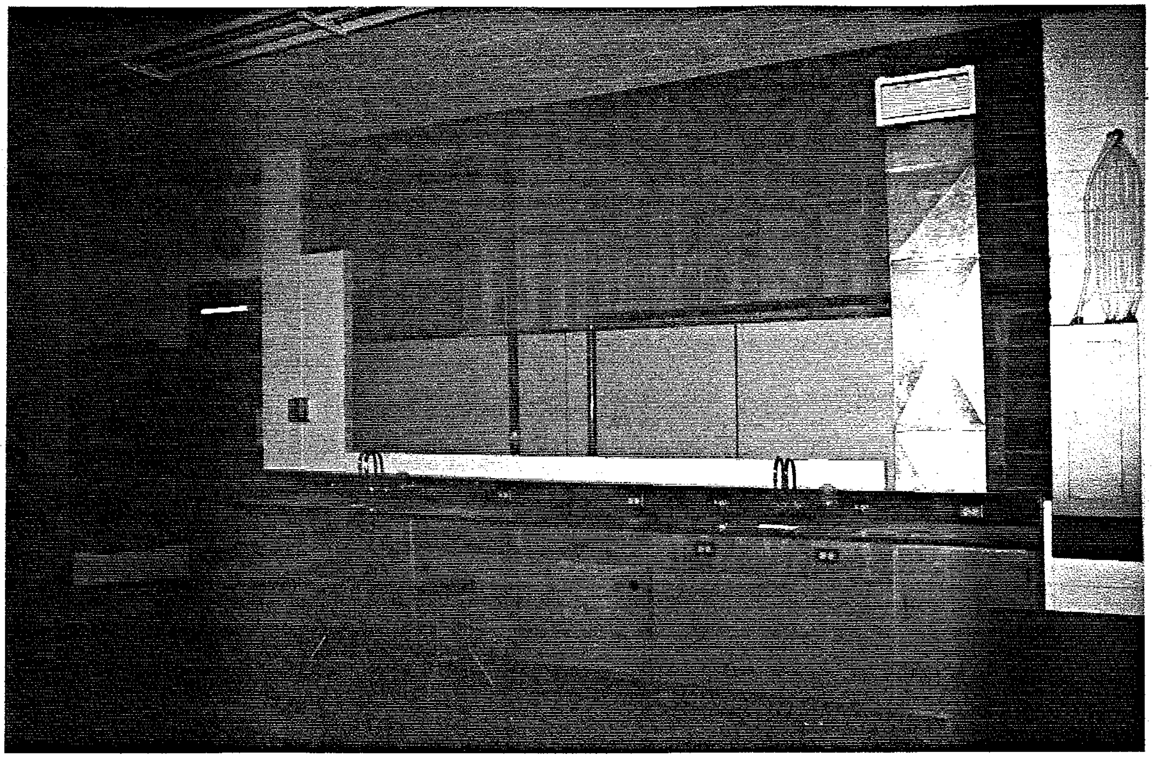


Figure 2.12 Collapse of masonry block wall in Physics laboratory of Dominique Racine school.

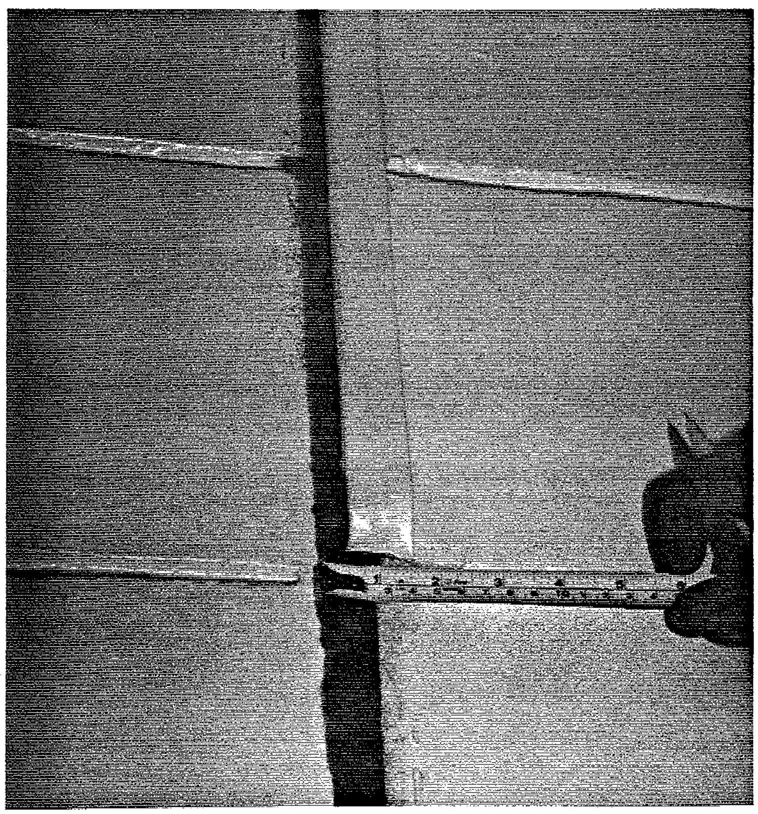


Figure 2.13 Expansion joint movement in Dominique Racine school.

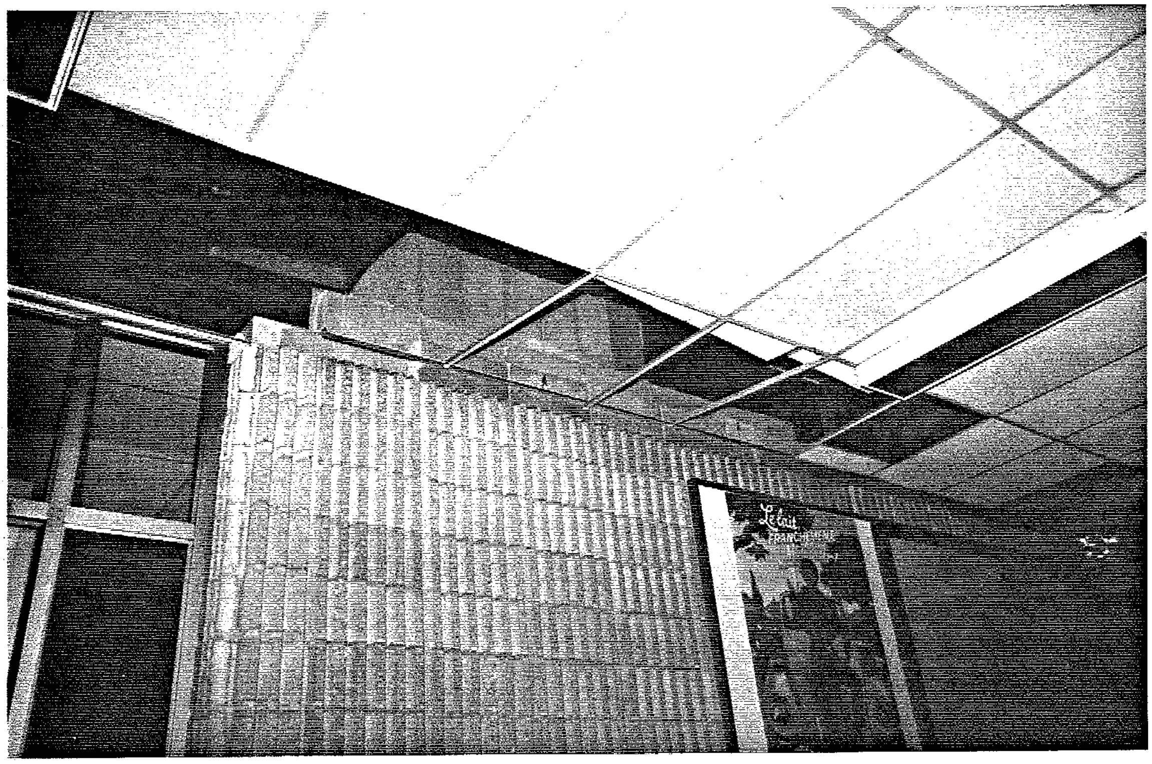


Figure 2.14 Masonry blocks standing on end in Dominique Racine school.

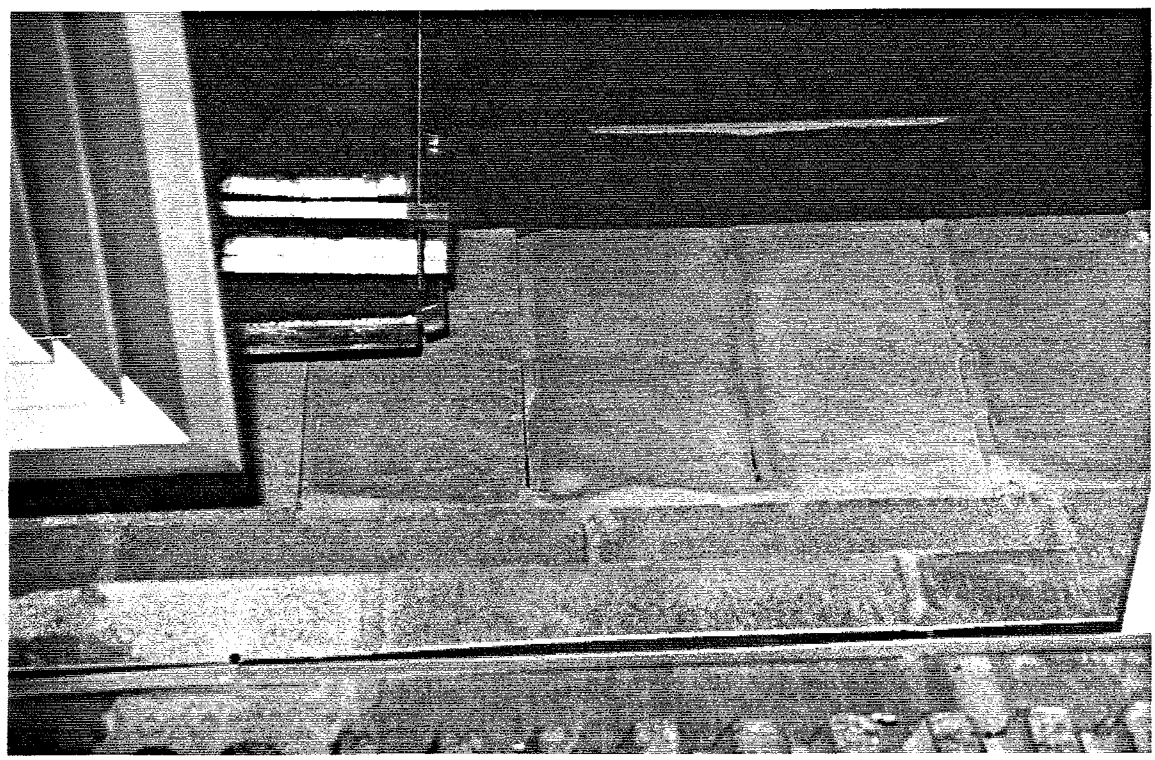


Figure 2.15 Close-up of masonry blocks standing on end in Dominique Racine school.

Figure 2.16 Identification of blocks to be removed for safety.

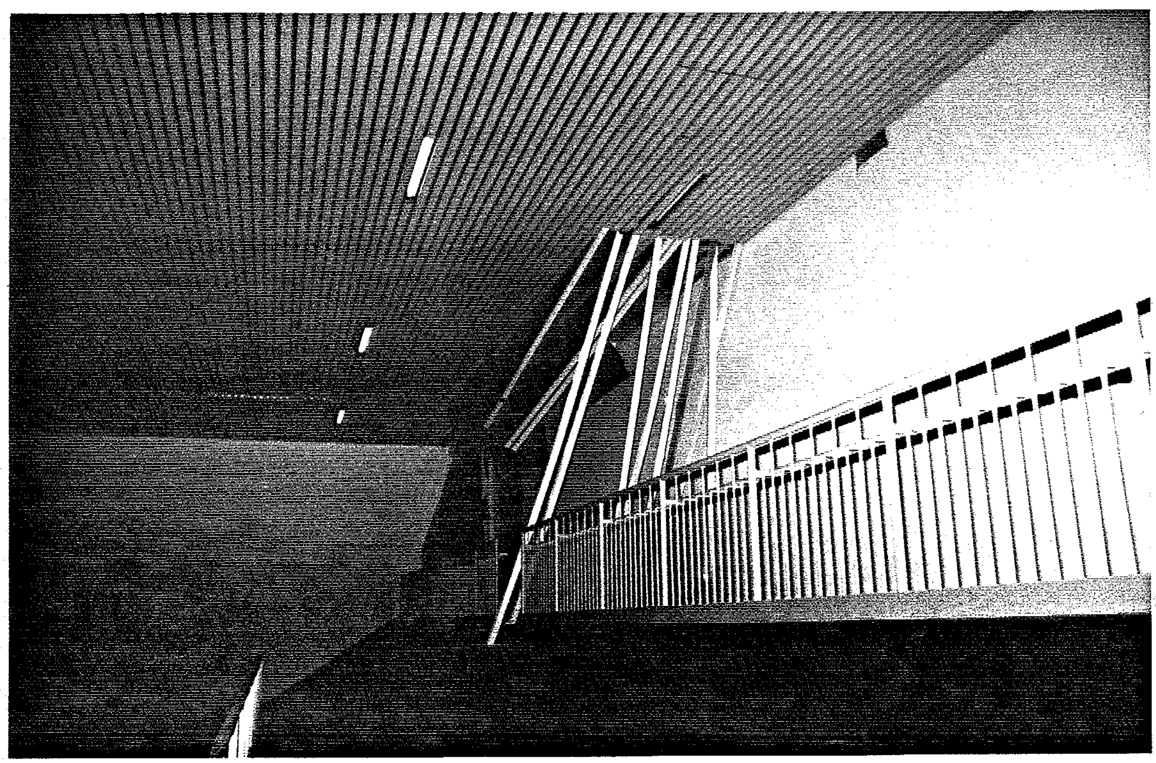


Figure 2.17 Damage to ceiling Dominique Racine school.

Figure 2.18 Debris from fallen masonry blocks in Dominique Racine school.

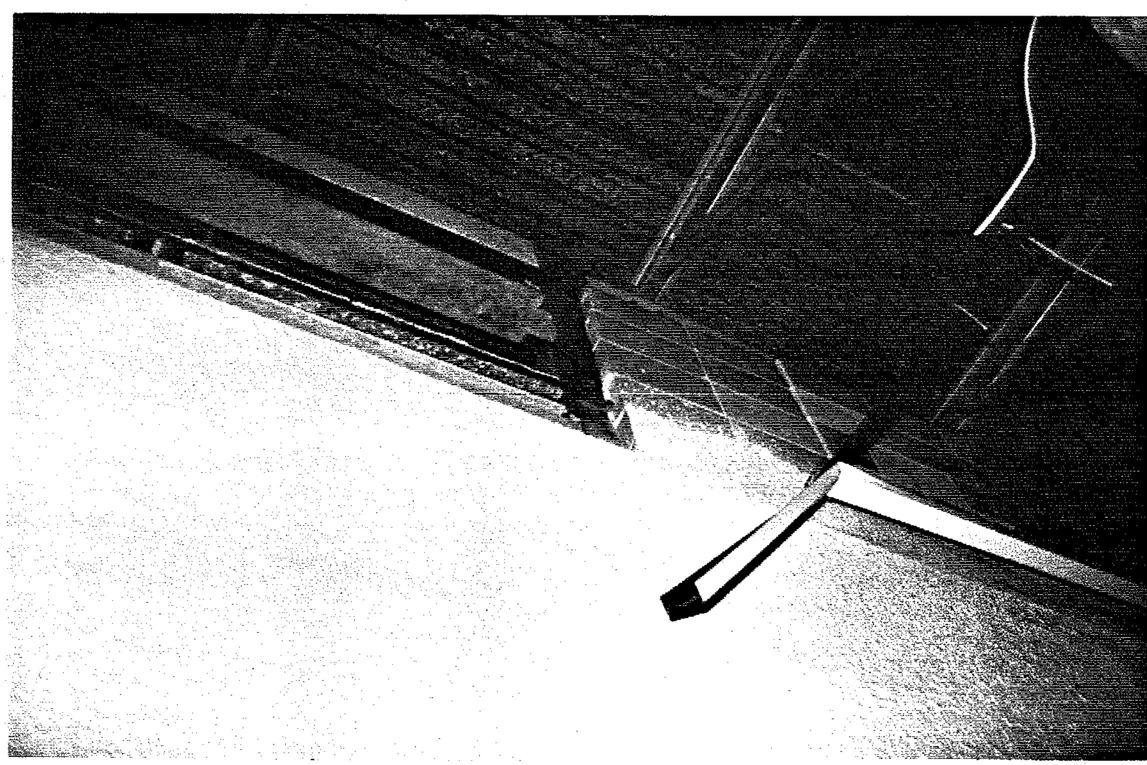


Figure 2.19 Missing masonry blocks at top of wall.

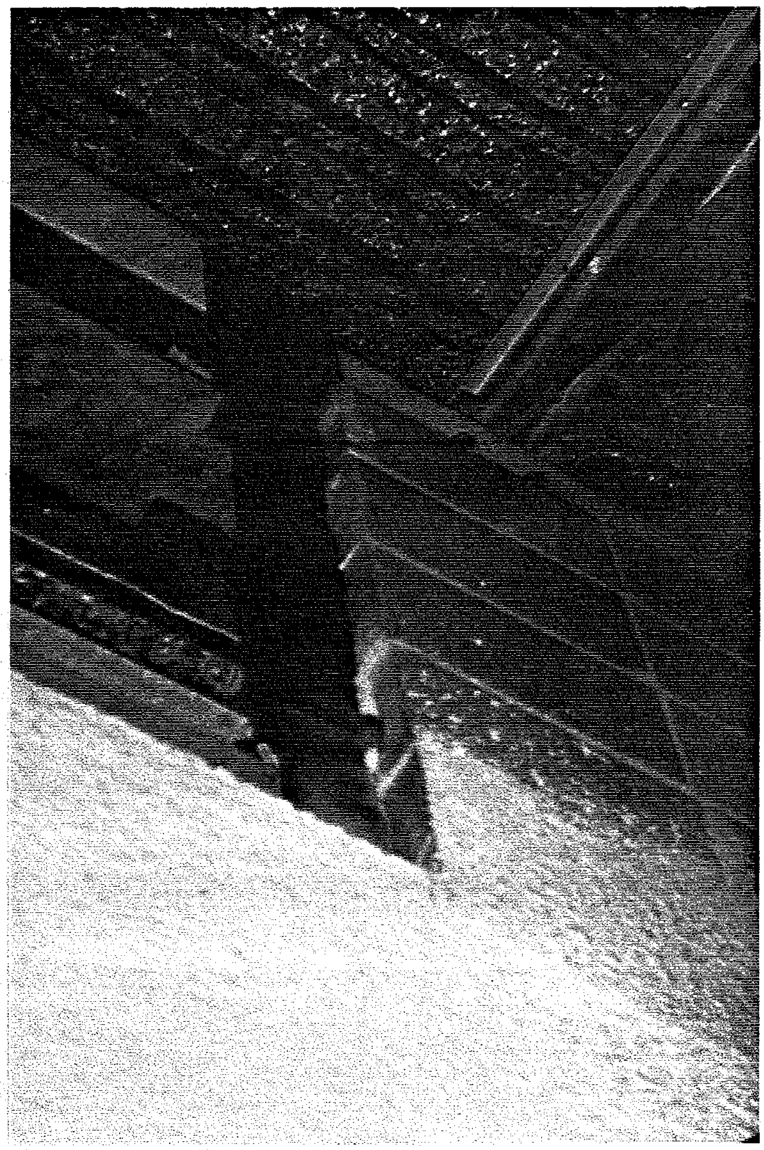


Figure 2.20 Detail of masonry blocks with no apparent connection to the steel frame.

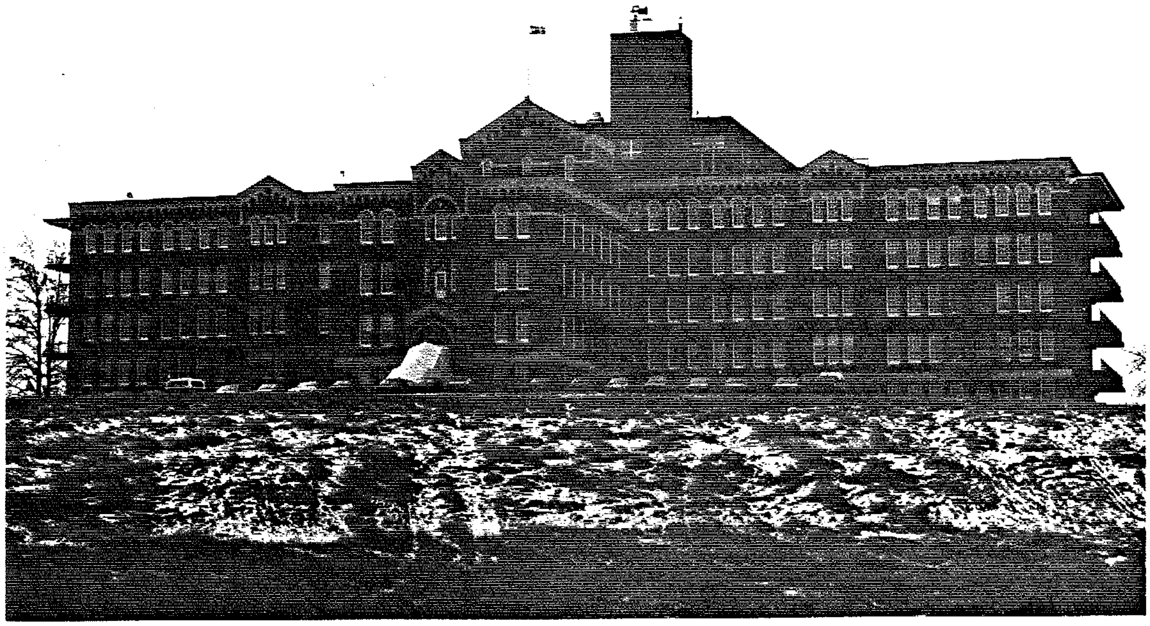


Figure 2.21 Front view of the old wing of Pavillon Sagamie.

Figure 2.22 New and old wings of Pavillon Sagamie.

Figure 2.23 Interconnecting corridor between new and old wings of Pavillon Sagamie.

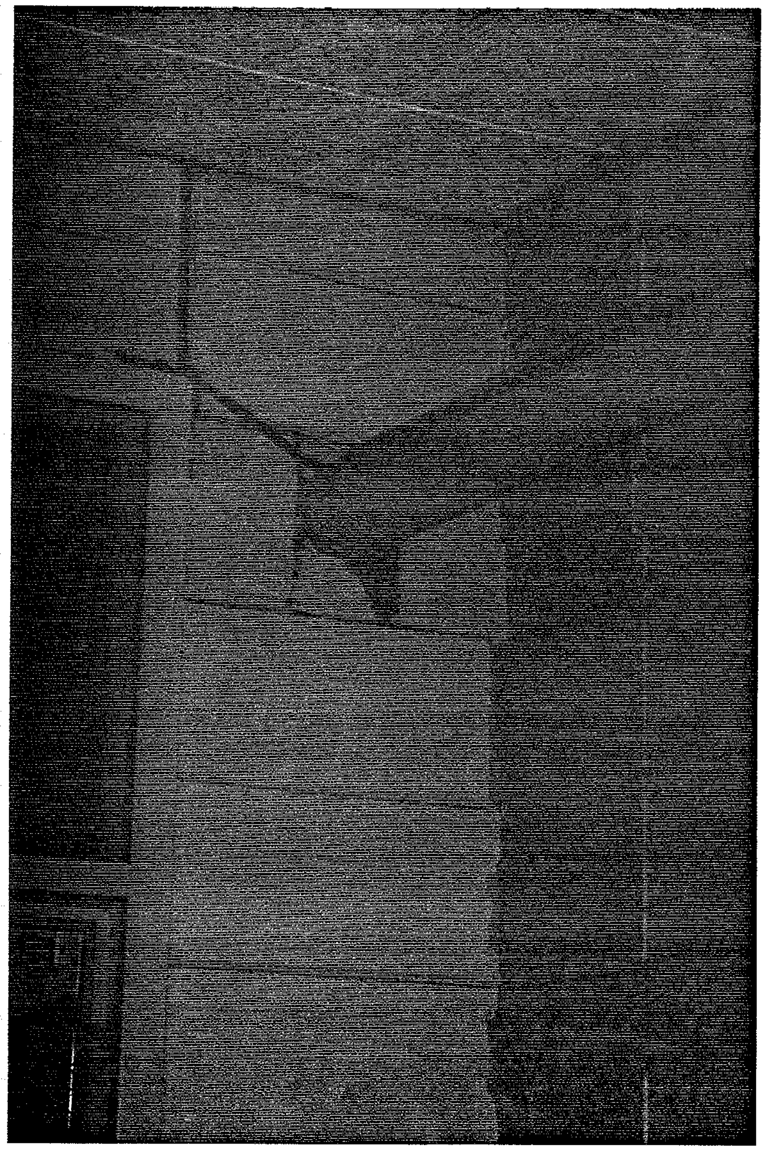


Figure 2.24 Cracking of interior masonry walls at joint between interconnecting corridor and older wing.

