

***MASONRY STRUCTURES
AND THE
EDMONTON TORNADO***

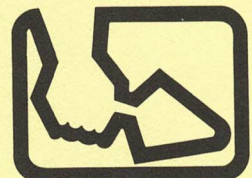
by

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INTRODUCTION

This paper reviews the damage suffered by masonry structures as a result of the tornado which went through the southeast part of the City of Edmonton, Alberta, Canada, during the summer of 1987. A total of 32 people lost their lives. Two of these fatalities are known to have resulted from the collapse of a concrete block wall. The damage to real estate properties is expected to be over 350 million Canadian dollars.

Because of the strength of the tornado no structure designed in accordance with the existing codes could experience this type of loading without collapsing or suffering extensive damage. However, some of the structures could have demonstrated a more ductile behaviour if proper detailing and proper construction procedures were followed.

This paper examines a number of structures which were damaged extensively by the tornado and discusses the failure modes and sequences of collapse. It also reviews the loading of structures resulting from wind and provides some guidelines for increasing the ductility of masonry structures.

THE TORNADO

The catastrophic tornado approached the City of Edmonton from the south travelling along the corridor shown in Figure 1. Photo 1 shows the tornado funnel as it travelled through the city. It is reported that the wind velocity reached by the tornado exceeded 330 kilometers per hour. The classification of this tornado is that of type F4 which is the most severe rating. The rating of tornados is as follows: F0 rated tornado is one with air speeds from 64 to 116 km/hour and causes light damage, F1 causes moderate

damage and reaches wind speeds from 117 to 180, F2 tornado is expected to cause considerable damage with wind travelling from 181 to 252 km/hour, F3 causes severe damage with wind speeds from 253 to 330, and finally, F4 tornado causes devastation of anything in its path with winds travelling with velocities from 331 to 417 km/hour. As it traversed through the outskirts of the city the tornado levelled everything under its path. Trees were cut like toothpicks or were uprooted. A trailer park, where most of the casualties occurred was levelled. Metal siding, wood pieces, debris and other construction materials were flying around, caused fatalities, and were eventually embedded in walls of buildings or went through parked vehicles.

The destruction was complete and dramatic. Photo 2 shows the complete destruction of a steel structure, and Photo 3 shows a similar condition for a masonry building. Although all types of structures in the path of the tornado were destroyed the photos in this paper concentrate on the damage to masonry structures.

WIND INDUCED LOADING ON LOW RISE STRUCTURES

Where wind interacts with a building it introduces loads on the building. The wind induced loads fall into two categories, namely, pressure and suction. Windward walls experience pressures, leeward and side walls experience suction. Flat roofs or roofs with small slopes experience uplift pressure. Where wind is forced around corners it induces large suction forces. If doors or windows fail under the action of wind pressure the whole structure is loaded from within. The types of loads resulting from the conversion of the kinetic energy of the wind to pressures or from vacuum generated by the deflection of the air flow, are shown schematically in Figures 2 through to 5. The response of the structure to these types of

loading conditions is affected by the capabilities of the wall, and by its interaction with and performance of the other structural elements. If, for example, the roof connection fails and it separates from the wall (Fig. 3), the wall acts as a cantilever and collapses inwards. The behaviour of walls such as shown in Figure 2 will depend on the layout of the structure as it relates to roof diaphragm action and unsupported height of the wall. Structures with small (short) walls and stiff and heavy roofs will deform as shown in Figure 2a. However, structures with tall walls and long roofs will tend to deform as shown in Figure 2b.

REVIEW OF DAMAGED STRUCTURES

A. Roof to Wall Connections

The structure shown in Photo 4 suffered extensive damage. Masonry walls collapsed outward indicating that external suction and/or internal pressure was the prime loading mechanism. It is surmised that the sequence of events leading to failure was as follows: uplift pressure on the roof and shear force at the connection of the steel joist to the walls caused the connection to fail and the free standing wall collapsed outwards. Note that the bond beam is attached to the roof and that no vertical reinforcement is evident in this photo.