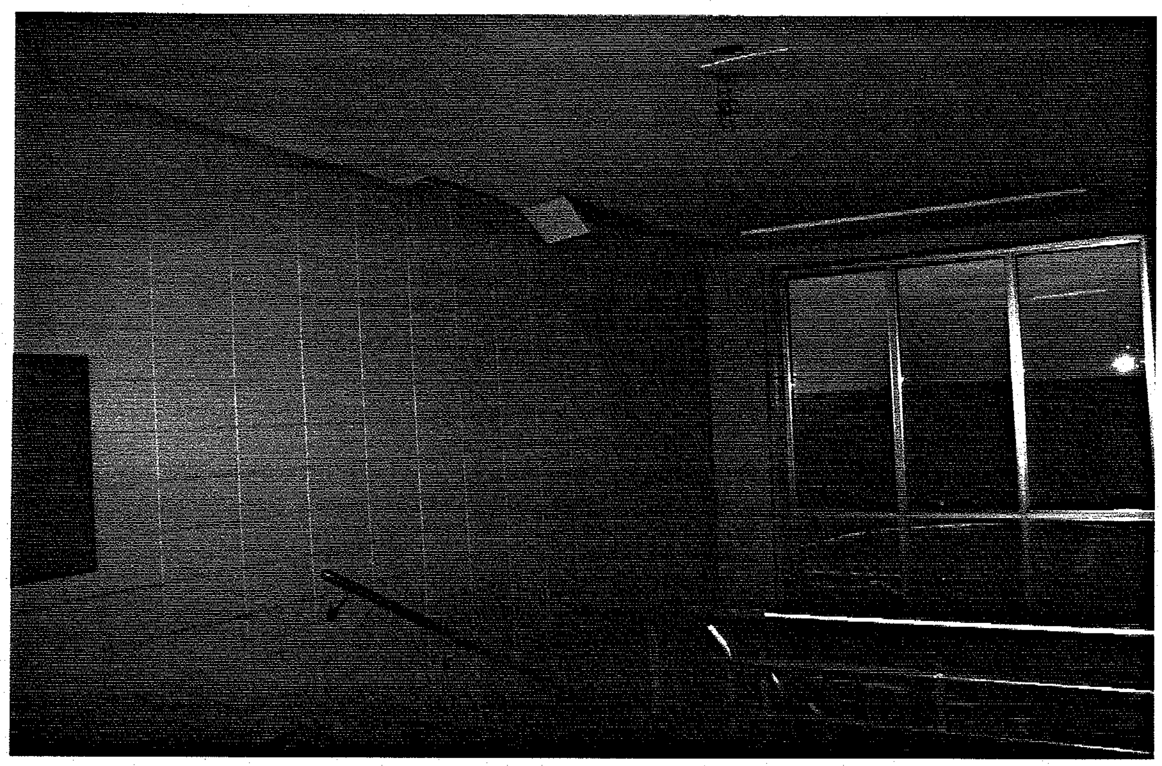


Figure 2.25 Failure of masonry wall in stairwell of Pavillon Sagamie .

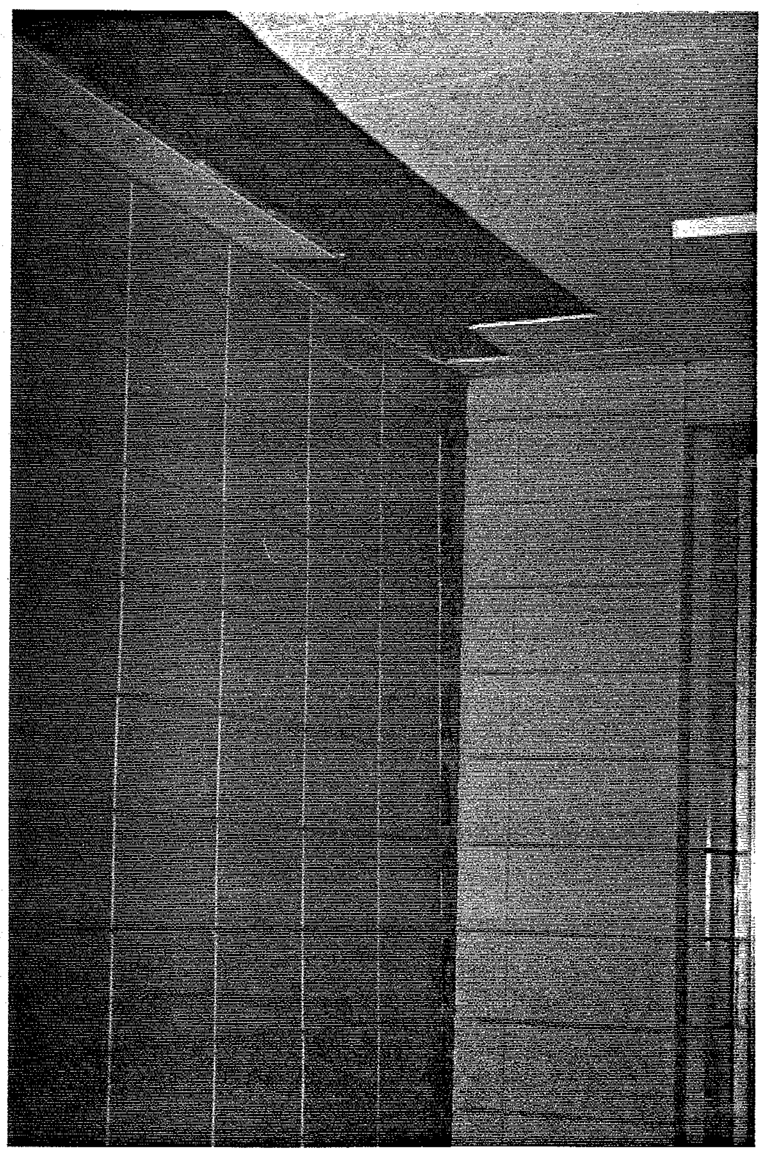


Figure 2.26 Four inch separation of walls and damage to ceiling tiles.

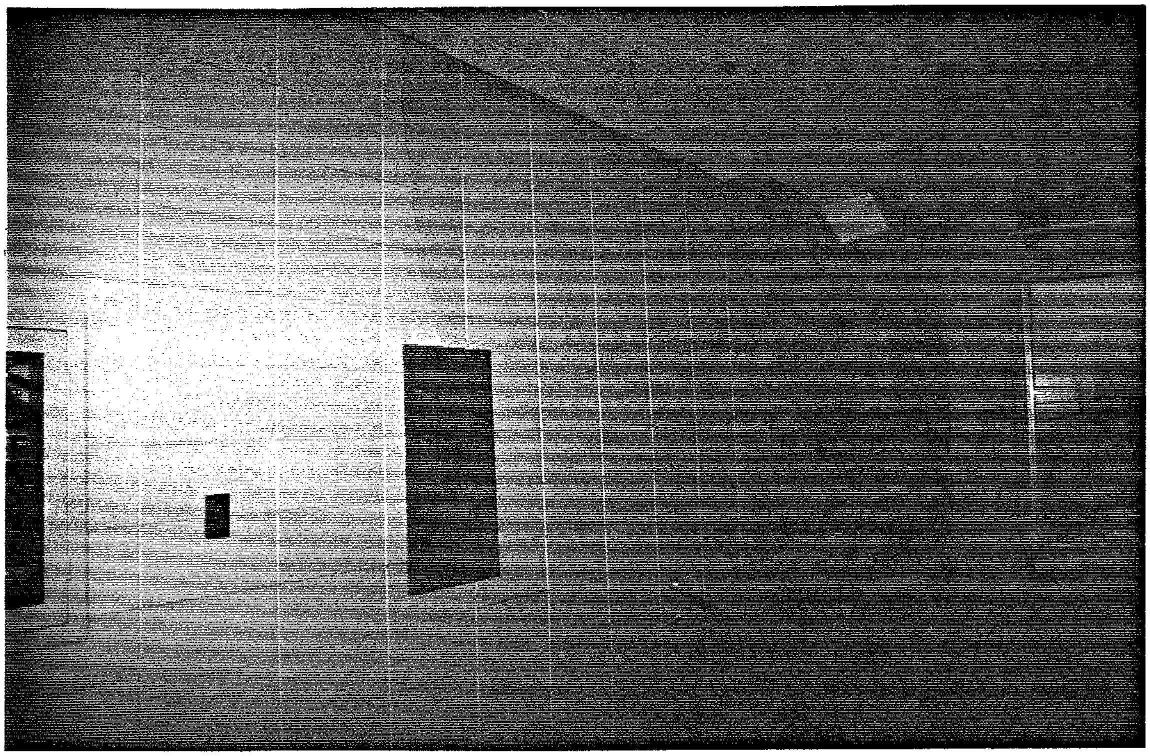


Figure 2.27 Flexural failure crack in masonry wall inside stairwell of Pavillon Sagamie.

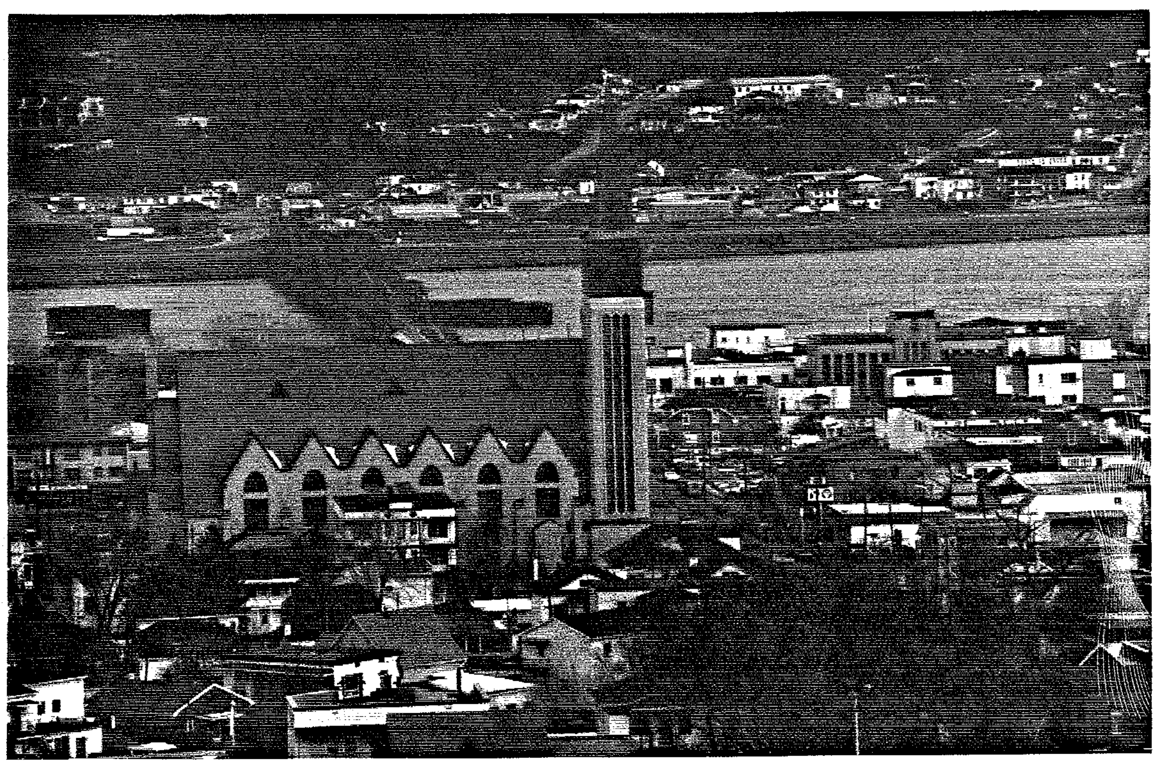


Figure 2.28 Overall view of Christ-Roi Church.

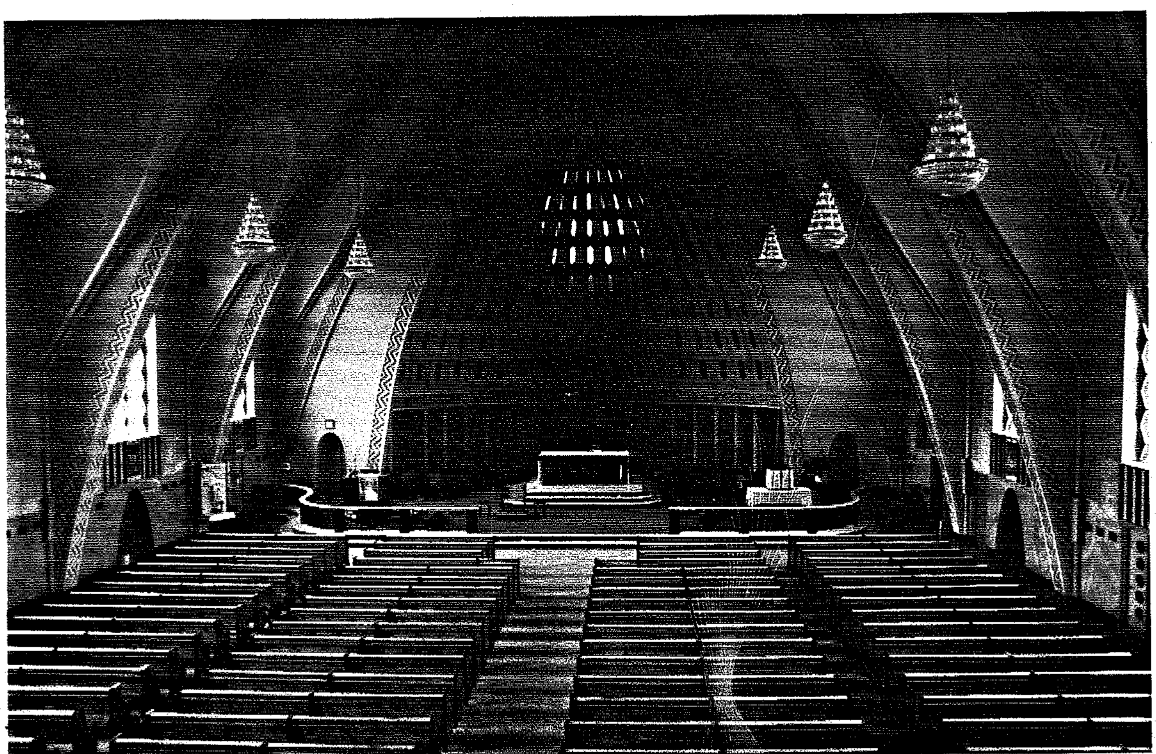


Figure 2.29 The nave of Christ-Roi Church.

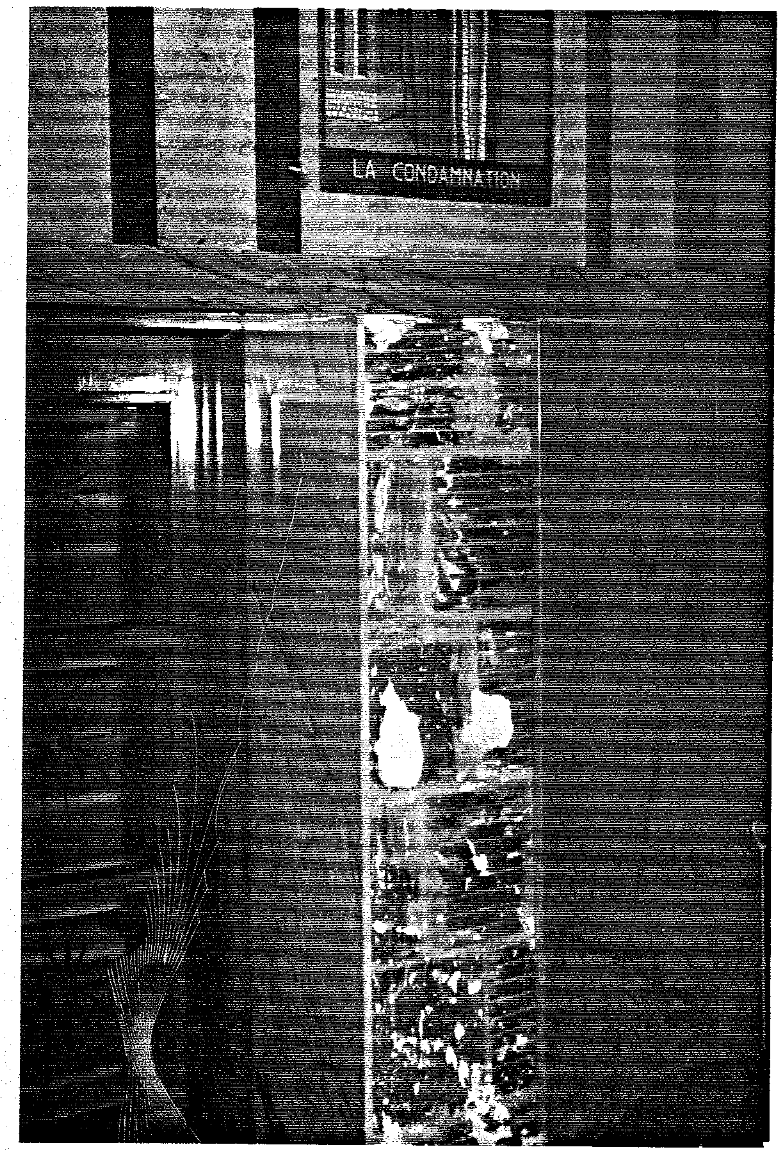


Figure 2.30 Detachment of marble panel located in the nave.

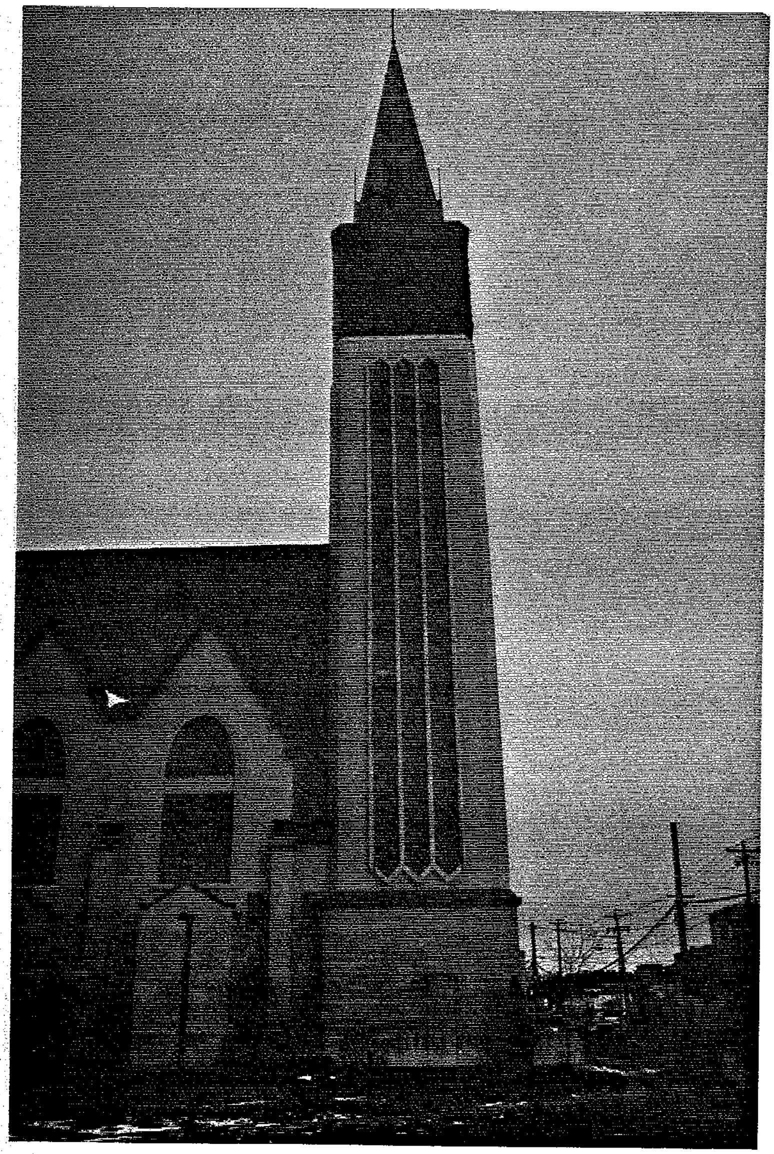


Figure 2.31 Nine-storey bell tower of Christ-Roi Church.

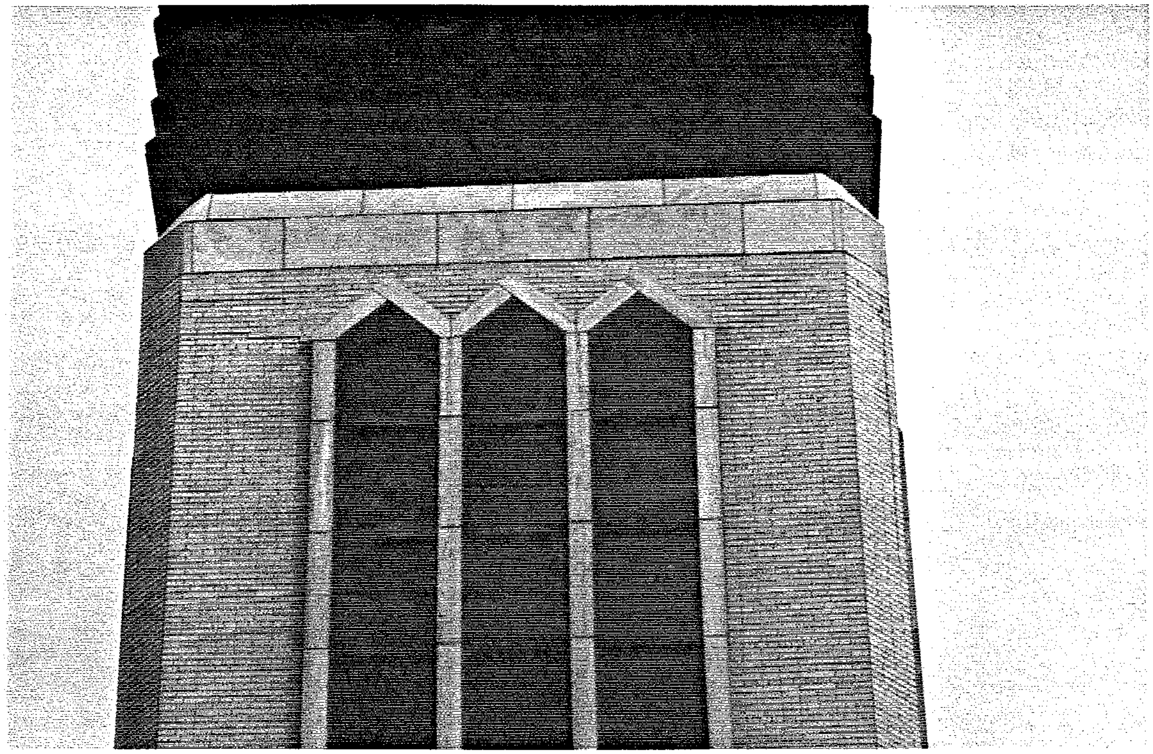


Figure 2.32 Cracking of masonry walls of bell tower.

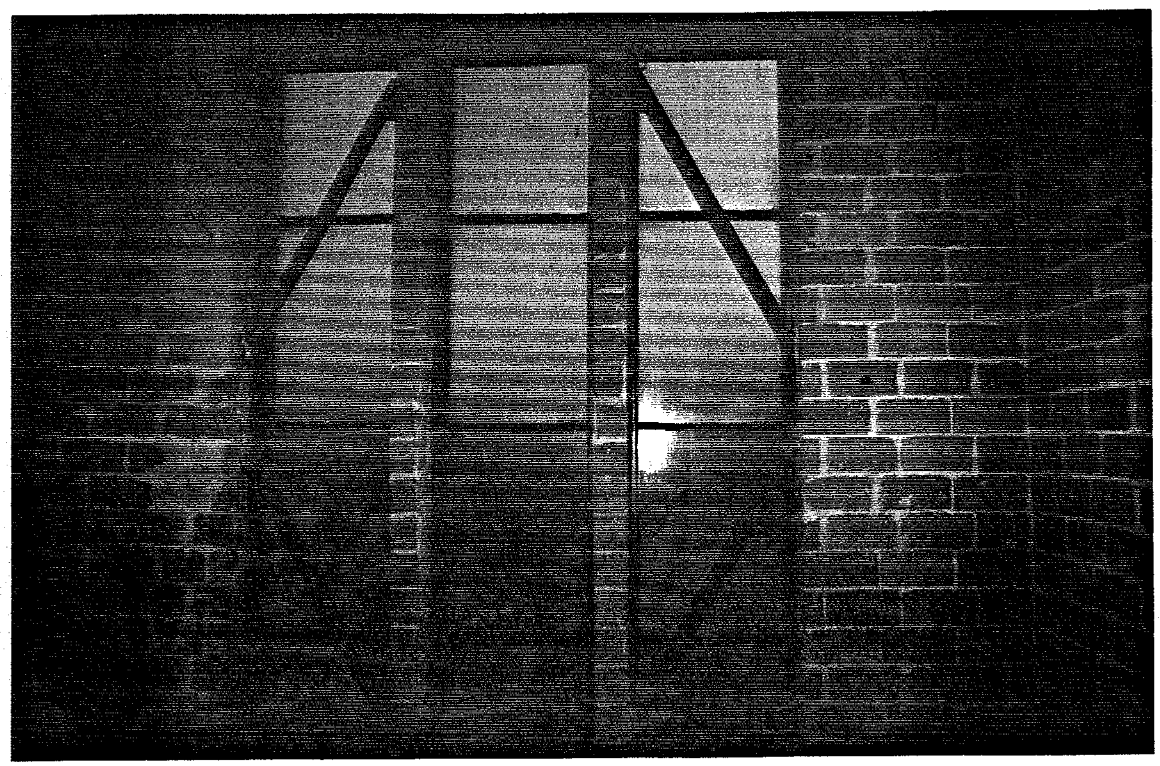


Figure 2.33 Inside view of cross-bracing and masonry walls of bell tower.

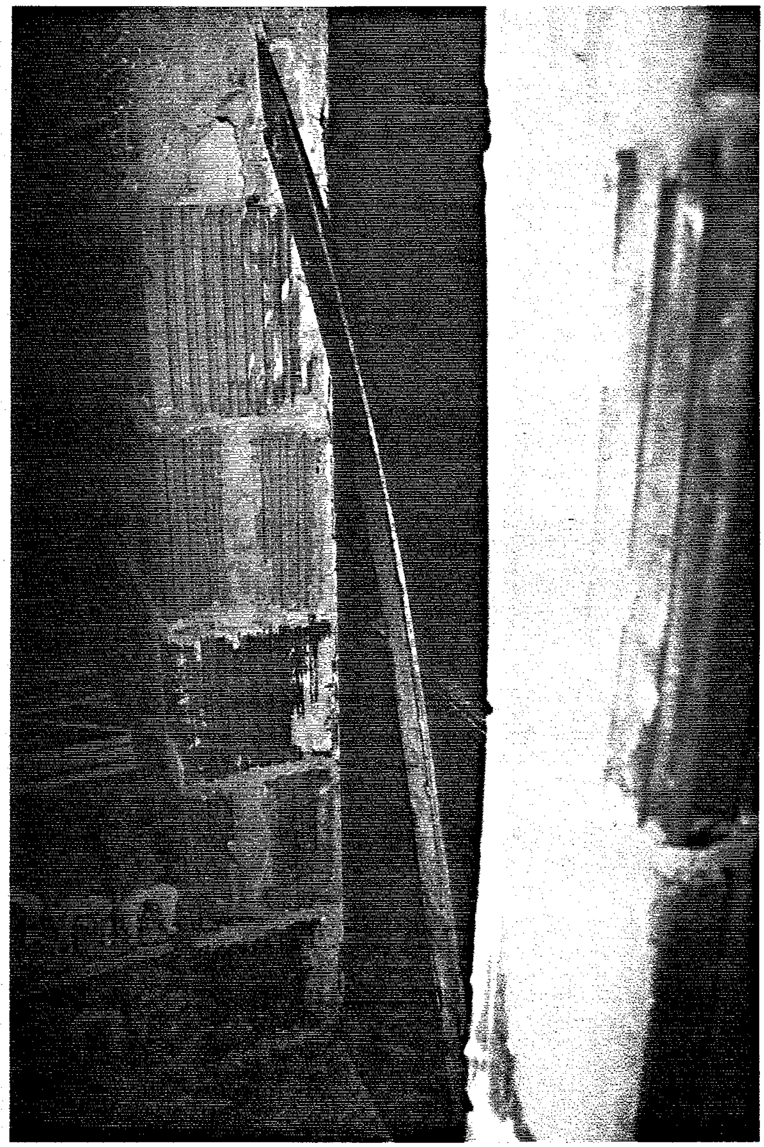


Figure 2.34 Buckled tension cross-brace.

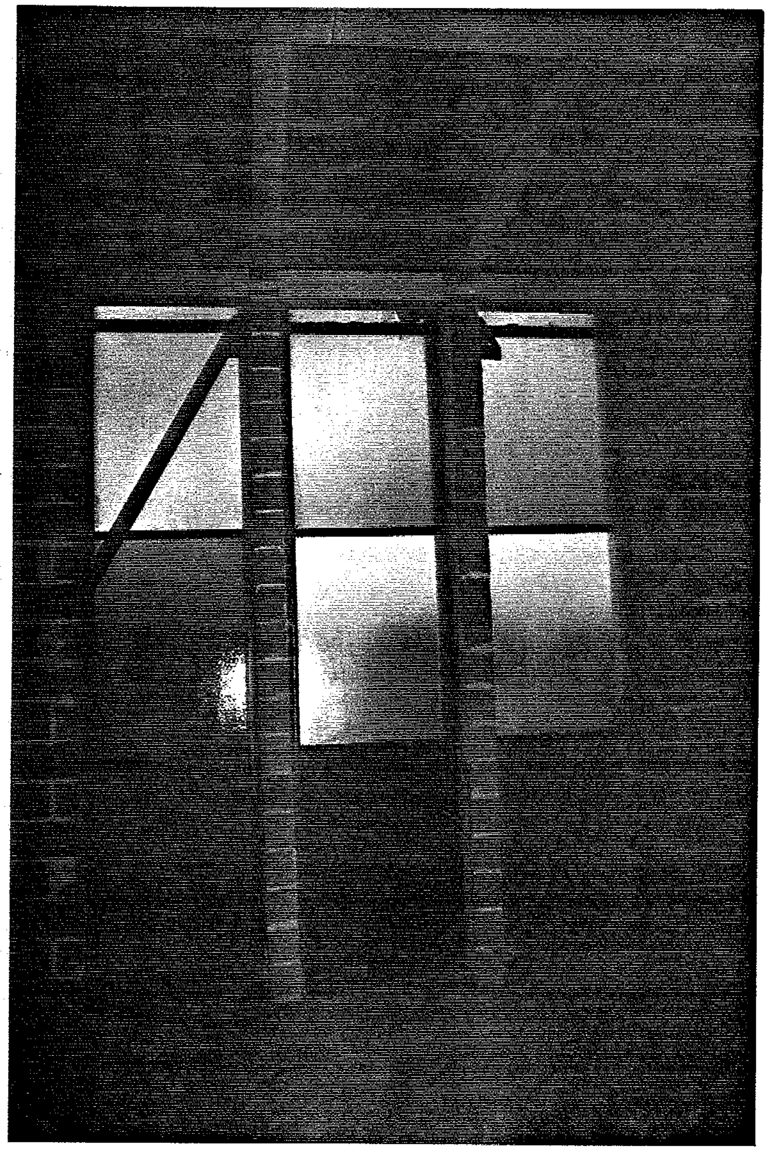


Figure 2.35 Missing cross-brace in the sixth storey of bell tower.

Figure 2.36 Steel tension braces in plane of floor.

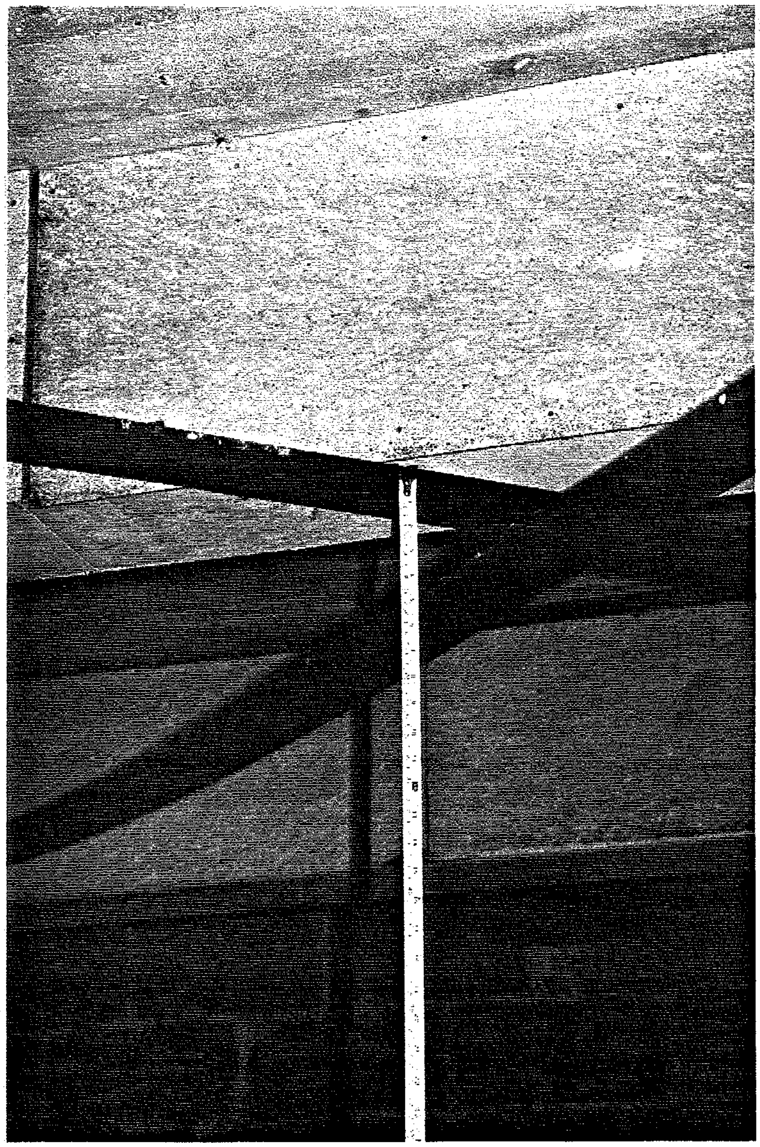


Figure 2.37 Buckling of steel braces in floor diaphragm.

Figure 2.38 Well compacted sand fill embankment near Boulevard Talbot.

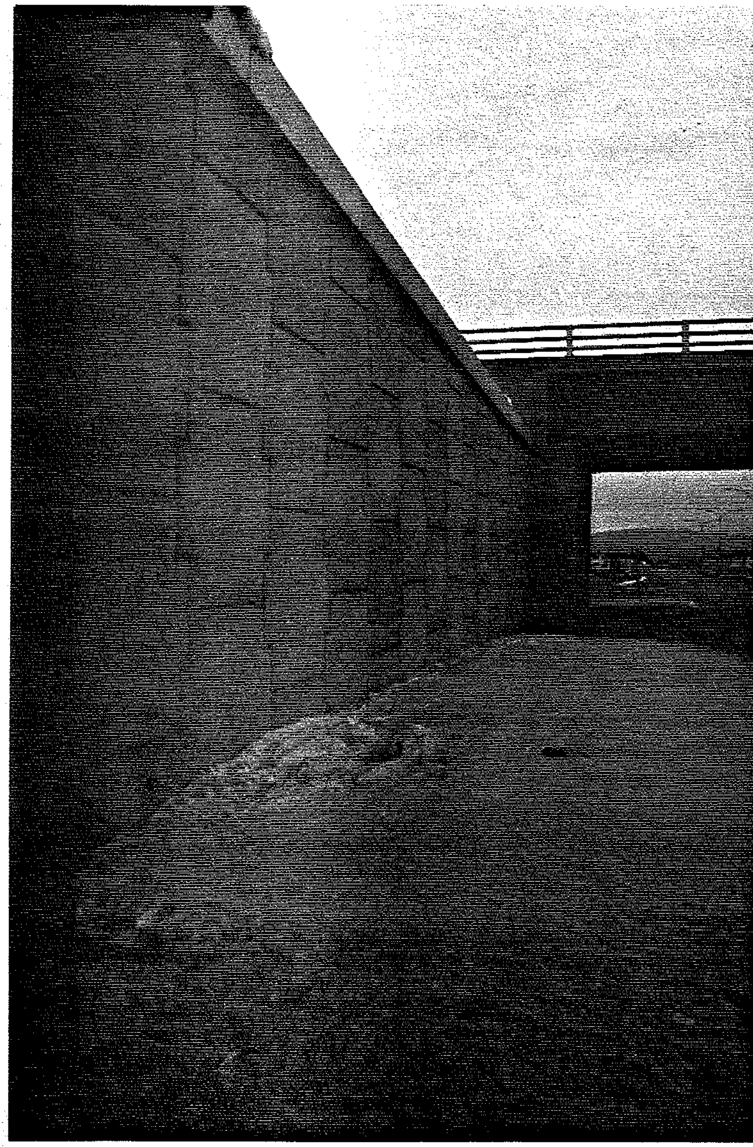


Figure 2.39 Reinforced concrete retaining wall for embankment.

Figure 2.40 Natural slope landslide close to the epicentral region.

CHAPTER 3

DAMAGE IN VILLE DE LA BAIE

3.1 Description of Ville de la Baie

The town of La Baie is located on the west shore of the Baie des Ha! Ha! on the Saguenay River. The major industries of this town (population of 21,000) are pulp and paper and aluminum production. Figure 3.1 shows an overall view of the town nestled between rolling hills and the Baie des Ha! Ha!. Figure 3.2 shows the overall plan of the town and location of all the buildings visited.

3.2 Edifice Municipal-Police et Feu

Figure 3.3 shows a side view of the Municipal Building which houses the police and fire departments for the region which is located at 491 Boulevard de la Grande-Baie. The original structure built in 1935 consists of a four-storey masonry structure with a six-storey tower. Two newer additions were added on at the back of the building in 1986 and 1988. Figure 3.4 shows cracks in the infilled masonry walls in the second storey in the new extension above the fire station garages. In spite of this minor damage, the facility remained fully operational following the earthquake. Reports indicate that the electric power was off for about three hours.

3.3 Maison Provinciale des Filles de Ste-Marie de la Présentation

Figure 3.5 shows the front view of the Maison Provinciale des Filles de Ste-Marie de la Présentation which is a two and a half storey reinforced concrete frame structure with a folded plate roof. The main wing is 230 ft. x 50 ft. in plan and the chapel wing, located at the back of the building, is 60 ft. x 30 ft. in plan (see Fig. 3.6).

Extensive architectural damage was observed inside the chapel behind the altar. Figure 3.7 shows a decorative unreinforced masonry wall that collapsed. The reasons for the severe damage of this wall include the following:

(i) Lack of reinforcement in the wall.

(ii) Stacking of the masonry concrete blocks in their upright position.

(iii) Ineffective horizontal ties between architectural wall and back-up wall due to severe misalignment of masonry joints (see Fig. 3.8).

(iv) The direction of the earthquake matches the weak plane of the wall.

The whole building suffered severe cracking in most of the unreinforced masonry walls. These were 8 in. x 16 in. hollow concrete blocks with 4 in. thickness.

3.4 Ecole Polyvalente de la Baie

Ecole Polyvalente de la Baie, located at 1802 Avenue John-Kane, is a large secondary school with 140 teachers and 1500 students. The original structure, built in 1964 consists of one and two-storey construction. Large additional wings were added in 1970 to the one-storey portions of the structure. These two-storey additions were placed on top of the older wing and created an open-air parking area at the street level as shown in Fig. 3.9.

This new addition was on pile foundations due to the soft clay underlying soil and due to the presence of a steep slope on one side of the school (see Fig. 3.10). Although this slope had to be stabilized in the past, there was no evidence of any ground movement due to the earthquake.

Due to the "soft-storey" created by the new wings there was differential movement of about 1 in. between the old wing and the newer wings. All the classrooms over the "open-air" parking space experienced significant cracking in the unreinforced masonry partition walls. Furthermore, all the corridor walls in the newer wings were cracked to such an extent that they were easily displaced by leaning against them.

Figure 3.11 shows one of the gymnasium walls that suffered damage. One of these walls, which was approximately 33 ft. high, lost the top three layers of masonry blocks. All the walls were unreinforced masonry. Temporary repair measures, which were already taking place, included bolting 8 ft. long channel sections to both sides of the interior masonry walls centred on the steel column locations (see Fig. 3.11). In addition, a steel angle was added in order to hold the top layer of blocks in place. This angle, which runs parallel to the face of the exterior wall, was attached to the bottom of the roof joists by welding as shown in Fig. 3.12.

Figure 3.13 shows architectural damage to the ceiling tiles and to the lighting fixtures (see Fig. 3.14) of the auditorium located on the second floor. The ceiling was supported by inverted tee sections supported by vertical hangers. Also, the two top-most layers of concrete blocks fell from the back wall of the stage of the auditorium.

In the older wings, cracking was observed in all of the exterior concrete columns at the second floor level as shown in Fig. 3.15. This cracking occurred in a horizontal plane at the top of the deep spandrel beam. It is believed that this crack has taken place at the location of construction joints in the column (see Fig. 3.16).

It was interesting to note that the emergency lighting generator did not function after the earthquake due to poor electrical contacts that had to be repaired. Temporary repairs were carried out and the school was to be re-opened on Wednesday, November 30, 1988.

3.5 Ecole Georges Vanier

The Georges Vanier primary school, located at 251 rue Sirois in Ville de la Baie, contains 14 classrooms. The plan dimensions of the school are 66 ft. x 240 ft. Figure 3.17 shows the overall view of the two-storey school which has a rather unusual structural system.

The open space at the ground level creates a "soft-storey" effect while the masonry walls at the ends of the structure stiffen the building. Furthermore, the folded plate roof is supported by one set of columns while the second floor slab is supported by a different set of columns. Due to the ground motion, there was considerable differential movement between the roof and the floor slab causing extremely severe damage to the masonry partition walls between the slab and the roof (see Fig. 3.18 and 3.19).

Temporary bracing (see Fig. 3.20) was added across the top of the unreinforced masonry corridor walls which were so flexible that they could easily be displaced by hand.

As can be seen from Figures 3.21 and 3.22 considerable cracking occurred in the roof around the perimeter of the column supporting it. Just below the roof level cracking was also observed in the column.

As expected, the differential movement between the roof and the floor slab was greatest in the middle of the structure, that is, halfway between the stiff masonry walls at the ends of the structure. Damage was further aggravated by the fact that the weak direction of the structure coincided with the direction of ground motion propagation.

Figure 3.23 shows the temporary repair which consists of lateral support for the masonry partition walls. This consisted of light gauge metal preformed angles that were moulded to fit the angle of the roof and were bolted through the partition walls.

3.6 Geotechnical Observations

Soil settlement resulting in a small surface slump was observed on Route 170 between Chicoutimi and Ville de la Baie as shown in Fig. 3.25. A small surface slump was observed just outside Ville de la Baie as shown in Fig. 3.26. A more obvious, but small slope failure (see Fig. 3.27) was observed on Route 381 between Ville de la Baie and the villages of Ferland and Boileau.

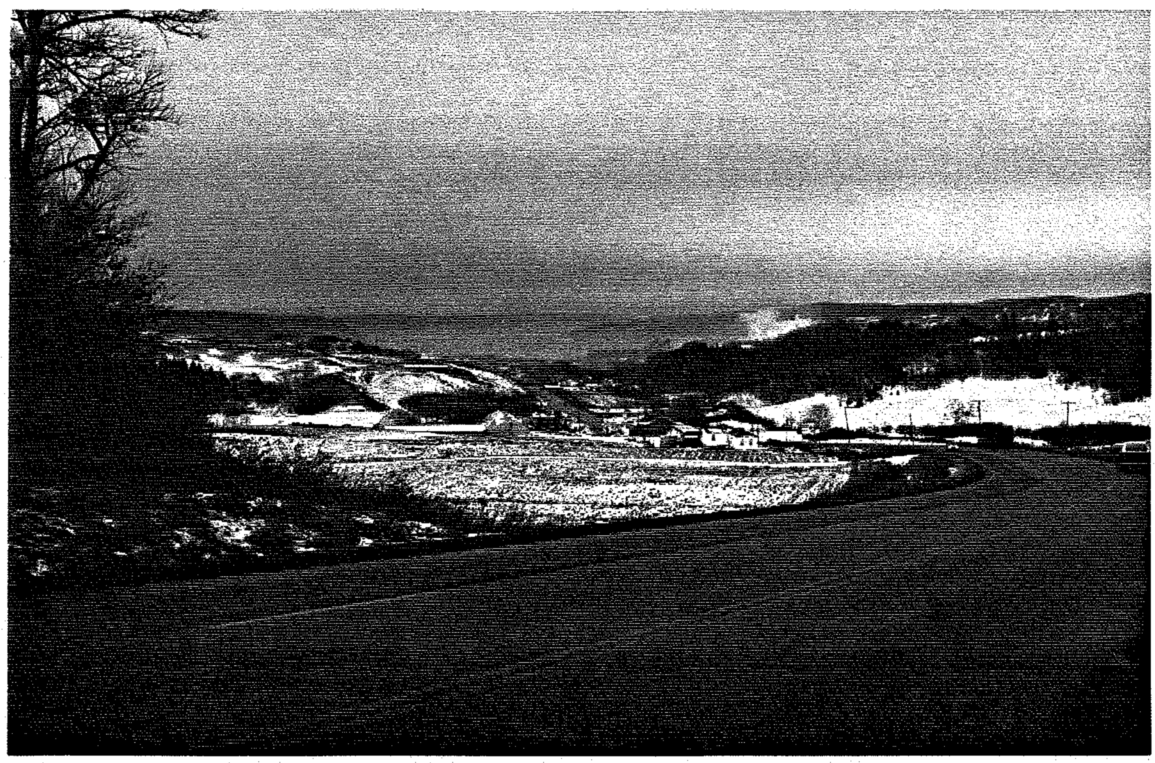


Figure 3.1 View of region surrounding Ville de la Baie.