Portions of three different buildings collapsed because the bearing plates and connecting studs under the ends of the open-web steel joints were pulled out laterally from the bond beam. In all three cases, the studs under the bearing plates had been placed just inside the face shell of the block. Under lateral loads the face shell broke away and the studs were pulled out of the bond beams with little, if any, damage to the grouted core.

Bond beams typically contain one or two longitudinal bars intended primarily to provide for diaphragm action, and control of cracking. They are not designed to surround and contain the studs and in that way create a proper connection. Although the drawings of this building are not available it is doubtful that the placement of the bearing plates and studs received any special attention. Photos 5 and 6 illustrate the type of failure discussed.

Inadequate anchorage of bond beams to walls caused the collapse of portions of four buildings and was evident in many others. Vertical bars in the wall are at times terminated at the joint below the bond beam, especially when a section through the walls shows the beams to be constructed from channel blocks. Photos 7 and 8 show separation of bond beams from the wall, and deflection of the assembly. From these photos the lack of reinforcement anchored into the bond beams, is very noticeable. In one case (Photo 8) a windward wall was shifted about 75 mm under a bond beam which was restrained by the roof diaphragm. This particular wall did not collapse, but it came uncomfortably close. Photo 9 shows the failure of the corner of a structure subjected to a mechanism schematically illustrated in Figure 5. Photo 10 shows uplift of a roof subjected to a mode shown in Figure 3.

Previous and current construction practices in Edmonton are typical of those in other parts of North America; there are a great many existing buildings whose roofs are not positively anchored. It is unlikely that these buildings would be able to resist any abnormal loading. Structures with improper connection of the vertical reinforcement into the top bond beam are very susceptible to damage even from length changes and rotation of the joists or beams attached to them. Any kind of loading will result in separation of the bond beam from the wall.

B. Wall to Wall Connection

Partition walls and intersecting walls that failed are shown in Photos 11 and 12. Infill panels failed because of improper connection to the steel column. Photo 13 shows an intersecting internal wall connected by means of a 28 gauge corrugated strip mechanically fastened in the mortar joint by means of a 12 mm concrete nail, the size of which is shown in Photo 14.

The walls discussed above were connected to the front of the structure by means of a piece of plywood on which corrugated strip ties were attached and then embedded into the mortar joint. The plywood in turn was nailed by standard 40 mm nails to a wood panel. This connection detail probably failed prior to the failure of the roof-wall connection.

At the rear of the same structure an exterior wall failed by complete separation at the connection to the intersecting masonry wall. The interior wall was connected by means of 28 gauge corrugated strip ties embedded in the mortar joint and fastened by means of a 12 mm concrete nail driven into the mortar joint of the rear wall. The connector was similar to that shown in Photo 14. Clearly, this type of connection is grossly inadequate.

C. Brick Veneers

Brick veneer walls attached to the back-up system by means of corrugated strip ties behaved admirably well. There were many structures where the brick veneer survived the extreme loads; other cladding flew off the wall assemblies. Pressure equalization due to presence of weep holes is likely the main contributor to their good performance.

DESIGN CONSIDERATIONS FOR WIND INDUCED LOADS

The loads resulting from wind can cause localized or complete failures of infill panels or load bearing walls. Plain masonry is very susceptible to wind induced damage especially when supporting light vertical loads. The relatively low tensile strength of masonry necessitates that wind induced loads be carefully evaluated and that proper steps are taken to safeguard against overstressing. Because of the extremely high loads encountered by the structures during the Edmonton tornado little could have been done to save the structures from collapse, nevertheless more ductile behaviour might have been experienced had the structures incorporated better anchoring and better connections in their makeup. Wind induced uplift on roof structures causes separation of the bond beam from the masonry wall. Figures 6 and 7 show lateral support reinforcement which with proper anchoring of the top and bottom of the wall can provide for better performance, without increasing the cost of the walls significantly. Failure of openings such as doors can result in high pressures on internal walls with catastrophic results. Interior walls must be properly supported and connected to intersecting walls through the use of at least corrugated strip ties or preferably metal anchors in the mortar joints of both walls.

Placing a 15 mm reinforcing rod in walls every 3000 to 3500 mm must be considered as a minimum for ductility requirements. Partially reinforced masonry must replace plain masonry in the construction of structures. This type of construction is now permitted by CSA Standard S304-M84 "Masonry Design for Structures". The type of failure described under roof to wall connection have been noted by other investigators (1,2,3). The minimum requirements for uplift protection should be as shown in Figures 6 and 7 which have been taken from CSA Standard A370-M84 "Connectors for Masonry".

Structures designed and built in accordance with Part 4 of the National Building Code of Canada will reach their ultimate lateral load capacity when acted upon by winds at a velocity of approximately 200 km/hour. Winds of this magnitude are not very common in Canada and provided that structures are designed to withstand internal pressures (assuming that large doors or windows will fail), the requirements are considered satisfactory.

When designing for uplift, reduction of the probable dead load by 20% is recommended. For masonry structures designed and constructed in accordance with the engineered portion of CSA Standard CAN3-S304-M84 "Masonry Design for Buildings" (4) the behaviour under abnormal load can be predicted. However, using the empirical part of the above noted standard can lead to unsatisfactory performance; its use should be limited to structures of secondary importance.

SUMMARY

The performance of masonry structures subjected to tornado wind loadings during the summer of 1987 is discussed.

Damage from tornado wind loads that resulted either in partial or full collapse of structures was shown to result from lack of attention to connection details. Specifically, roof to wall connections and wall to wall connections were areas of definite weakness.

Recommendations are presented for improvement of ductility of masonry structures under abnormal loadings. In summary it is recommended that minimum reinforcement be provided in masonry walls and that careful attention be paid to connection details.

ACKNOWLEDGMENT

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4. CSA Standard A370M-84 "Connectors for Masonry", Canadian Standards Association, Rexdale, Ontario. 1984.
5. CSA Standard CAN3-S304-M84 "Masonry Design and Construction for Buildings." Canadian Standards Association, Rexdale, Ontario. 1984.