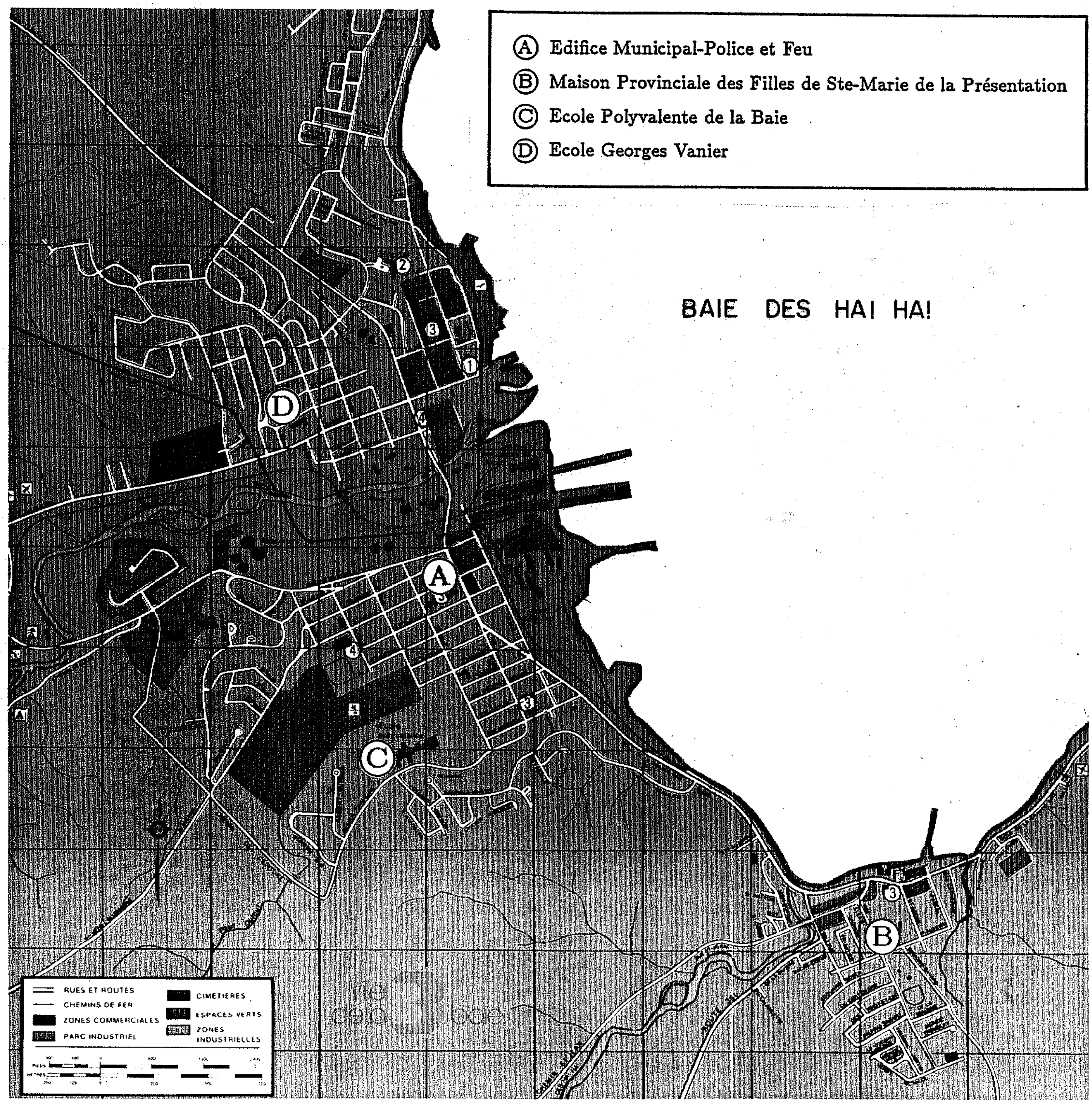


Figure 3.2 Map of Ville de la Baie.

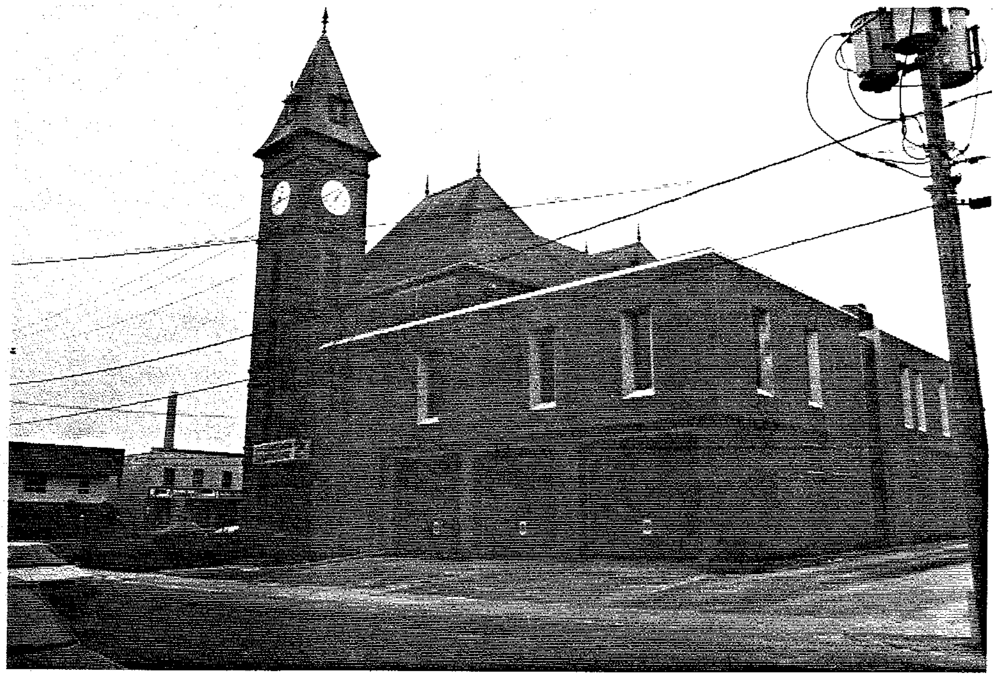


Figure 3.3 Edifice Municipale - Police et Feu, Ville de la Baie.

Figure 3.4 Cracks in second floor masonry walls of Edifice Municipale.

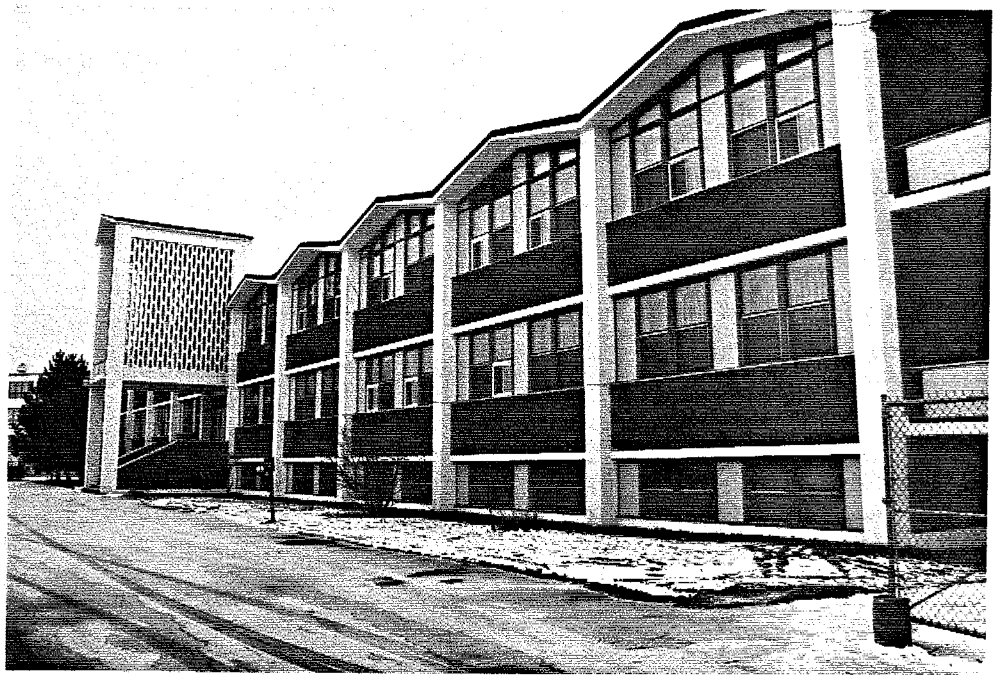


Figure 3.5 Front view of Maison Provinciale des Filles de Ste-Marie de la Présentation.

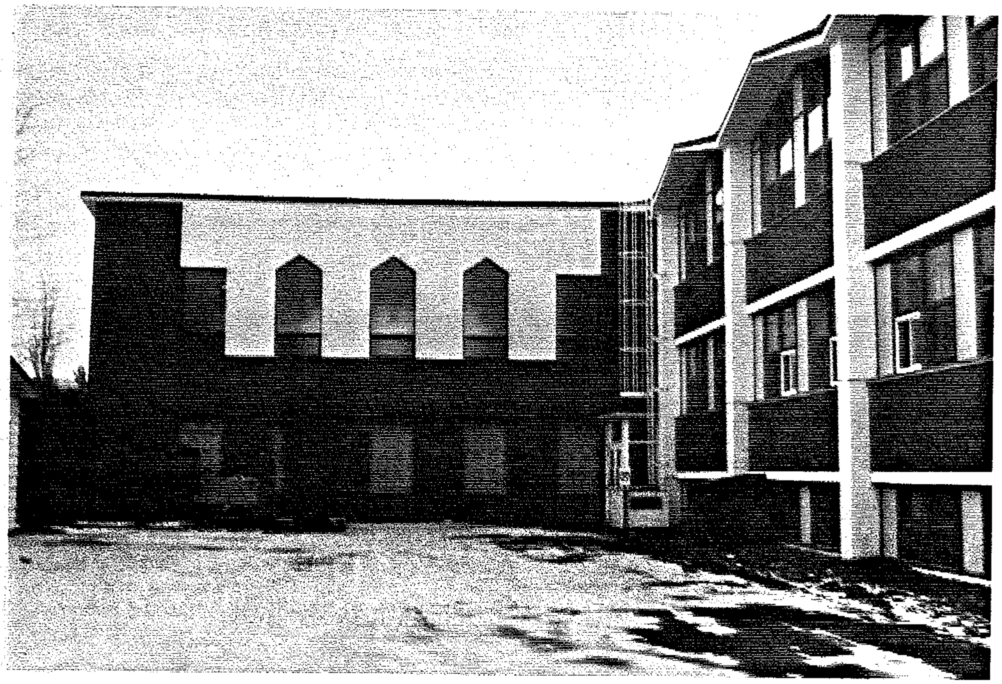


Figure 3.6 Chapel Wing of Maison Provinciale des Filles de Ste-Marie de la Présentation.

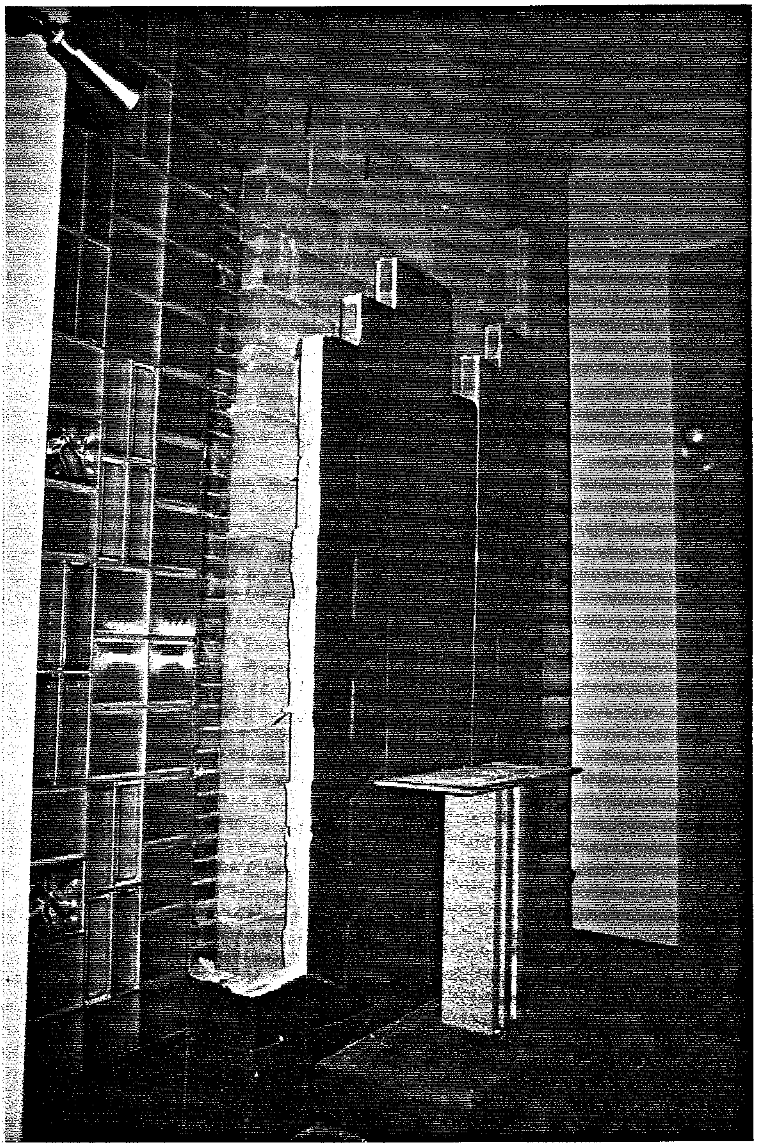


Figure 3.7 Damage to decorative wall in Chapel.

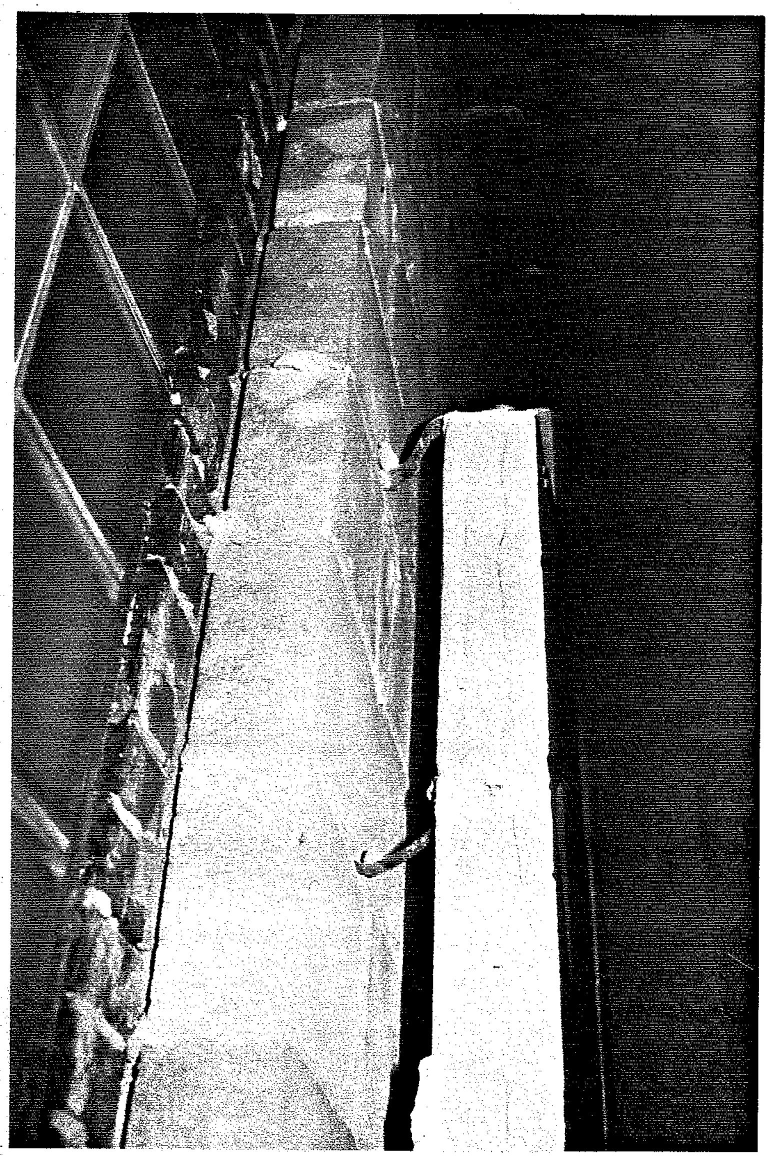


Figure 3.8 Ineffective tension ties between decorative wall and back-up wall.

Figure 3.9 Two-storey added wing of Ecole Polyvalente de la Baie.

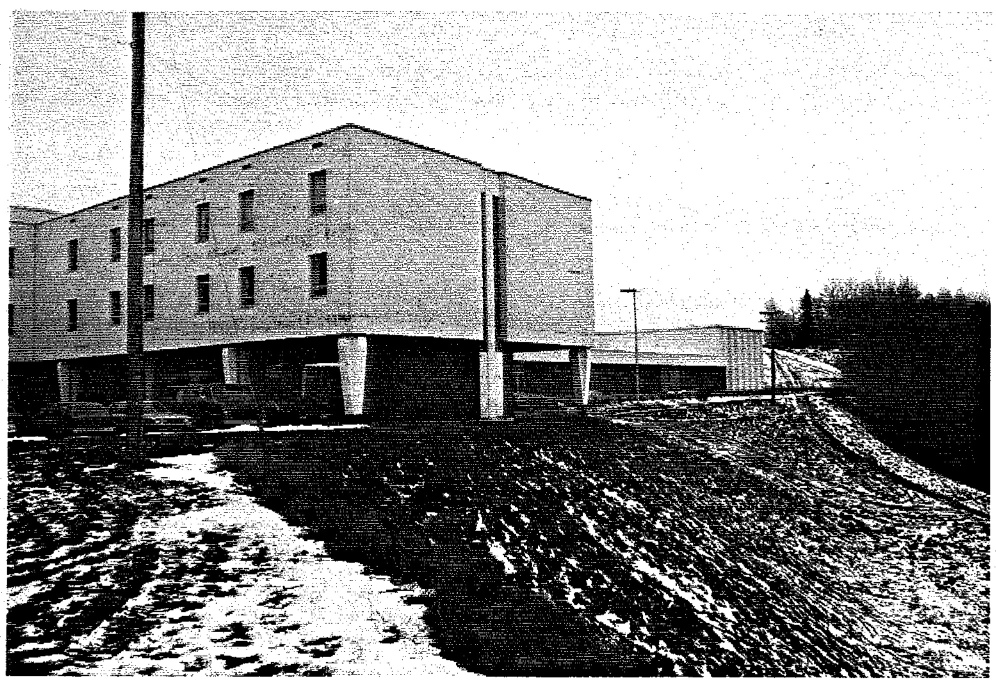


Figure 3.10 Front view of additional wing.

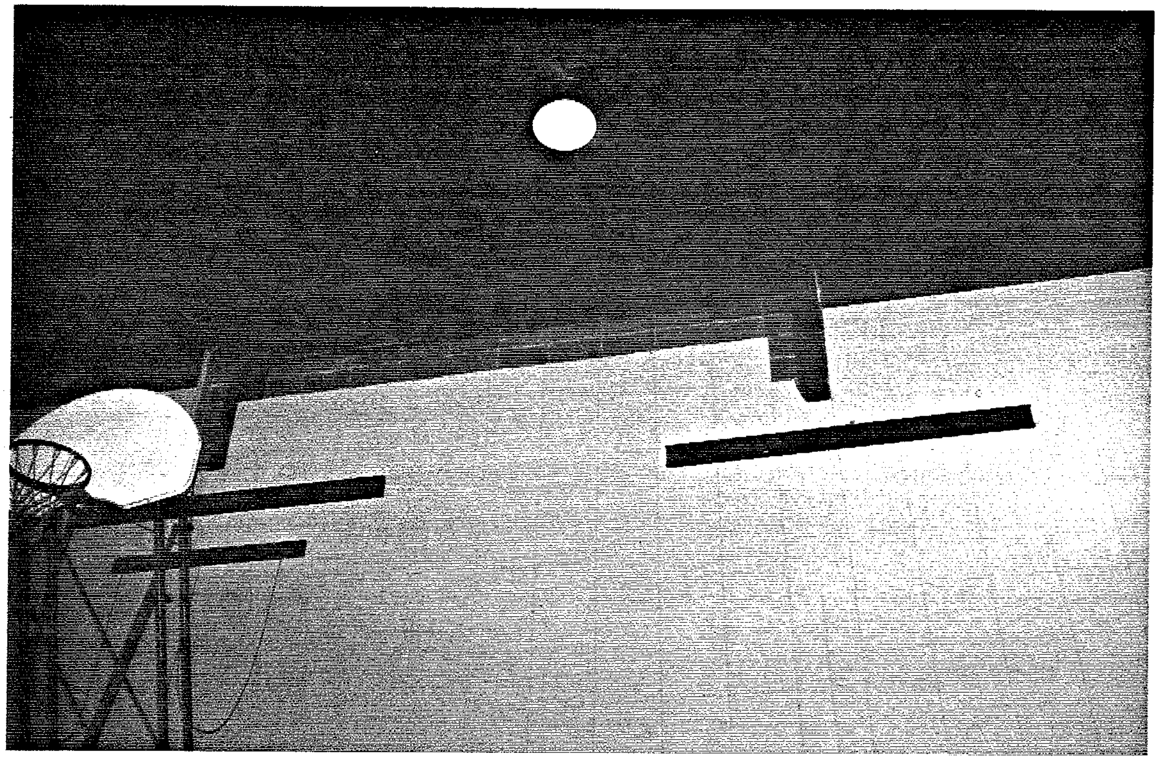


Figure 3.11 Damage to unreinforced masonry wall in gymnasium.

Figure 3.12 Temporary lateral support for wall provided by steel angle attached to bottom chord of roof trusses.

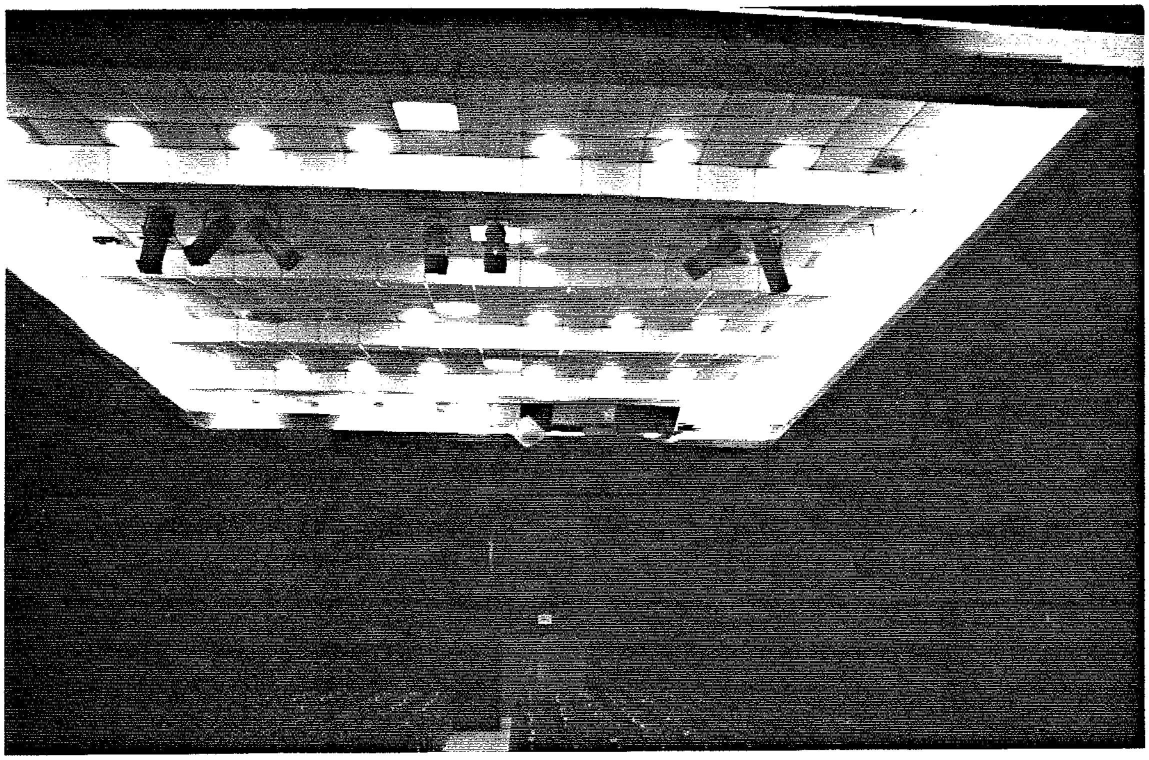


Figure 3.13 Damage to ceiling tiles and light fixtures of auditorium.