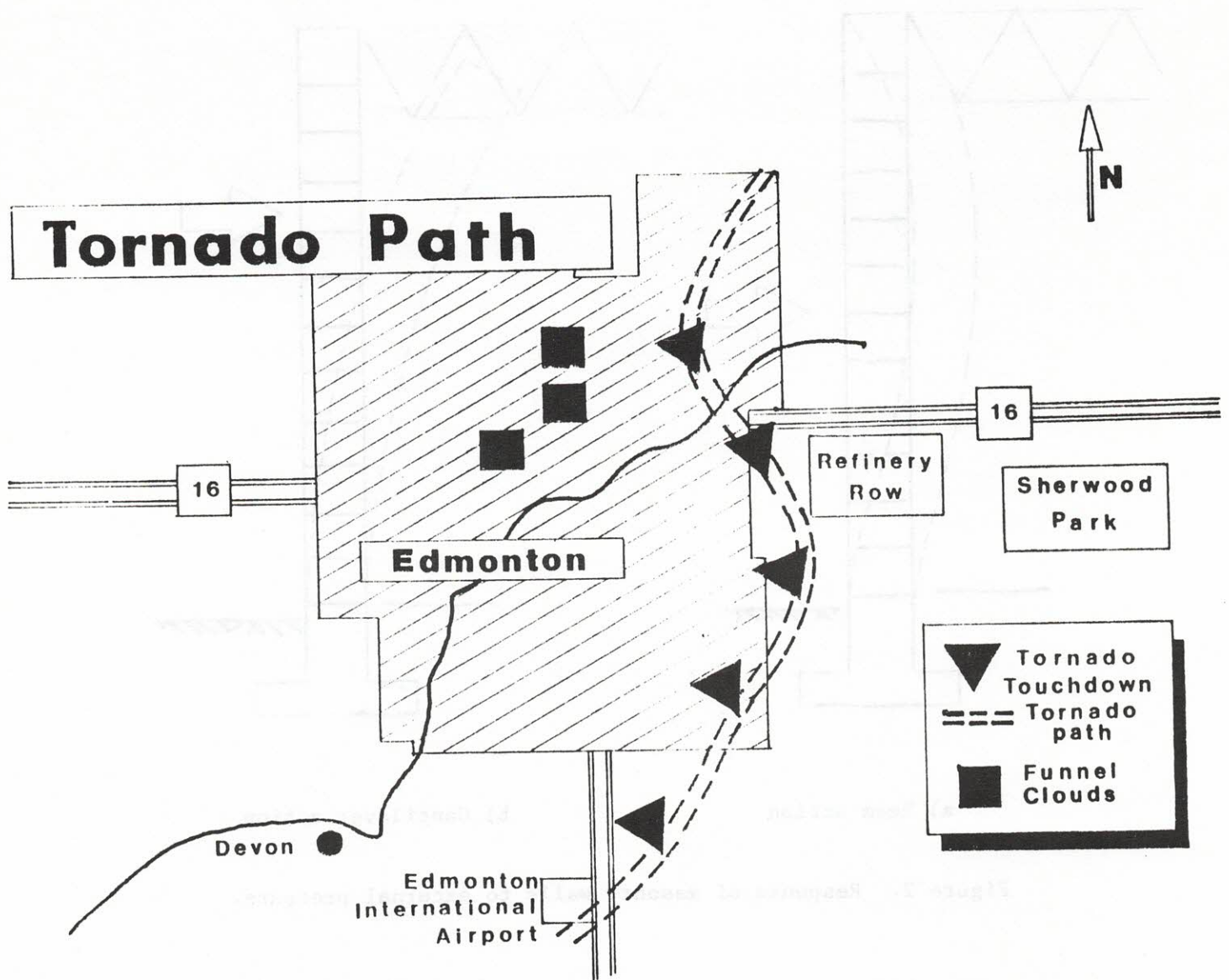


Figure 1. Tornado path.

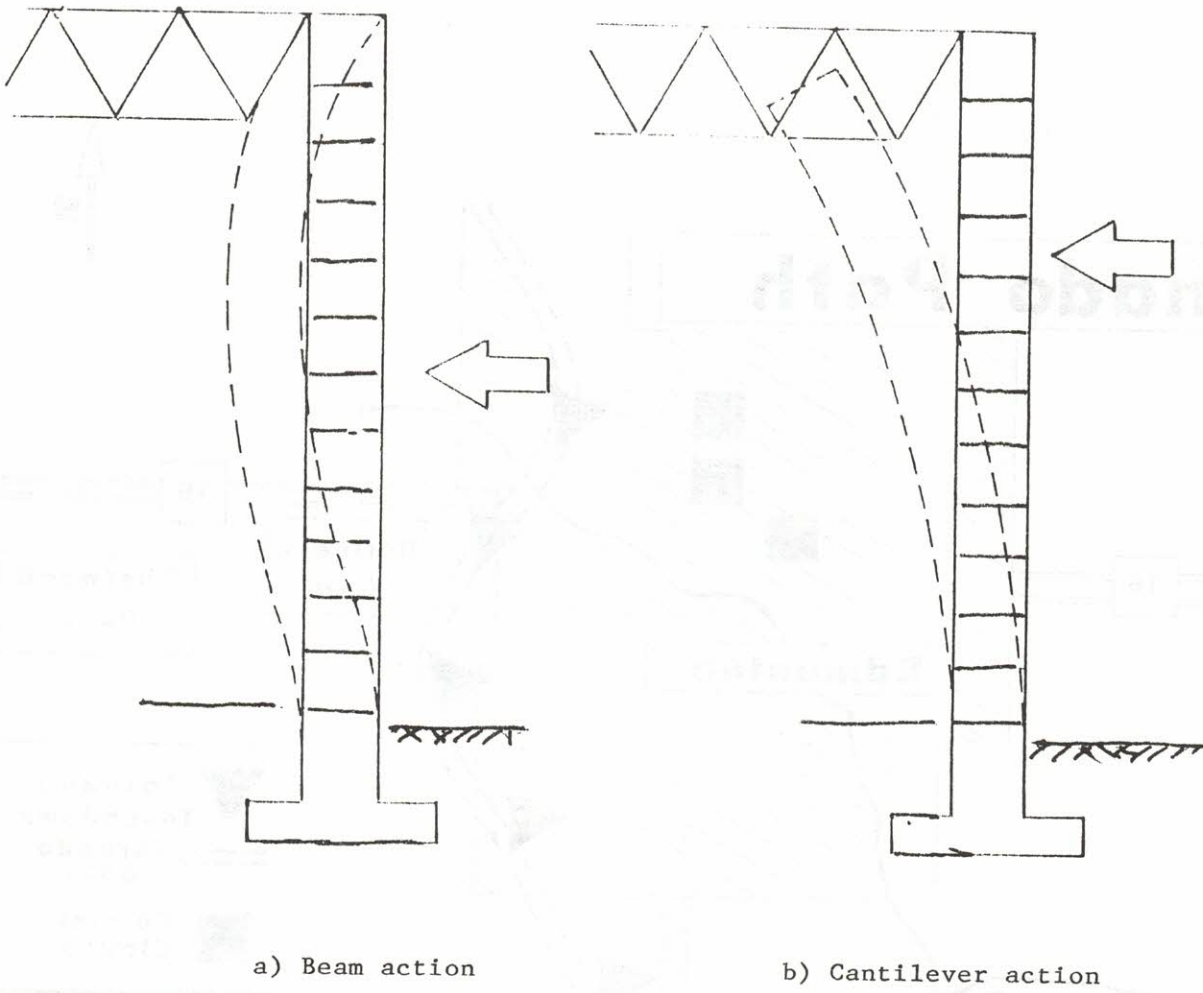


Figure 2. Response of masonry walls to external pressure.

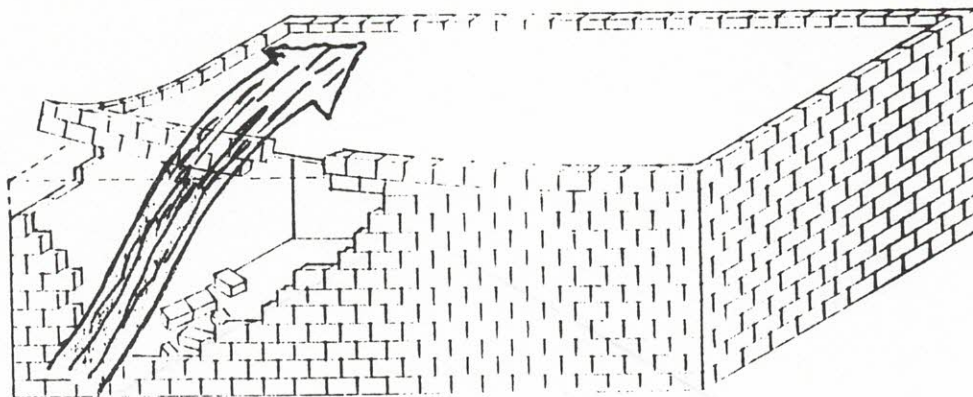


Figure 3. Uplift of roof due to suction.