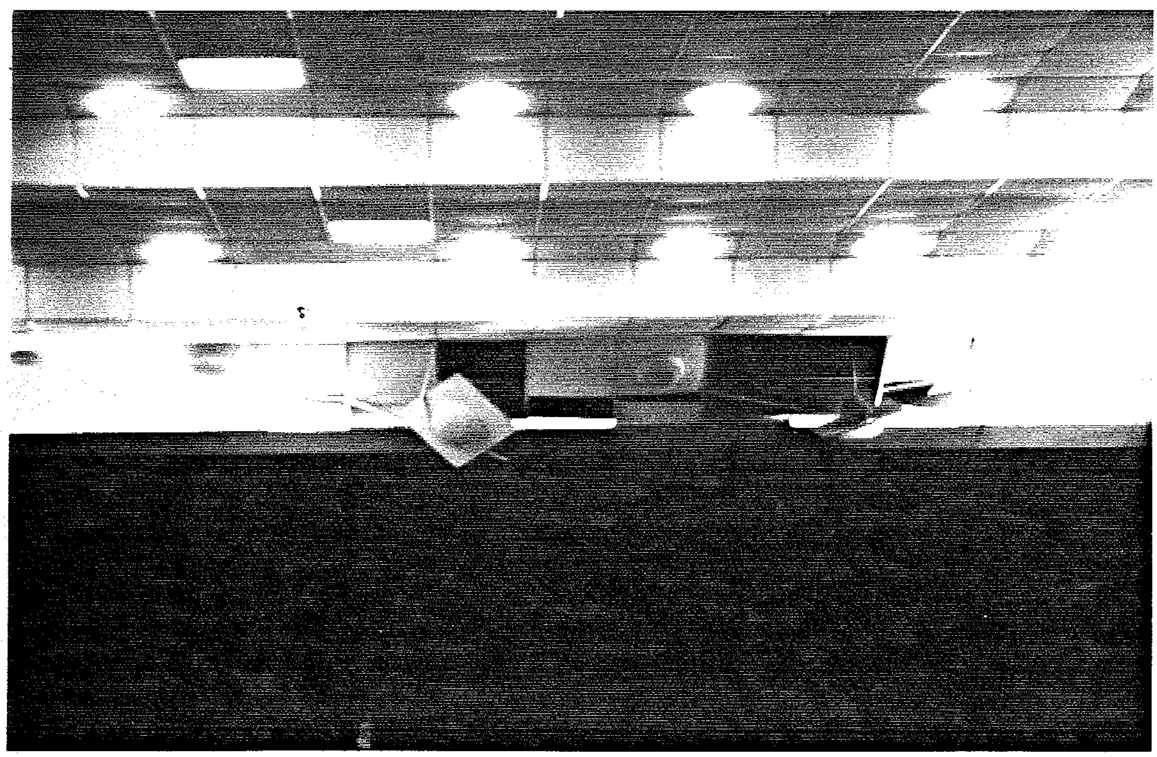


Figure 3.14 Close-up of damage to ceiling tiles and light fixtures.

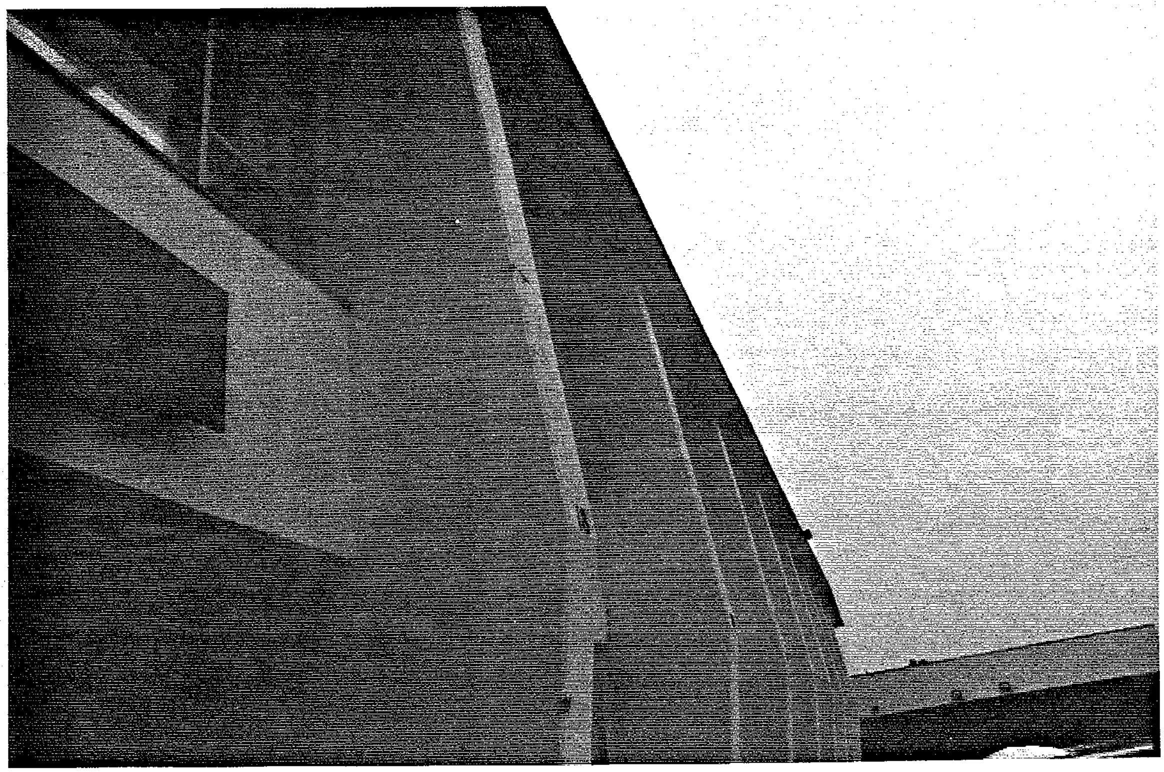


Figure 3.15 Horizontal cracking in exterior columns of older wing.

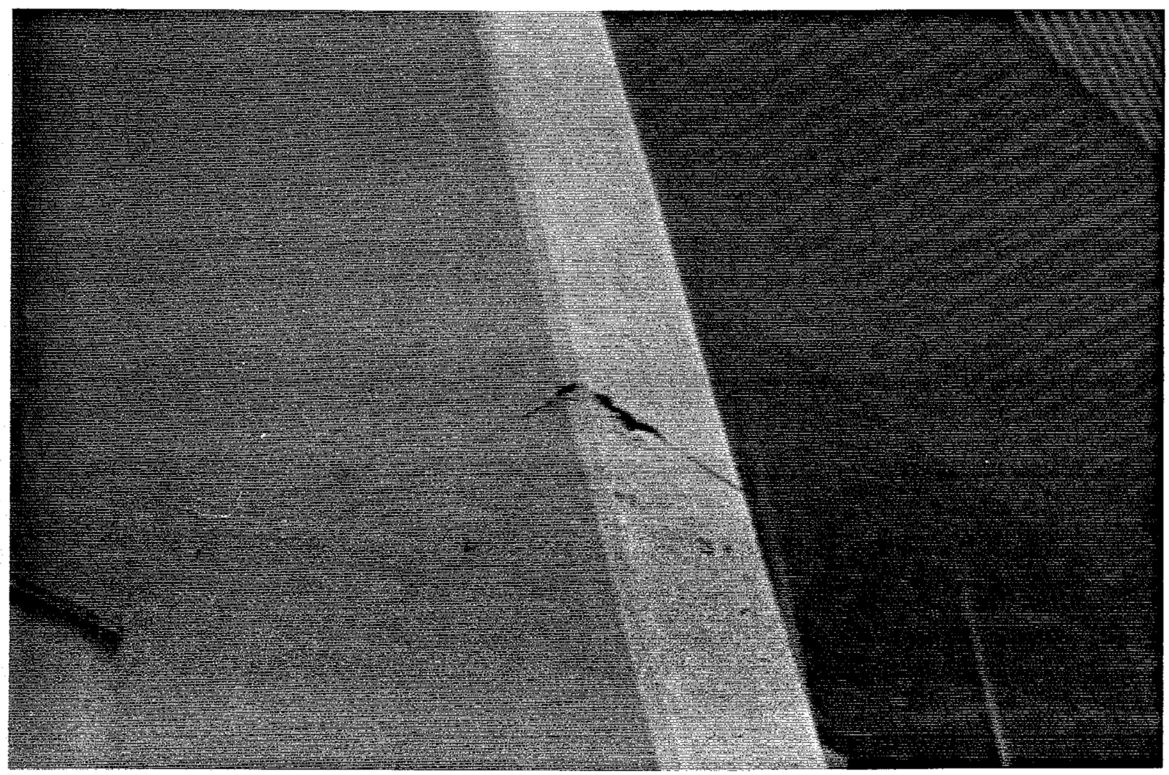


Figure 3.16 Close-up of horizontal cracking in column.

Figure 3.17 Overall view of Ecole Georges Vanier.

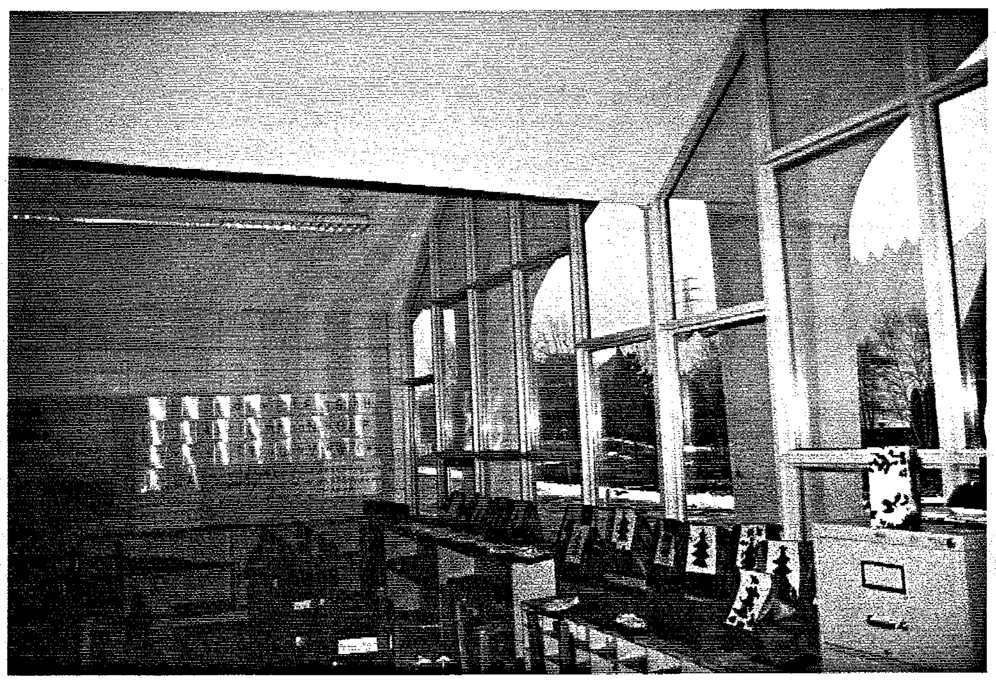


Figure 3.18 Inside view of classroom.

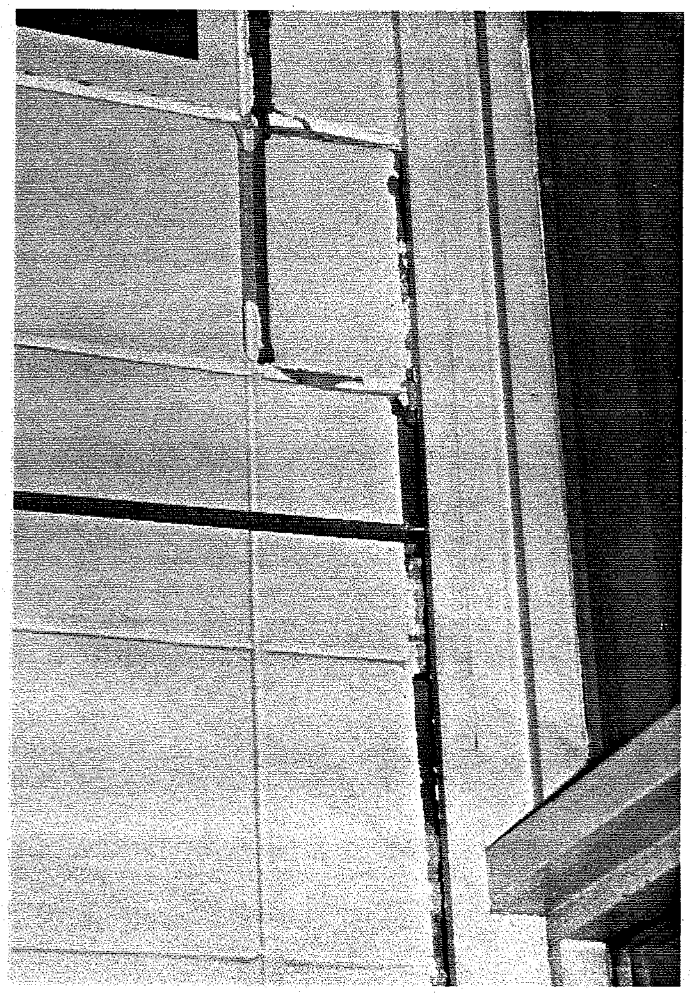


Figure 3.19 Close-up of cracking in masonry partition wall.

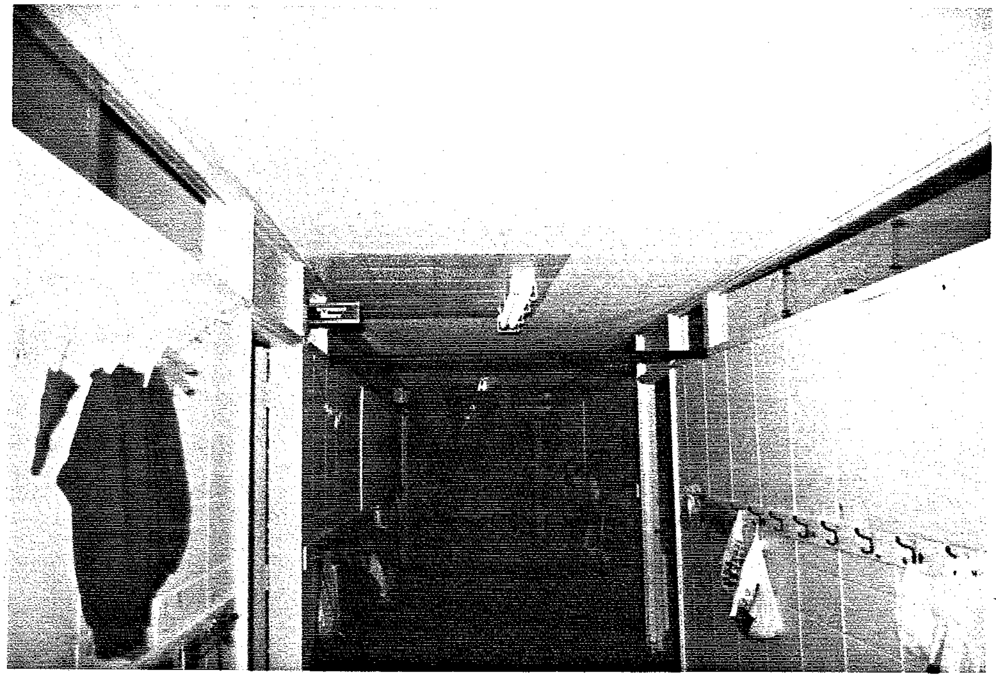


Figure 3.20 Temporary bracing of corridor masonry walls.

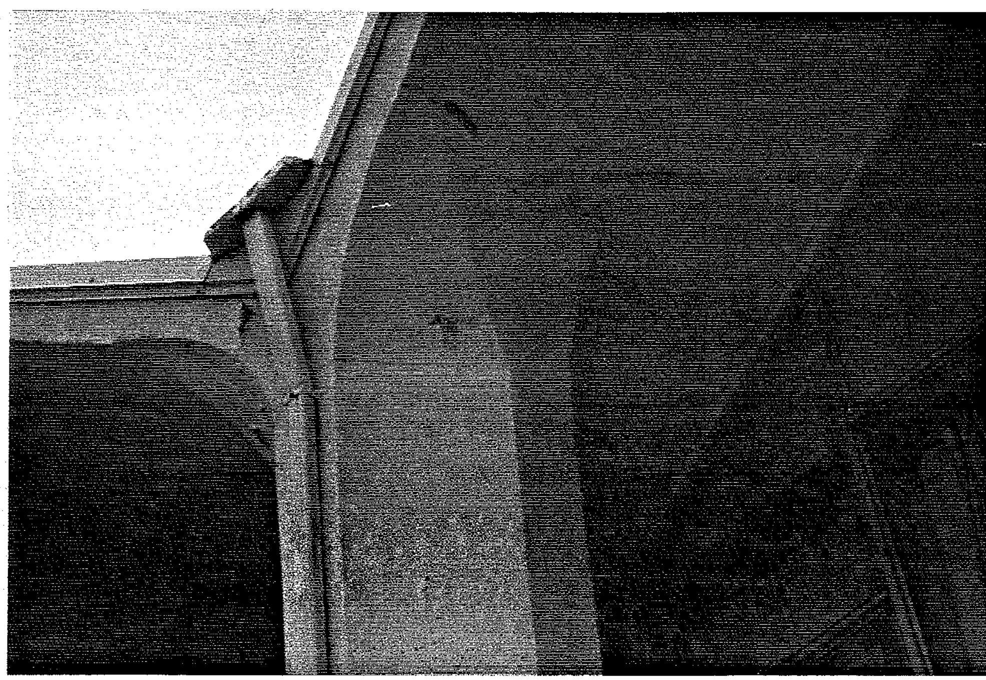


Figure 3.21 Cracking of column and roof slab.

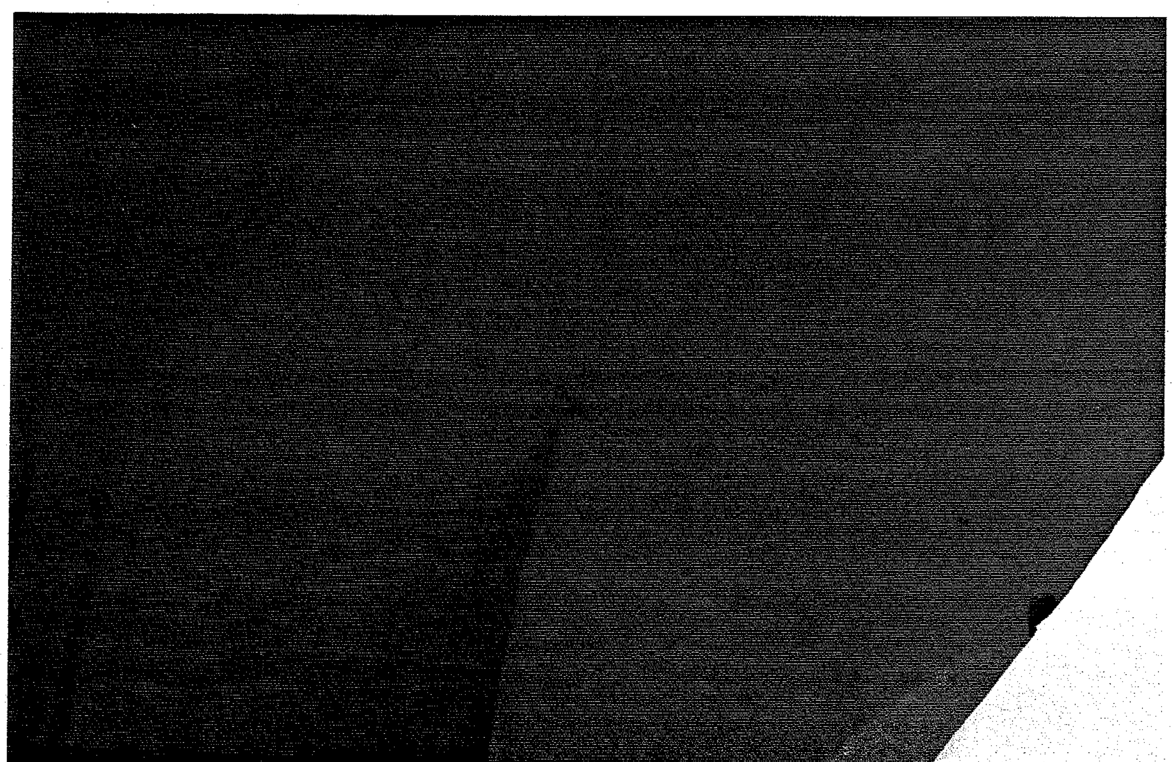


Figure 3.22 Close-up of cracking in roof slab.

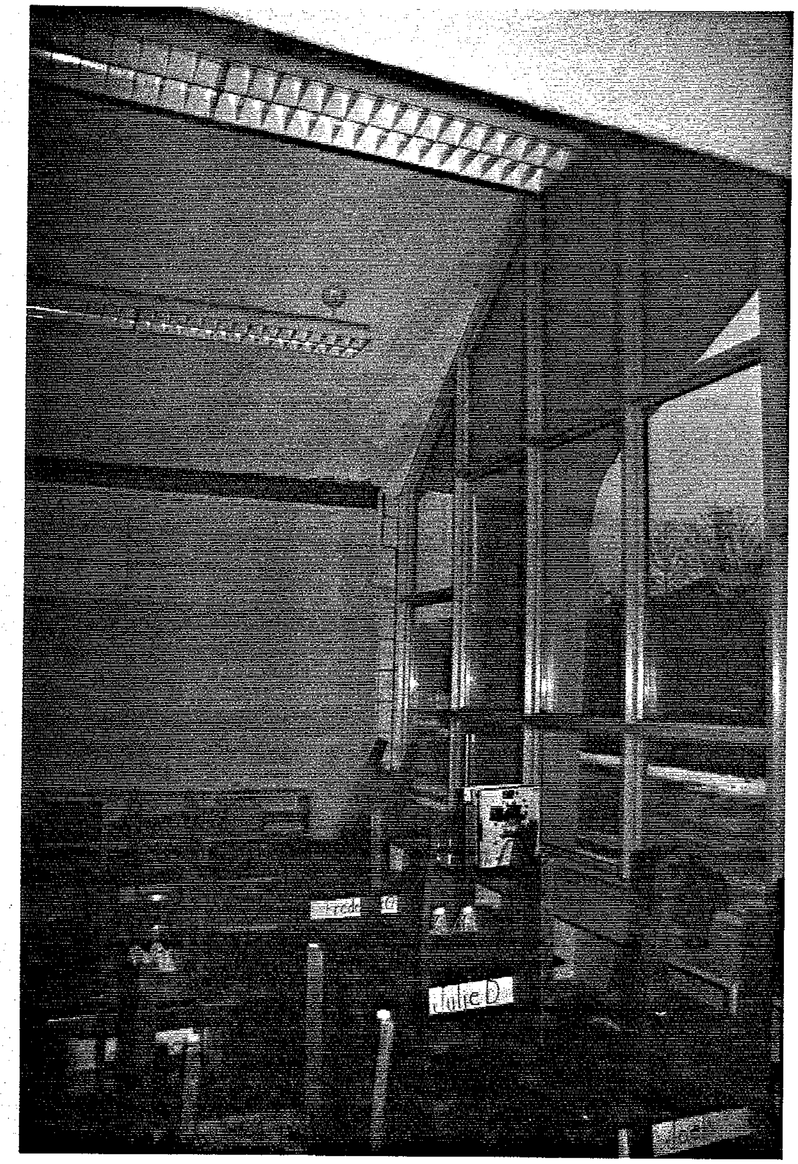


Figure 3.23 Temporary lateral support for partition walls.