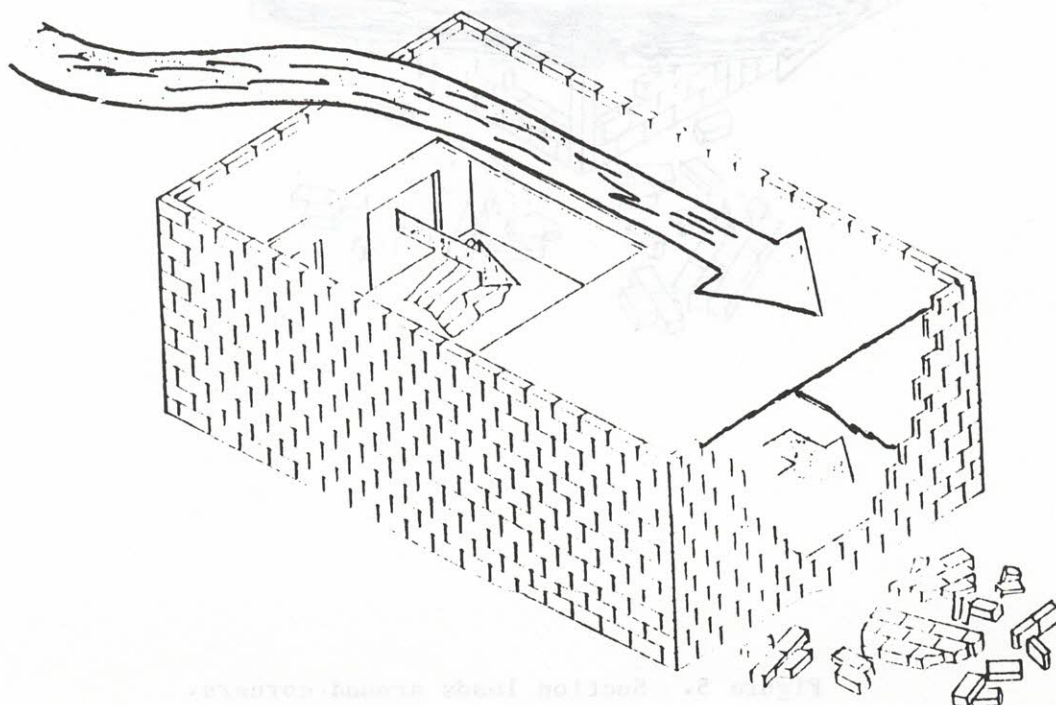


Figure 4. Internal pressure on walls after door collapsed.