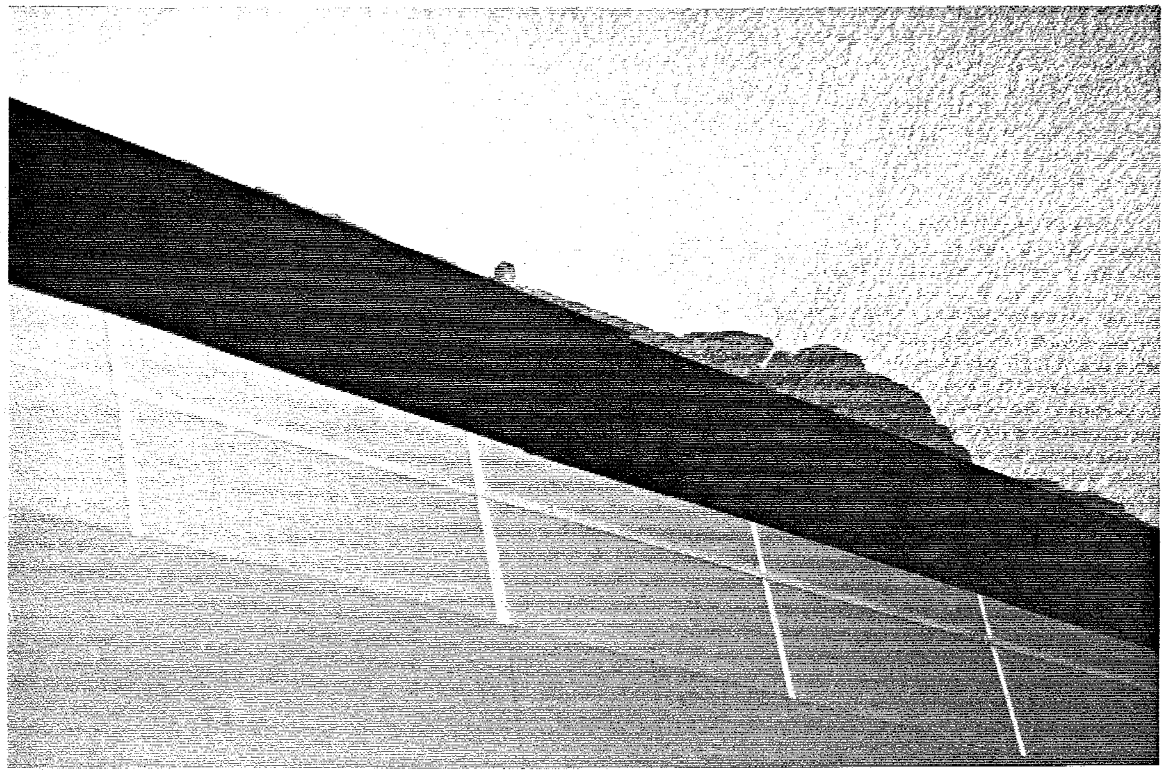


Figure 3.24 Light-gauge metal preformed angles bolted through partition walls.

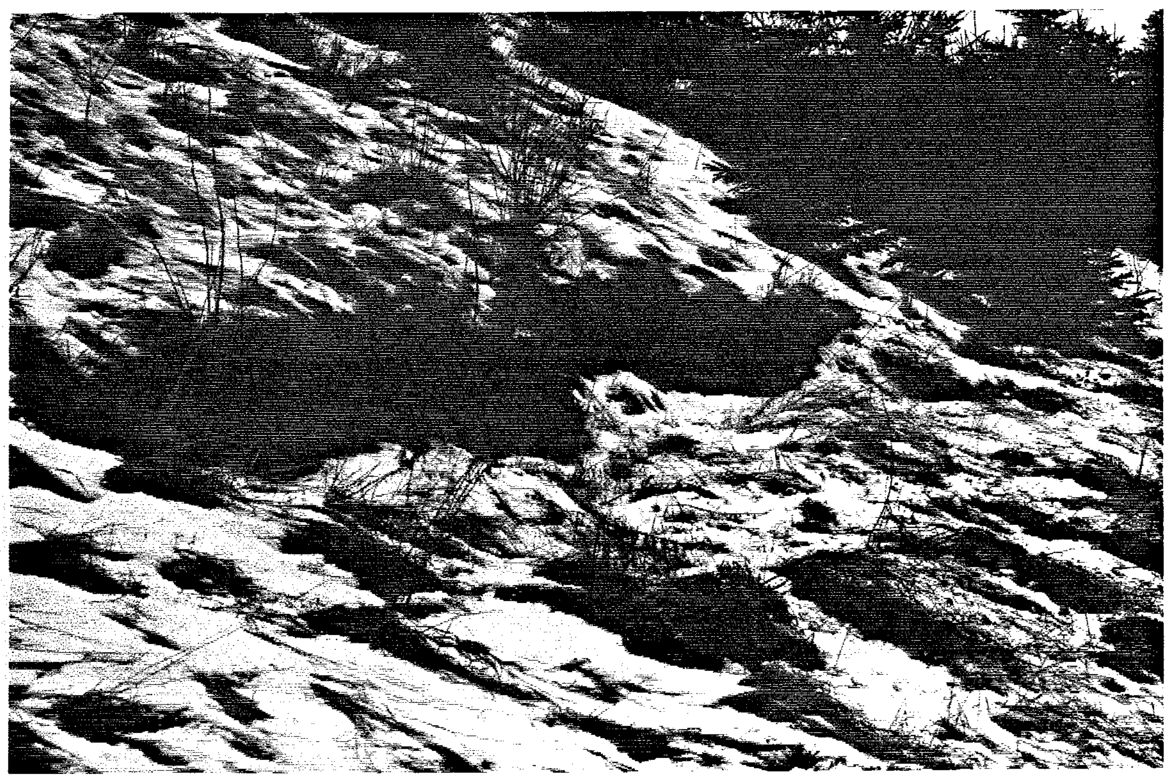


Figure 3.25 Small surface slump on Route 170.

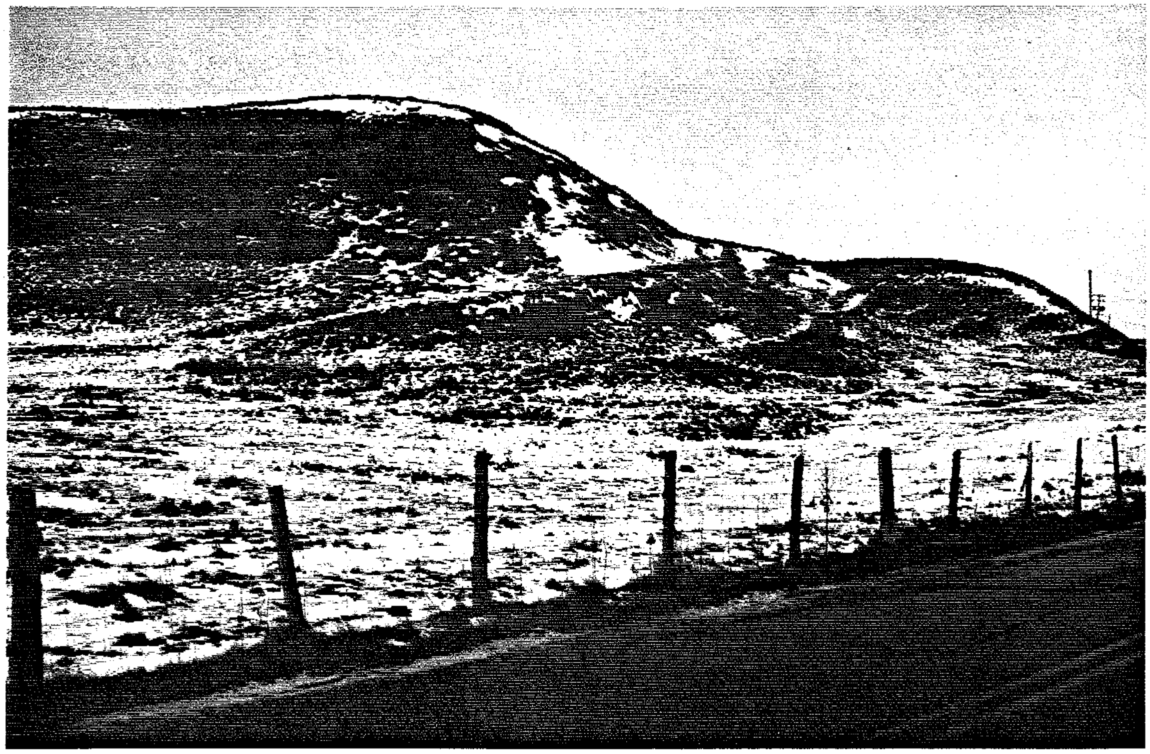


Figure 3.26 Small surface slump outside of Ville de la Baie.

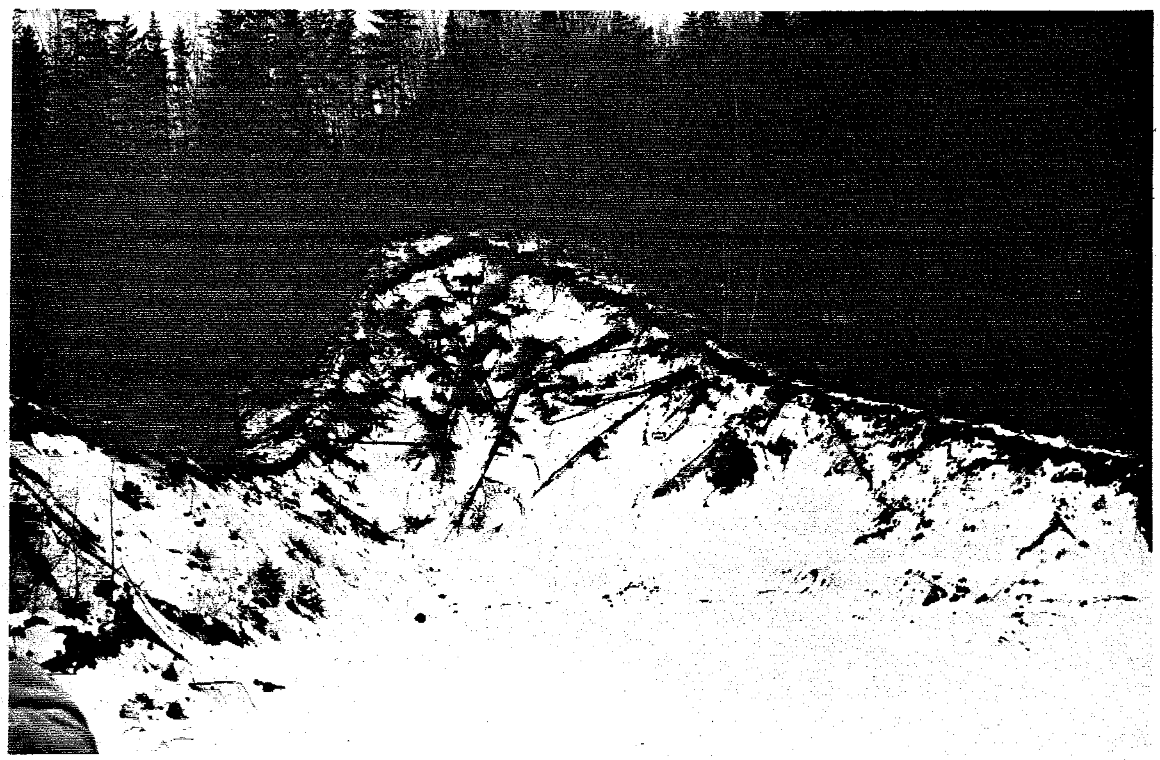


Figure 3.27 Small slope failure on Route 381.

CHAPTER 4

DAMAGE IN QUEBEC CITY

4.1 Description of Quebec City

Quebec City, the Provincial Capital, has a population of about 170,000. It is located on the north shore of the St. Lawrence River (see Fig. 1.1), approximately 150 km south of the epicentre.

4.2 Soil Conditions in Quebec City and its Influence on Damage

Quebec City can be divided into lower town ("basse ville") and upper town ("haute ville") based on the regional topology. The subsoils of lower town are largely composed of thick alluvial deposits from Rivière St. Charles. The approximate extent of these deposits is shown in Fig. 4.1. A cross–section of the subsoil profile along an axis coinciding with Dufferin–Montmorency (Route 138) near the mouth of Rivière St. Charles is shown in Fig. 4.2. As can be seen from Fig. 4.2 the soft soil deposits exceed 50 m in depth. In upper town the surface soil deposits are generally thin with bedrock out–cropping to the ground surface at some locations.

In the past major earthquakes (1870 and 1925) significant damage was reported in the soft soil regions of Quebec City due to amplification of the ground motion by the thick soil deposits in the lower town. Although there was no significant structural damage caused by the 1988 Saguenay earthquake in the Quebec City region, architectural and light structural damage was observed in the soft soil regions of lower town. The structural damage is described in Sections 4.3 to 4.5.

There were no reports of liquefaction of the alluvial deposits nor in the fill supporting large oil tanks on the north shore of the St. Lawrence River. Based on existing knowledge, liquefaction is possible only in very loose granular soil for an earthquake magnitude and epicentral distance similar to those of the present earthquake.

4.3 The Hippodrome

The Hippodrome in Quebec City is part of Parc de l'Exposition located on Boulevard du Colisée. This structure is an older unreinforced masonry building which suffered partial collapse of a masonry wall. Figure 4.3 shows the debris on the ground from the masonry wall that had existed between the two towers. The photograph in Fig. 4.3 was taken 7 days after the earthquake and after the remainder of the masonry wall had been demolished. Figure 4.4 shows the very flexible light steel frame that was used to provide lateral support for the wall.

4.4 Christ-Roi Hospital

Christ-Roi Hospital is located on Boulevard Wilfrid Hamel between Gauvin and Bernatchez streets. The hospital is very close to the St. Charles River and is in the soft soil region of Quebec City.

The structure is composed of three separate wings, two four-storey wings connected together by a five-storey wing to form a building that is U-shaped in plan (see Fig. 4.5).

Damage was very light throughout the hospital and consisted mainly of cracks in the plaster walls which were evident in the corridor connecting the two four-storey wings of the structure.

4.5 Saint-François d'Assise Hospital

Saint-François d'Assise Hospital is located at 10 rue de l'Espinay, very close to the St. Charles River. The main part of the hospital consists of three interconnecting wings some of which were built on an old river bed. The oldest wing (wing C) was built in 1914 with an extension added in 1929. Wing C is primarily a timber frame structure that suffered little damage in the 1988 earthquake. Wing B, built in 1947 is a nine-storey masonry structure (see Fig. 4.6). There was significant damage to the plaster walls of this wing, particularly in the corridors and the stair wells (see Fig. 4.7).

Wing A, which is the newest wing built in 1969, is connected to wing B by a corridor. Since this wing was built directly on an old river bed it had pile foundations. No significant damage occurred to this wing.

Two chimneys located on the roofs of wings A and C were damaged by the earthquake. One of these chimneys totally collapsed while the other chimney was severely damaged but did not collapse due to the presence of light gauge metal sheathing which had been wrapped around the chimney. A large unreinforced masonry chimney, built over 50 years ago and having a diameter of 8 ft. and a total height of approximately 150 ft. had suffered severe cracking over its service life. Fortunately, this chimney was demolished just two weeks before the earthquake.

At the time of the site-visit, tests were underway in order to determine the location of small leaks in the hot water radiator system. These leaks in the hospital heating system together with a power outage that lasted for about two hours were caused by the earthquake. It is important to note that the emergency generating system functioned without any problems.

One very interesting incident occurred in an elevator shaft of Wing B. During the earthquake, the 6,000 lb. counterweight of one of the elevators was derailed from its two vertical guide-rails. This derailment was probably due to the lack of stiffness of the guide-rails which were made from T-shaped steel sections. These vertical guide-rails were attached by steel brackets to the side faces of 6 in. x 14 in. reinforced concrete beams which were located inside the elevator shaft at each floor level. Each of the steel brackets were connected to the reinforced concrete beam by four 1/2 in. diameter bolts.

When the electrical power was restored and the elevator was functioning again, approximately two hours after the earthquake, the counterweight which was unrestrained by the guides impacted against one side of the steel guide-rail supports causing a shear failure in the reinforced concrete beam.

During the site-visit the position of the counterweight was adjusted (see Fig. 4.8) in order to recreate the sequence of events that had led to the failure of the reinforced concrete beam. Figure 4.9 shows the shear failure in the concrete beam, the guide-rail support and the temporary repair measures for the concrete beam. The concrete beam contained three #5 top bars and three #5 bottom bars but had no shear reinforcement. Figure 4.10 shows a close-up view of one of the four counterweight guides and the guide-rail.

Pavillon Notre Dame is a separate structure built in 1958. This 7 storey building has a steel frame with exterior masonry walls. Figure 4.11 shows diagonal cracks that occurred at the ground floor level due to shear failure of the masonry between the window openings.

4.6 References

Chagnon, J.-Y. and Doré, G. 1987. Le microzonage séismique de la région de Québec: essai méthodologique. vol. 11, no. 1, Cahiers du Centre de Recherches en Amenagement et en Développement (CRAD).

Ⓐ Hippodrome

Ⓑ Christ-Roi Hospital

Ⓒ Saint-François d'Assise Hospital

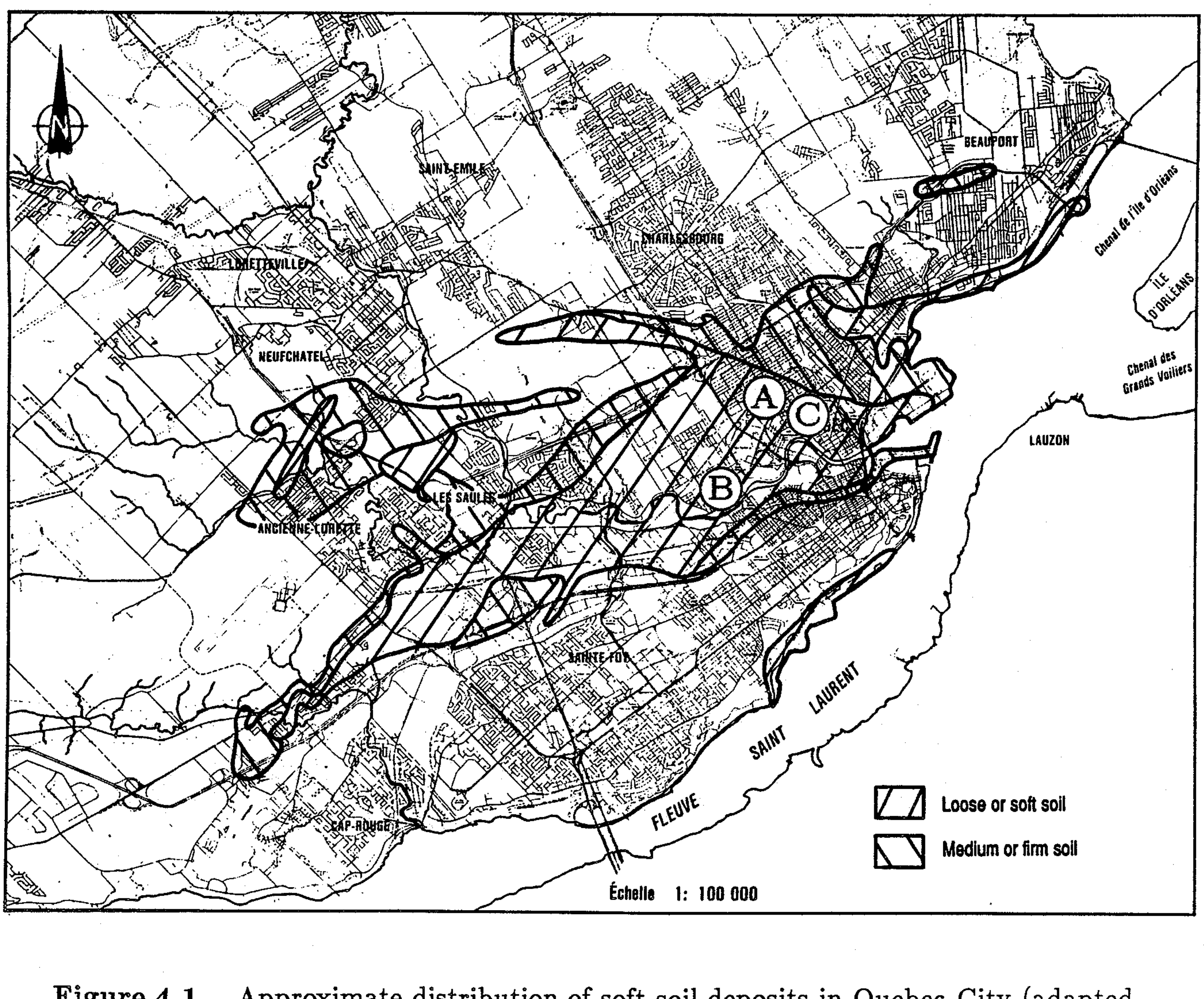


Figure 4.1 Approximate distribution of soft soil deposits in Quebec City (adapted from Chagnon and Doré 1987).

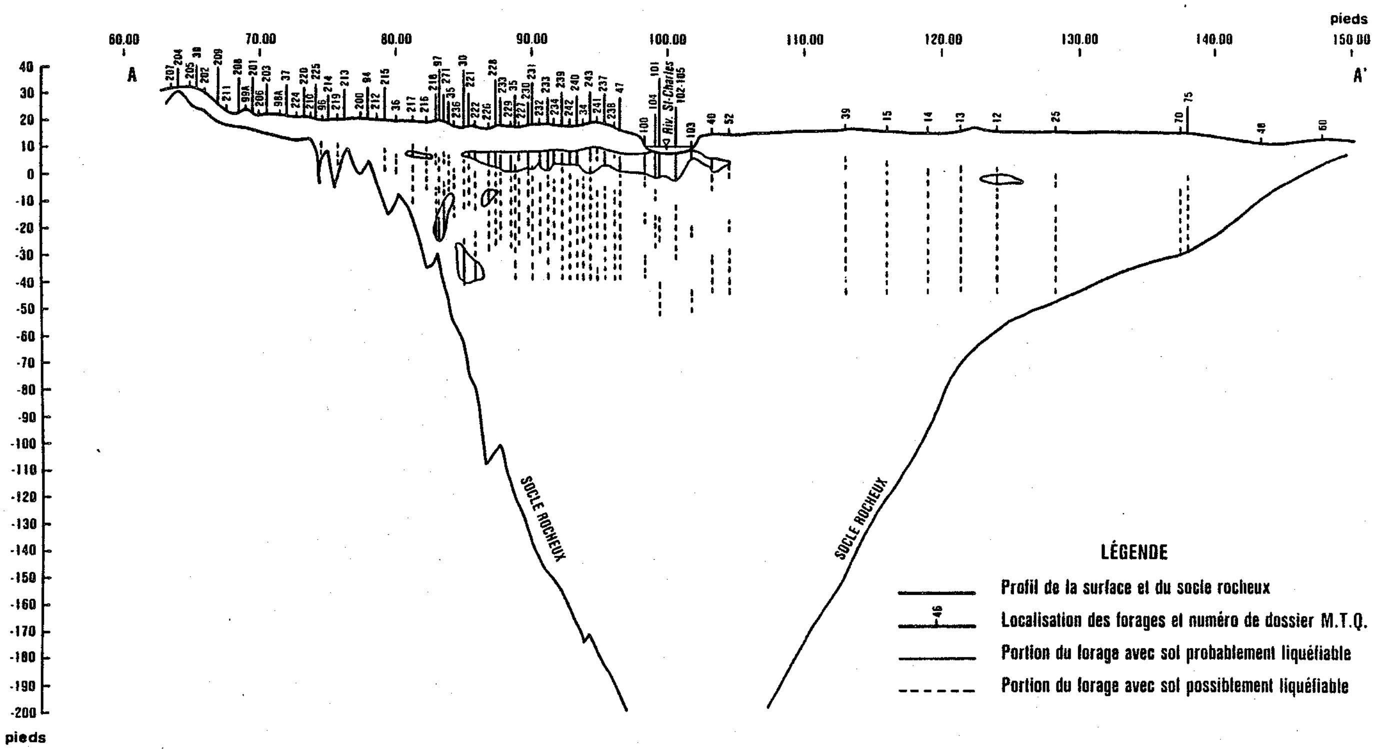


Figure 4.2 Cross-sectional profile along Dufferin-Montmorency (Route 138) in Quebec City, (adapted from Chagnon and Doré 1987).

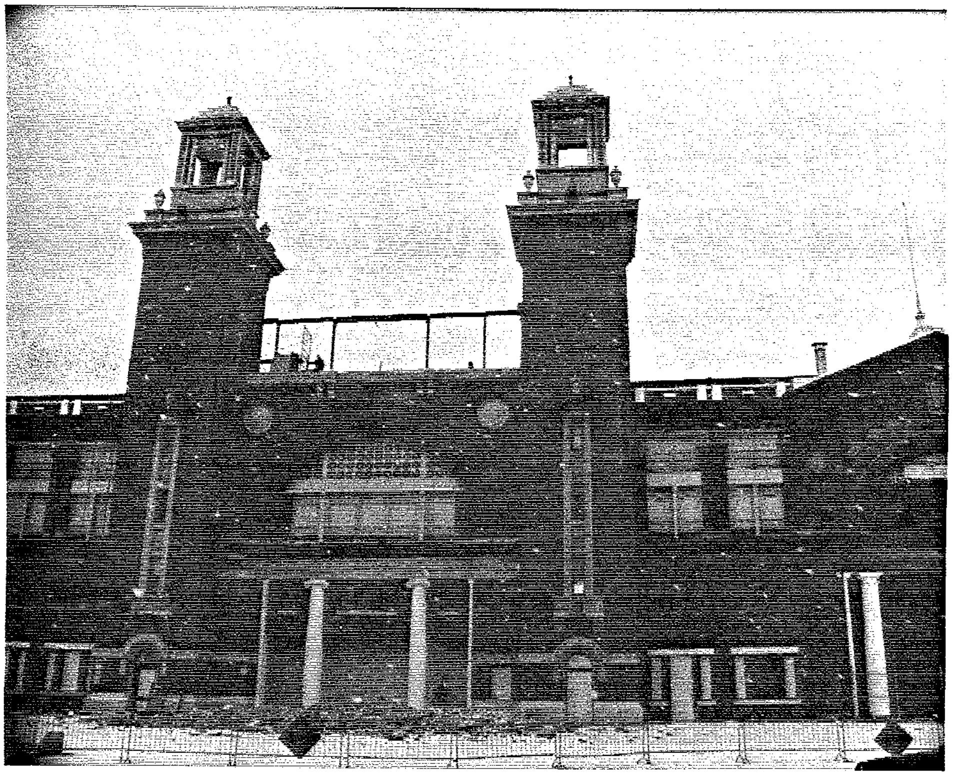


Figure 4.3 Overall view of Hippodrome.

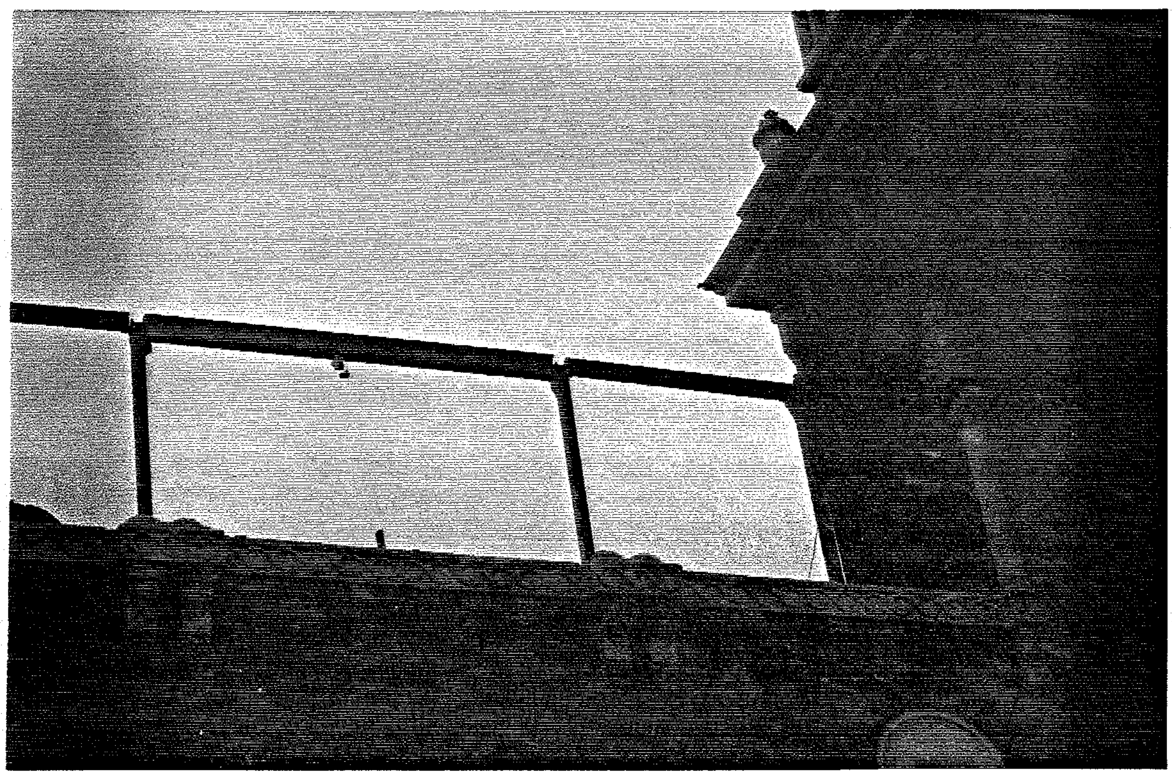


Figure 4.4 Close-up view of steel frame that supported masonry wall.

Figure 4.5 Christ-Roi Hospital.

Figure 4.6 Wing B of Saint-François d'Assise Hospital.

Figure 4.7 Severe cracking and spalling of plaster in stairwell of Saint-François d'Assise Hospital.

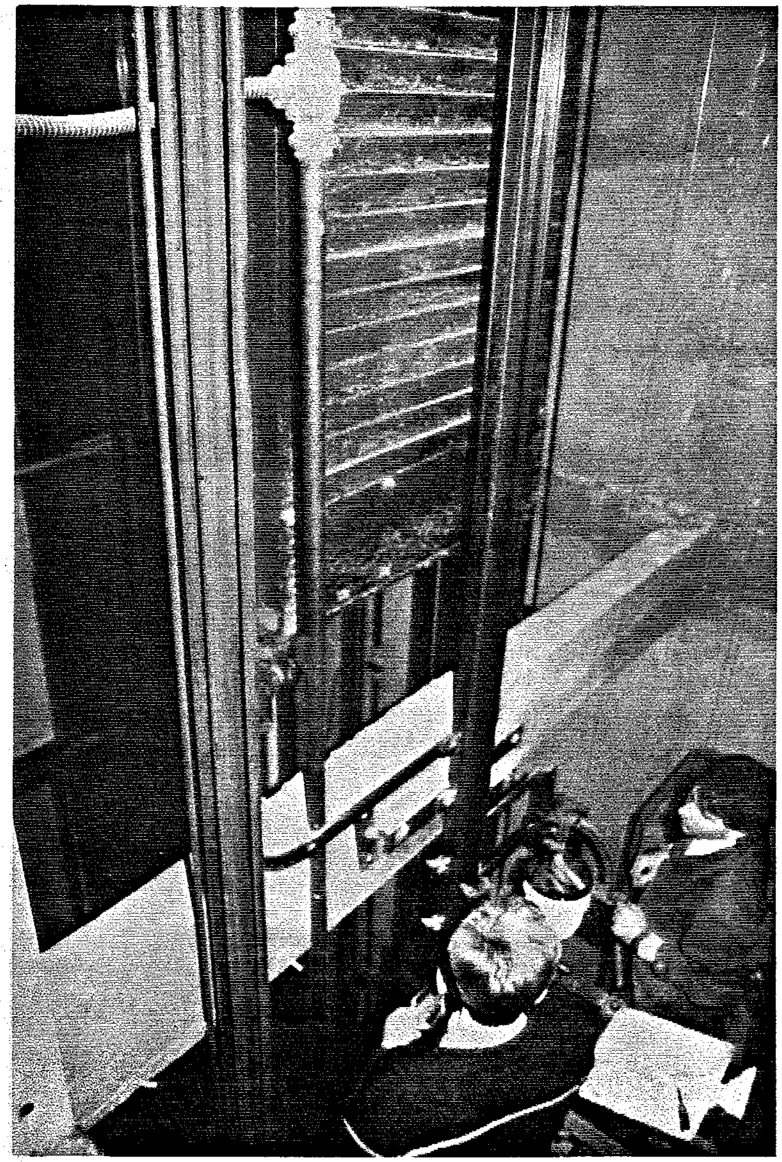


Figure 4.8 Elevator counterweight positioned above reinforced concrete beam.

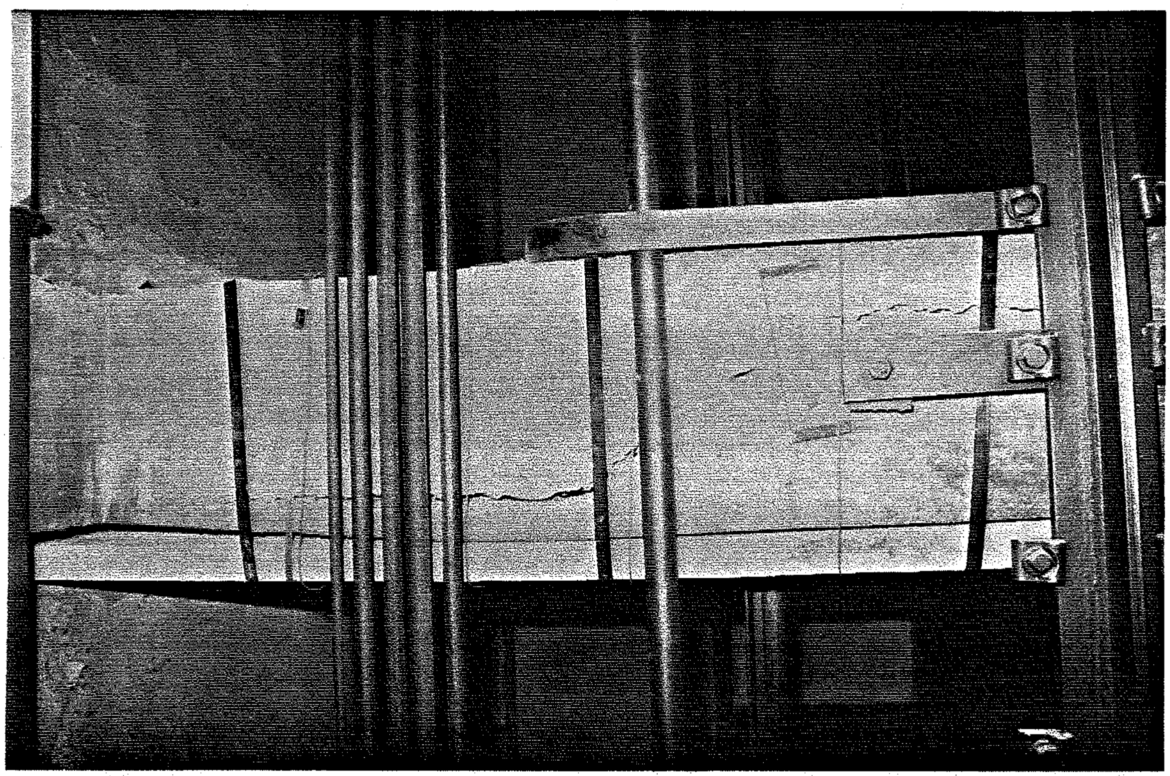


Figure 4.9 Photograph showing shear failure of reinforced concrete beam, counterweight guide support bracket and temporary repairs to beam.

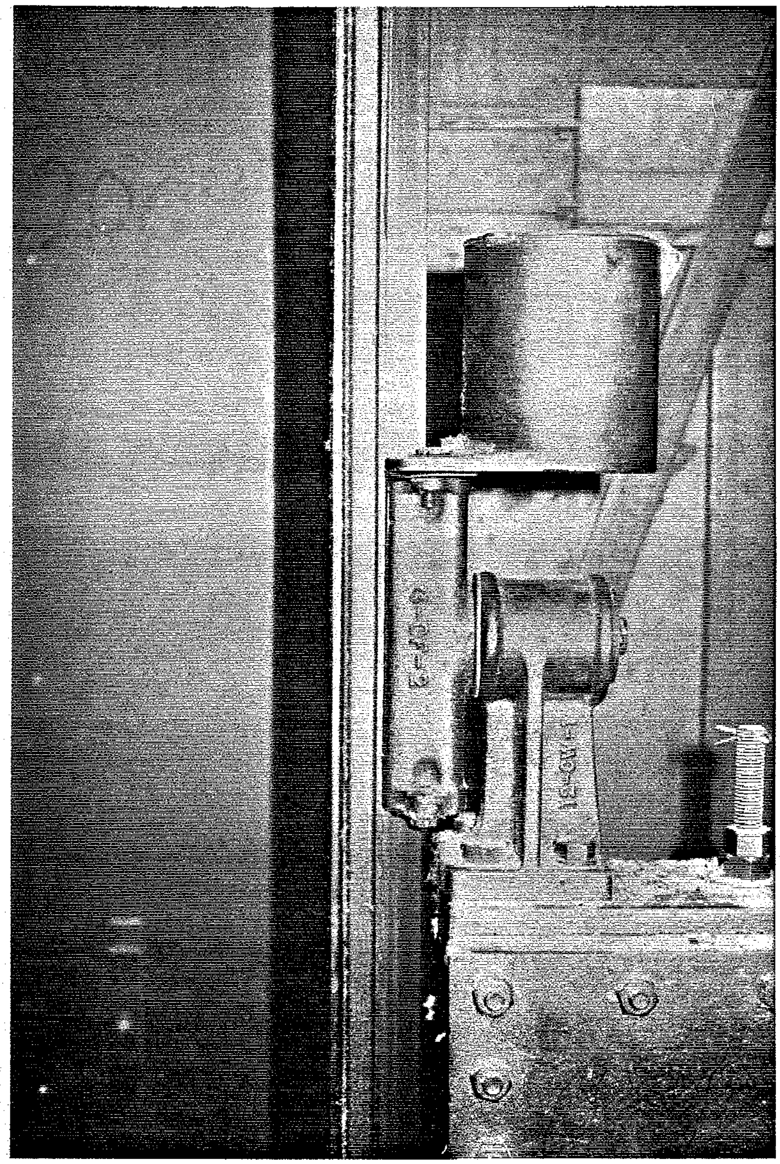


Figure 4.10 View of counterweight guide and guide-rail.

Figure 4.11 Diagonal cracks in masonry wall of Pavillon Notre Dame.

CHAPTER 5

DAMAGE IN MONTREAL

5.1 Description of Montreal

Montreal is a city with a population of about two million people, which is located on an island, with the St. Lawrence River to the south and Rivière de Prairie to the north. The subsoil conditions consist of rock close to the surface, with glacial till overburden. The regions closer to the river banks in some locations have significant depths (over 50 feet) of soft clay and therefore pose a potential risk for amplification of the ground motion.

There were reports of cracking of some plaster walls and isolated incidences of some damage to poorly connected unreinforced masonry cladding on some residential dwellings.

5.2 Shell Oil Refinery

The Shell Oil Refinery, which is located in the east end of Montreal at 10501 Sherbrooke St. East, suffered extremely light damage due to the 1988 Saguenay earthquake. Discussions with plant personnel revealed that some spalling of the concrete cover on a reinforced concrete support for a heat exchanger had occurred as shown in Fig. 5.1. Although there was evidence of a differential thermal movement of about 1/2 in. between the heat exchanger and the support, it was clear from an examination of the spalled surface of the concrete that the spalling had occurred recently and could have been caused by the earthquake.

A distillation tower clad with fire brick suffered from the loss of several bricks during the earthquake as shown in Fig. 5.2. Some emergency warning switches tripped on a compressor during the earthquake but the compressor kept operating. It was interesting to note that some blasting in a nearby rock quarry also resulted in the tripping of the emergency switches on some of the compressors. This blasting, in some instances resulted in the shut-down of some compressors.

5.3 City Hall in Montreal East

The Montreal East City Hall (Hotel de Ville) located at 11370 Notre Dame Est was constructed in 1937. It is a two storey reinforced concrete frame structure with masonry cladding. This structure suffered severe damage to the masonry cladding as shown in Fig. 5.3, 5.4 and 5.5. It is important to realize that this structure is founded on 17 m of clay. The foundation consists of a one foot thick mat located 8 ft. below grade. The structure had a problem with excessive settlements and this problem had been under investigation since 1983. A differential settlement between the front and the back of the structure of over 90 mm had caused excessive cracking in both the masonry cladding and the interior walls before the earthquake occurred. The clay is stiffer near the surface with an average shear strength of 60 to 70 kPa. It is believed that the numerous large trees at the back of the structure resulted in significant moisture loss in the clay and therefore caused a larger settlement at the rear of the structure.

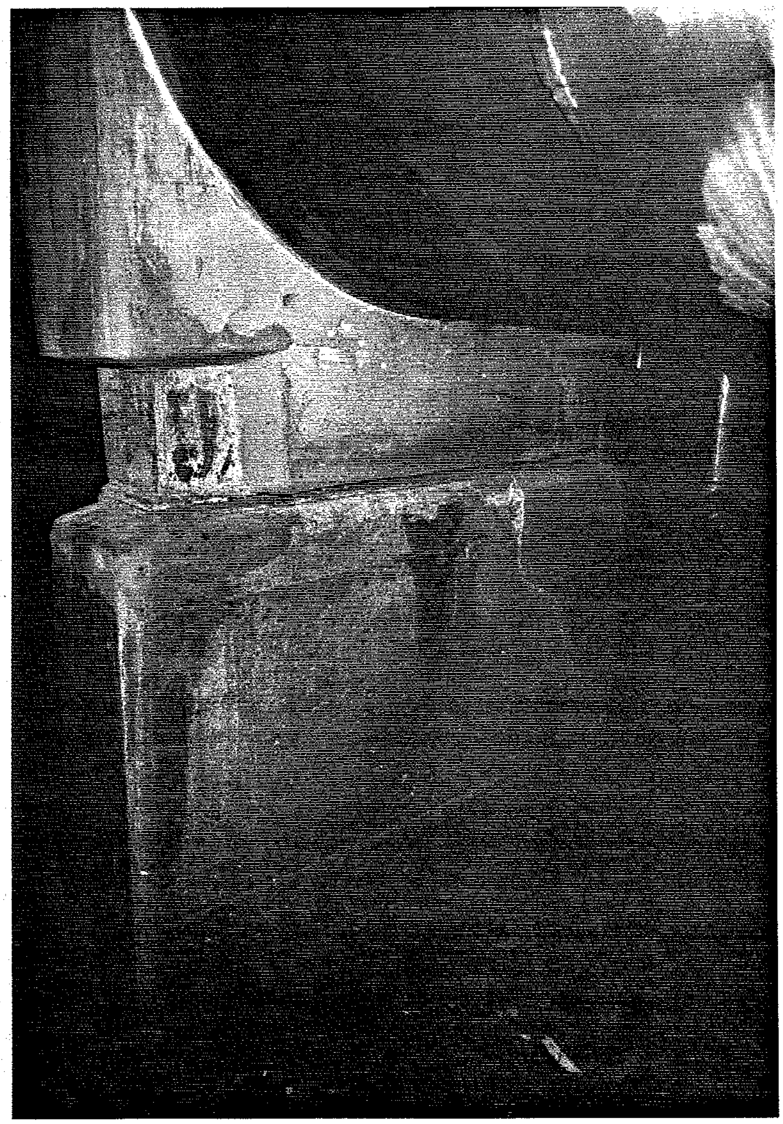


Figure 5.1 Spalling of cover of reinforced concrete support for heat exchanger.

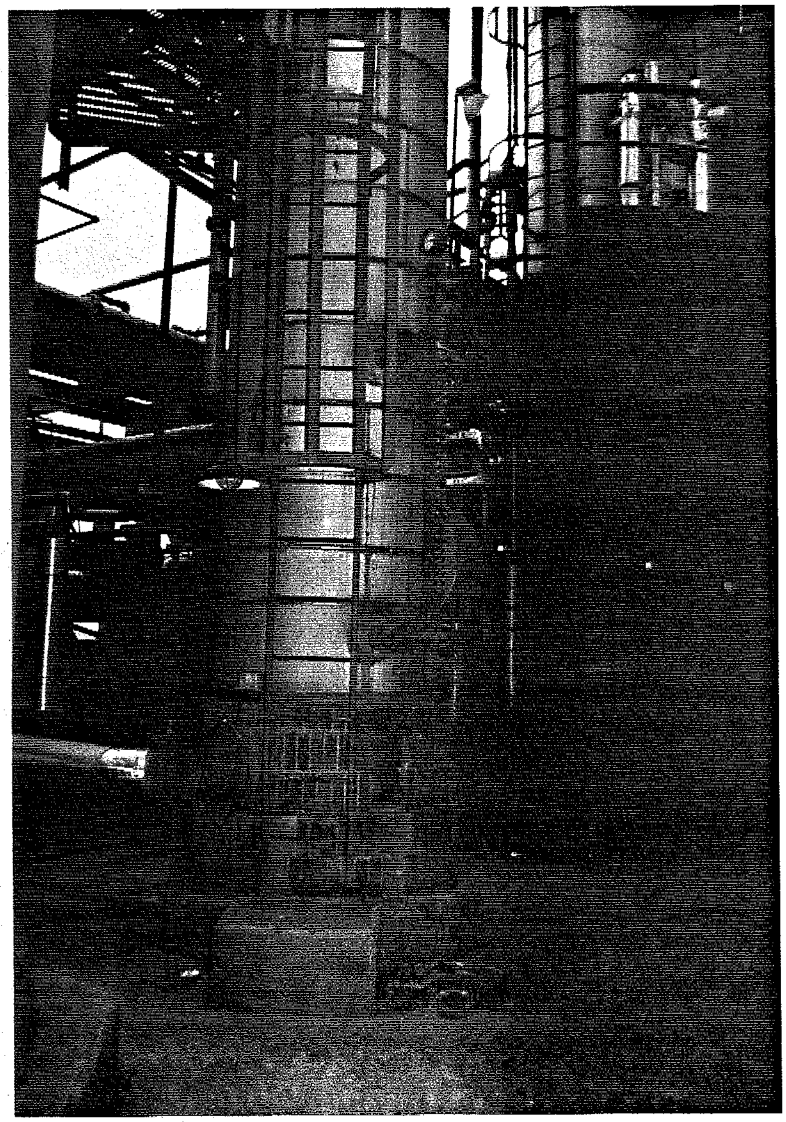


Figure 5.2 Dislodged fire brick from distillation tower.

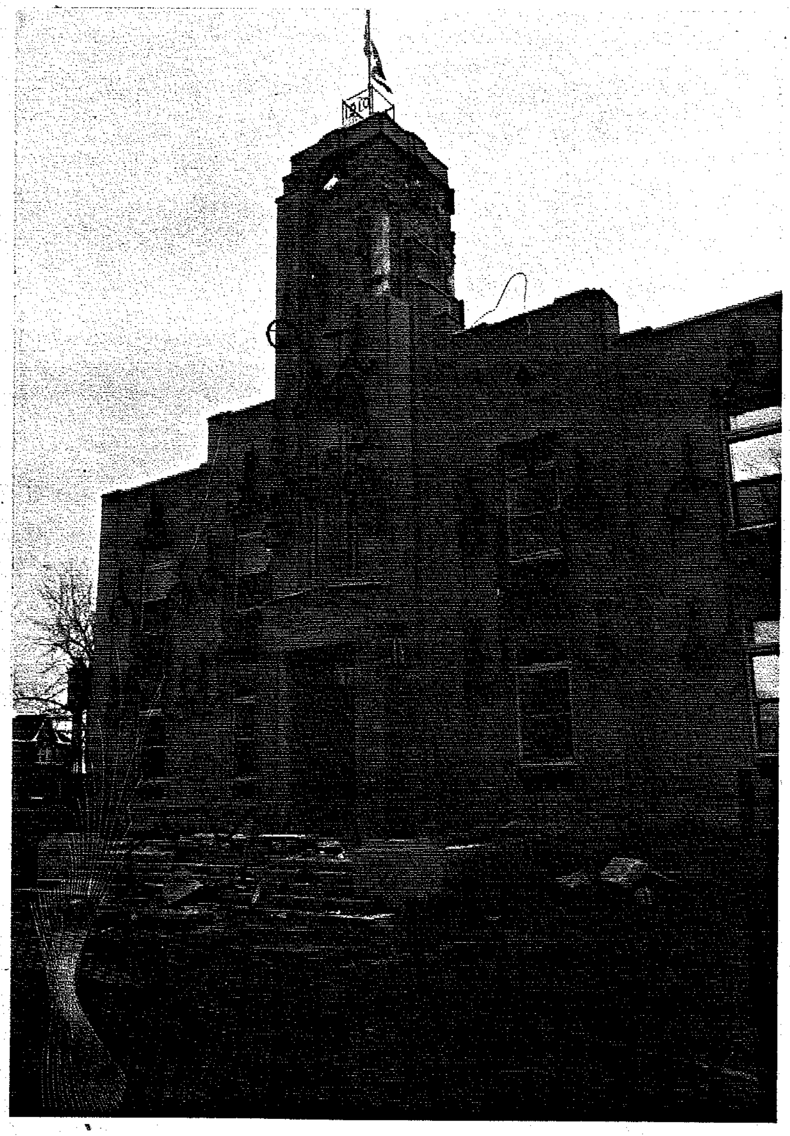


Figure 5.3 Overall view of Montreal East City Hall.

Figure 5.4 Debris from fallen masonry cladding.

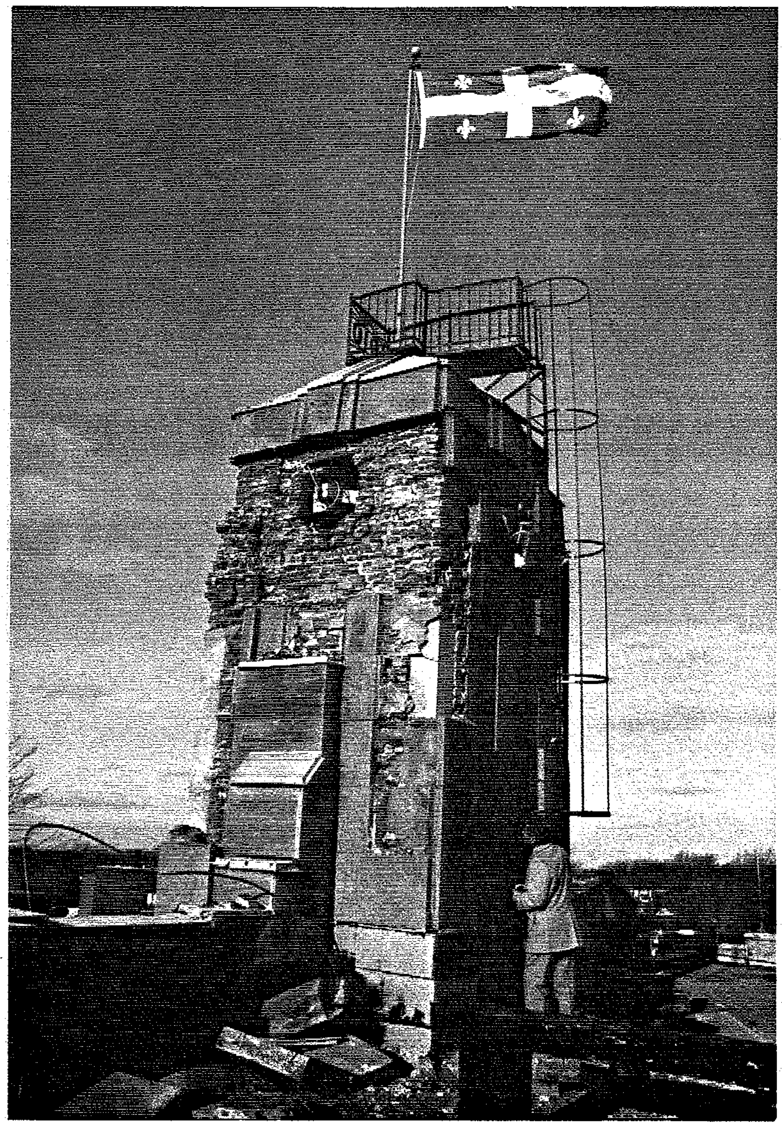


Figure 5.5 Loss of masonry cladding from clock tower.

CHAPTER 6

EMBANKMENT FAILURES

6.1 Highway Embankments

At least 6 highway embankment failures and several large natural slope landslides due to the earthquake were reported. Two highway embankment failures were visited. One of the embankment failures was on Route 175 near Stoneham just north of Quebec City which had been repaired just before the team's visit. This embankment consisted of a granular fill of about 4 m in height with a 1:1 slope. The second embankment failure was located on the shoulder of Route 138 just east of Ste. Anne de Beaupré on a steeply inclined (11% slope) stretch of the highway as shown in Fig. 6.1. The embankment material consisted of medium to fine sand fill and the slope failure took place over a stretch of about 60 m. At the location of the failure the fill was about 24 m high with a slope of 1.5 horizontal to 1.0 vertical. As can be seen from Fig. 6.2 and Fig. 6.3 the failure consists of a relatively shallow slip that did not appear to involve much of the subsoil. As a result of the slide the inside lane of the highway was closed as shown in Fig. 6.4. Liquefaction appeared to be the mechanism for this failure. It was reported that the sand at the toe of the fill flowed out as if it were liquefied. Soil investigations at the site, by the Ministère des Transports du Québec shows that there are some very loose granular layers that are susceptible to liquefaction in an earthquake.

6.2 Railway Embankments

There were two reported failures of railway embankments on the Canadian National railway lines. One failure occurred on an approach embankment to a bridge near Rivière-à- Pierre. This sand and gravel embankment was 25 to 30 feet high with side slopes of about 1.5 horizontal to 1.0 vertical. The embankment slid into the river over a length of about 75 to 80 feet. The embankment was repaired immediately after the earthquake and opened to rail traffic.

The second failure, shown in Fig. 6.5, took place 10 miles north of Hervey Junction. The 50 ft. high embankment consisted of fine to medium sand and therefore was well drained. The original slope of the embankment was 1.5 horizontal to 1.0 vertical. The total length of the slide was 300 ft. which left the rails suspended as shown in Fig. 6.6 and Fig. 6.7. The repair consisted of placing a berm at the bottom of the embankment to a height of about 30 ft. and then constructing a new embankment with a height of about 20 ft. and a new slope of 2.0 horizontal to 1.0 vertical. The railway line was closed for a total period of one week.

In the Lac St. Jean subdivision a number of longitudinal cracks were observed in the sand fill embankments. These longitudinal cracks generally occurred on the shoulder of the embankment along the edge of the railway ties and in most cases were relatively minor requiring no repair. The maximum width of cracks measured was 2.5 in.

Figure 6.1 Location of embankment failure on Highway 138 near Ste. Anne de Beaupré.

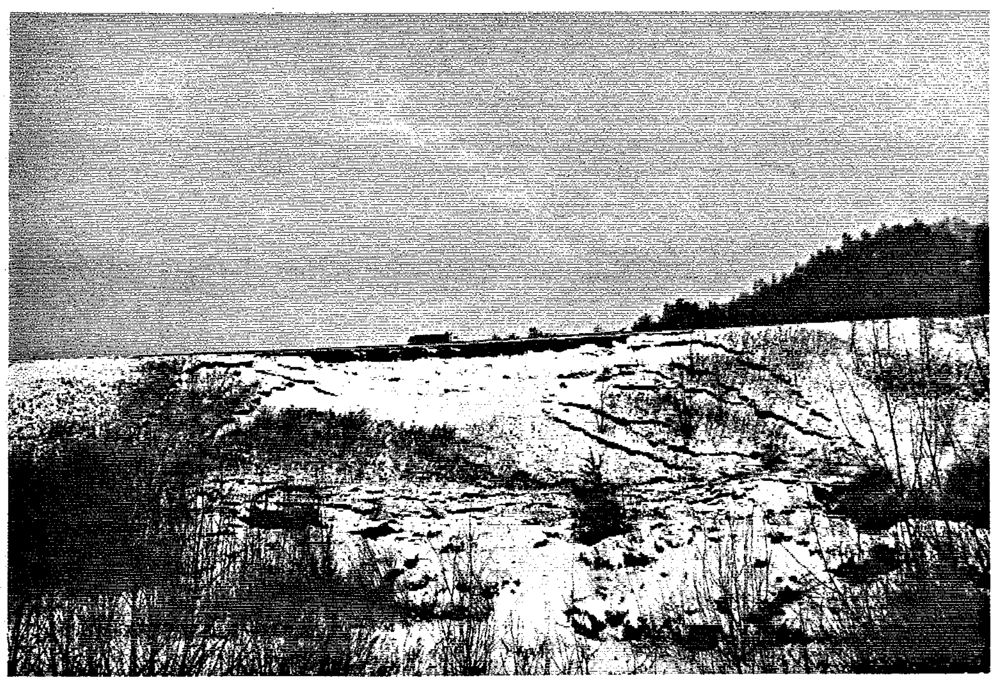


Figure 6.2 Overall view of embankment failure.

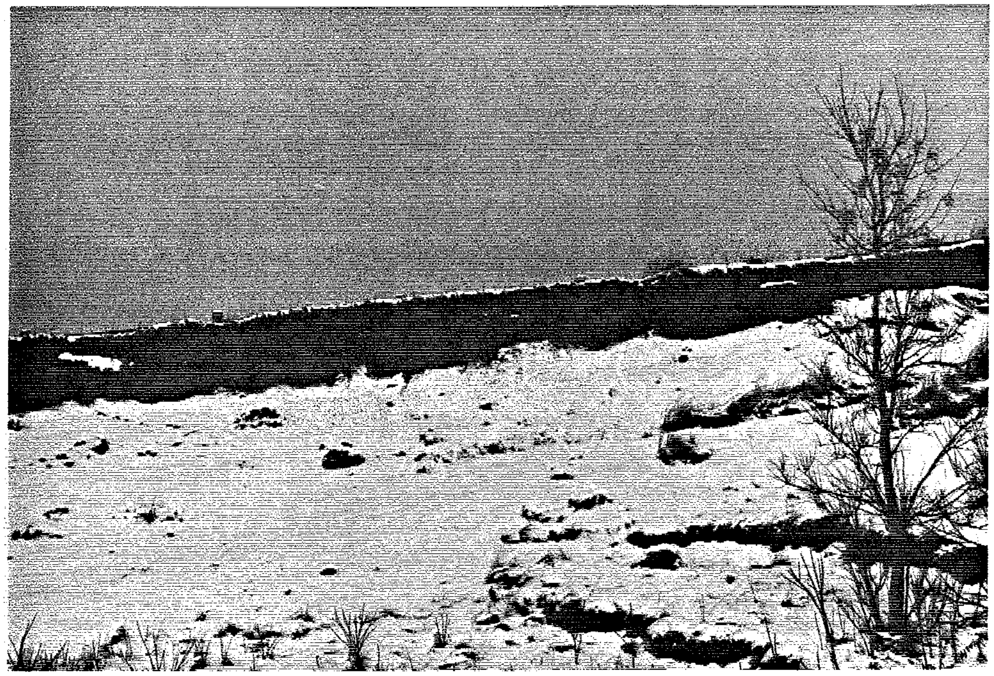


Figure 6.3 Close–up view of embankment failure.

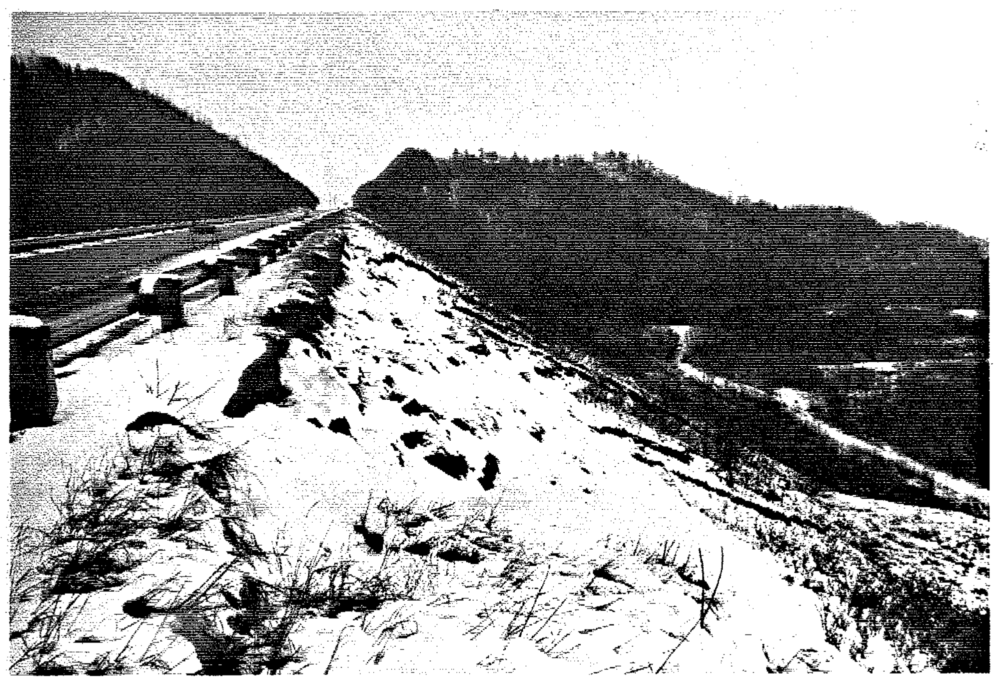


Figure 6.4 View of embankment failure from shoulder of highway.

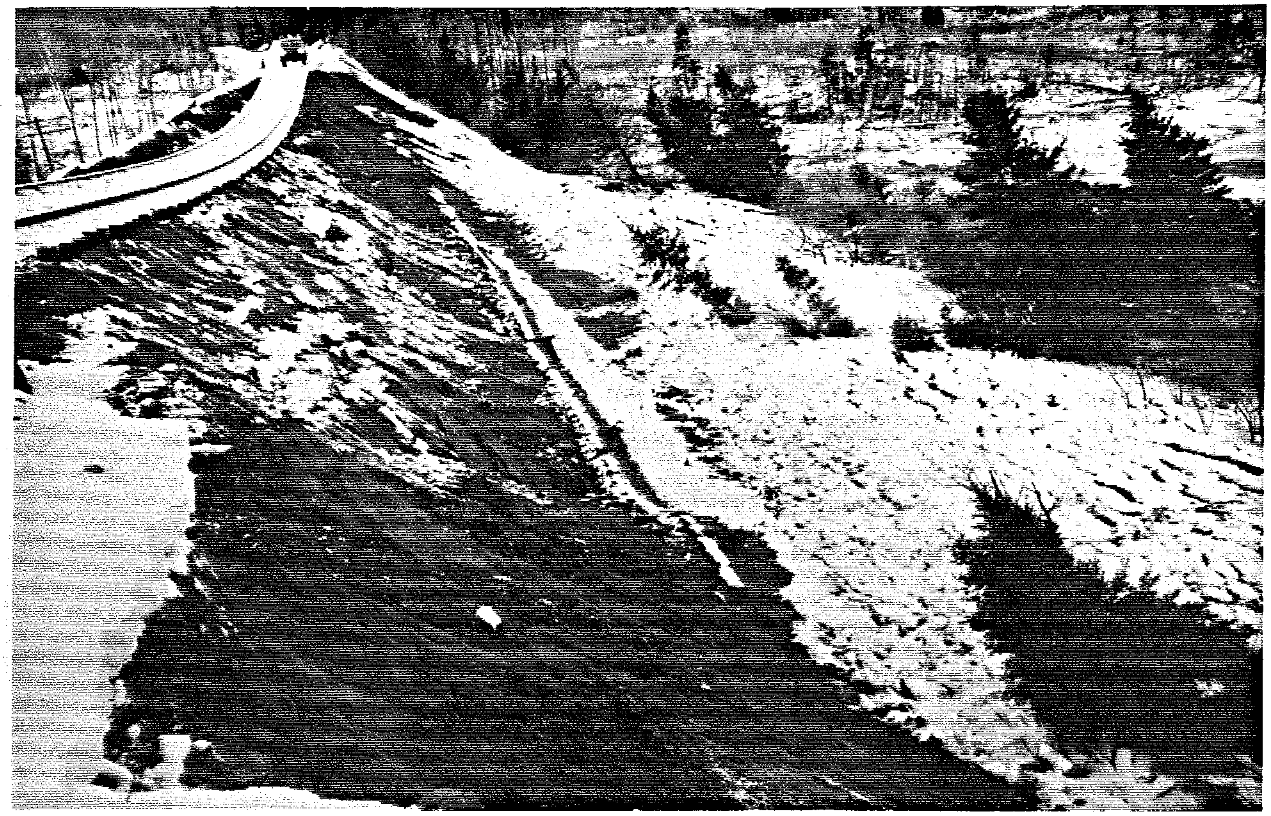


Figure 6.5 Overall view of railway embankment failure, photograph courtesy of Canadian National.

Figure 6.6 Railway embankment failure, photograph courtesy of Canadian National.

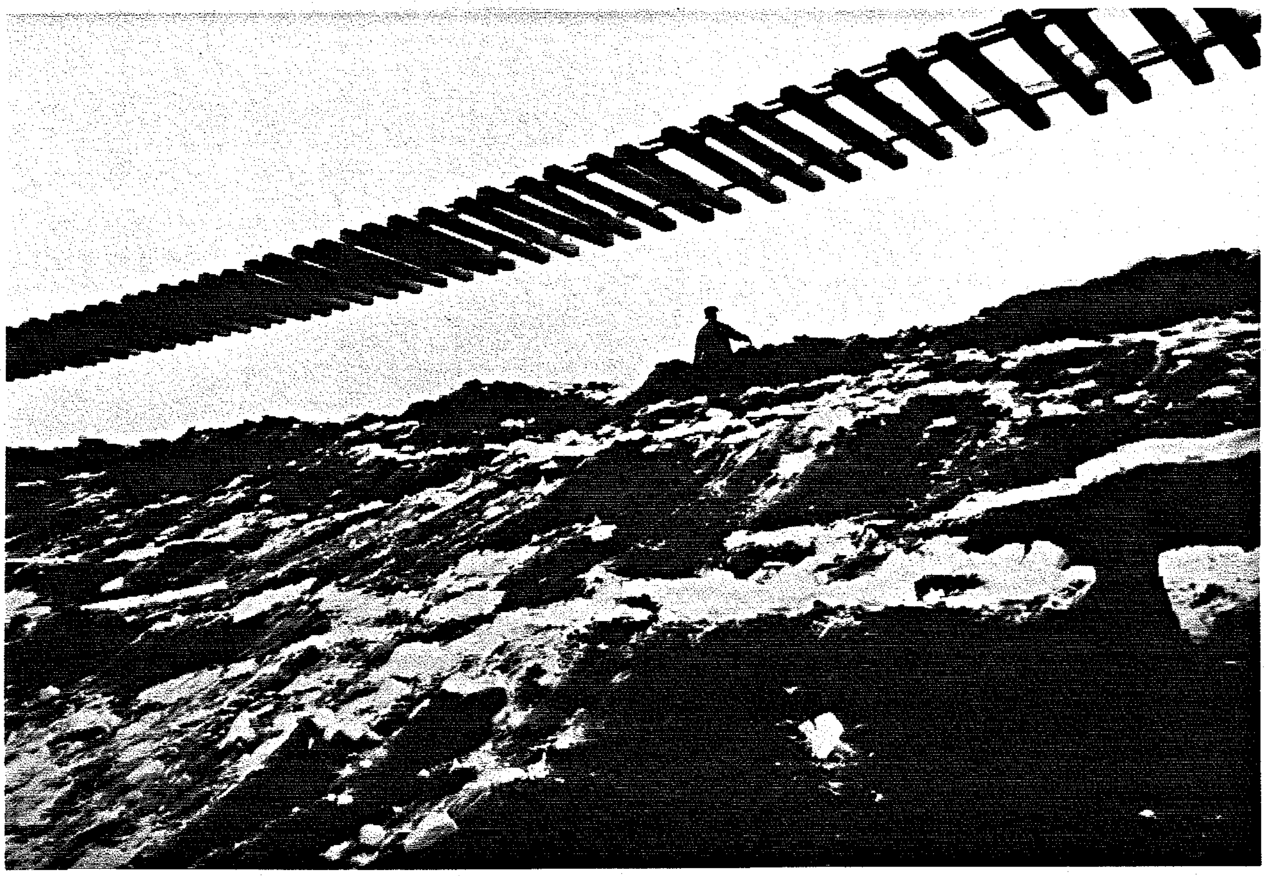


Figure 6.7 Railway embankment failure, photograph courtesy of Canadian National.

CHAPTER 7

SUMMARY AND CONCLUSIONS

7.1 Value of Site-Visits

The Canadian National Committee on Earthquake Engineering should continue to send site-visit teams to investigate earthquakes which are considered to be relevant to Canadian engineers and seismologists. Although this earthquake did not cause major structural damage it was considered essential to investigate every aspect of this event since it is the first, well instrumented, significant event in Eastern Canada.

An important role played by the site-visit team involved the evaluation, from a technical point of view, of all reports by the press concerning the type and degree of damage. Many press reports of damage were exaggerated partly due to the general state of panic of the local residents. Since this earthquake was the largest seismic event in Quebec in the last 50 years it is understandable that the general population was greatly effected by the event.

7.2 Ground Motion

Preliminary reports provided by the Geological Survey of Canada indicated that the magnitude of the earthquake was 6.0 (focal depth of 28 km) and the epicentre was located 36 km south of Chicoutimi. The maximum horizontal acceleration measured in the epicentral region was about 11% g (frequency of about 13 Hertz) and the vertical acceleration was about 7% g (frequency of 18 Hertz).

The largest free-field horizontal acceleration of 15.5% g was recorded at Baie-St-Paul which is 91 km from the epicentre. It is believed that the local soil conditions caused this amplification and also resulted in a decreased frequency (4 Hertz).

In Quebec City, located 150 km south of the epicentre, the measured horizontal and vertical accelerations, on firm ground, were 5.0% g and 2.2% g respectively (with frequencies of 10.5% Hertz and 10.0% Hertz respectively).

At the time of writing this report the complete seismological data had not yet been processed. The Geological Survey of Canada will be presenting acceleration-time histories and spectra for this event at a later date.

7.3 Landslides and Embankment Failures

At least 6 highway embankment failures and a number of natural slope landslides occurred. In addition there were two reported failures of railway embankments on the Canadian National railway lines. The largest highway embankment failure (60 m long failure) occurred near Ste. Anne de Beaupré and the largest railway embankment failure (300 ft. long failure) occurred near Hervey Junction.

7.4 Influence of Soil Conditions

As observed in past earthquakes, regions in Quebec City where damage occurred to buildings, coincided with the soft soil areas. It is believed that the soft soil resulted in an amplification of the acceleration levels together with lengthening of the ground motion period. Soil amplification during the earthquake combined with differential settlement before the earthquake caused damage to the Montreal East city hall. As noted in past earthquakes (e.g., Mexico City), large settlements due to poor soil conditions are often a precursor to structural damage due to ground motion amplification.

7.5 Performance of Different Structural Systems

The nature of the ground motion and the location of the epicentre did not cause loss of life or severe structural damage. However, this earthquake provided an excellent opportunity to investigate the "serviceability performance" and permitted the site-visit team to assess the likelihood of damage in the event of stronger ground motion.

The poor performance of a structure with "tension-only" cross bracing and structures containing "soft-storeys" was demonstrated in this earthquake.

The poor performance of unreinforced masonry was clearly demonstrated in the 1988 Saguenay earthquake. Shear cracking in the plane of the unreinforced masonry walls as well as flexural failures in walls about their weak axes were common occurrences. If the ground motion had been stronger more severe damage and loss of life would most probably have occurred. The abundance of unreinforced masonry, even in important buildings such as schools and hospitals, poses one of the greatest potential hazards in the event of a larger earthquake.

The site-visit team urges the Canadian National Committee on Earthquake Engineering to attempt to address the problem of evaluating and upgrading of existing buildings for seismic resistance.

7.6 Post-Disaster Preparedness

The lack of public awareness with regard to earthquakes and their effects was evident throughout the province. This is due to the lack of government programmes to educate the general public on the nature and potential hazards of earthquakes. Seismically prone areas in Canada should adopt educational programmes similar to those that exist in many other countries with earthquakes.

The site-visit team strongly urges all levels of government (federal, provincial and municipal) to assess the adequacy of their civil protection agencies in the light of this very valuable experience.

7.7 Follow-Up Work required

Although preliminary information on the strong ground motion was available at the time this report was written, complete information on the ground motion, including acceleration time-histories and response spectra will be made available by the Geological

Survey of Canada. The availability of this data will enable a more complete assessment of the damage that resulted.

At the time this report was written internal reports of industrial facilities and government agencies concerning the earthquake damage were not yet released. As part of the follow-up work the site-visit team intends to report at a later date on the following items:

(1) Damage to industrial facilities in the epicentral region

(2) Measured responses of concrete and earth-filled dams

(3) Investigation of occurrence of liquefaction

(4) Causes and extent of power failures

(5) Assessment of total cost of damage throughout the province

(6) Breakdown of costs of damage into different categories

(7) Assessment of structural repairs

(8) Assessment of stability of embankments and natural slopes