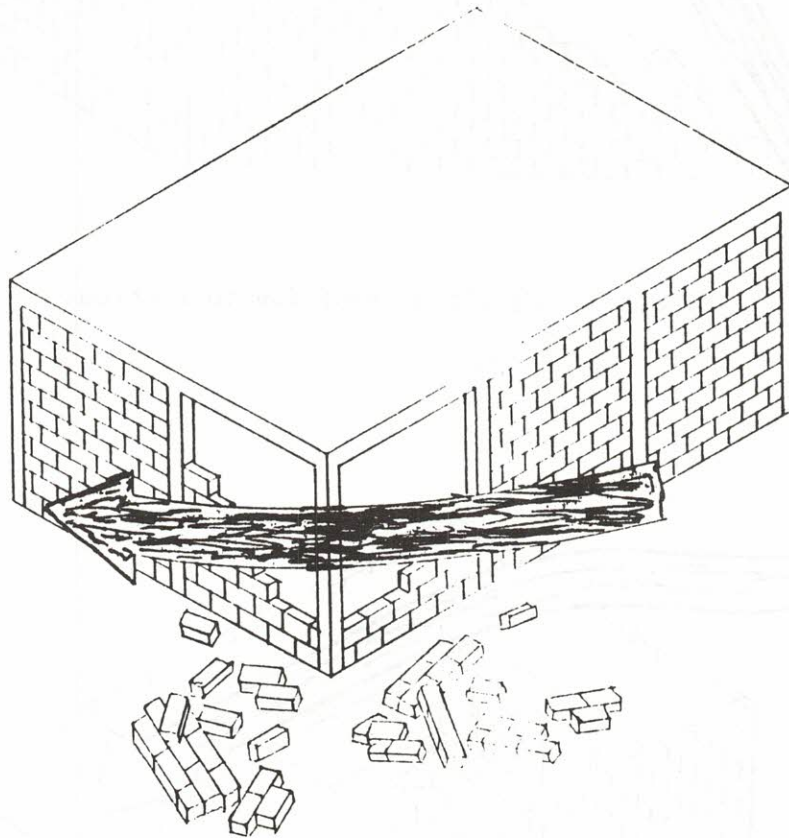


Figure 5. Suction loads around corners.

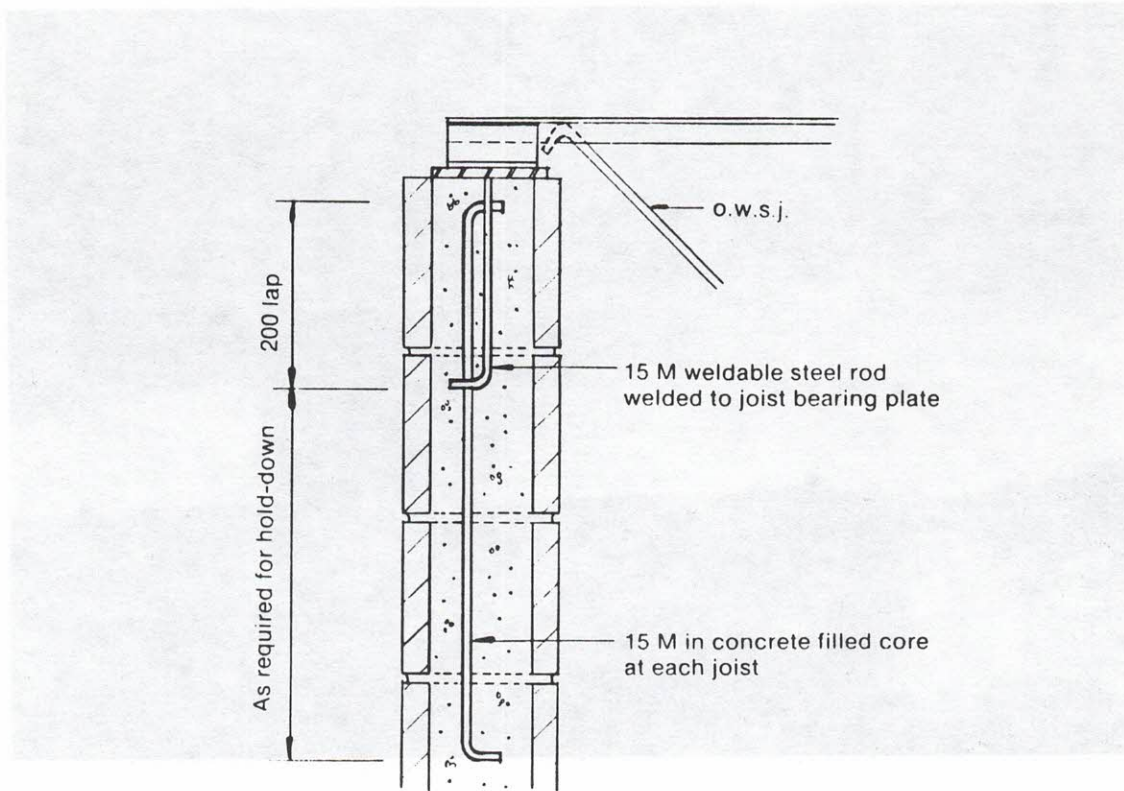


Figure 6. Properly anchored joist.

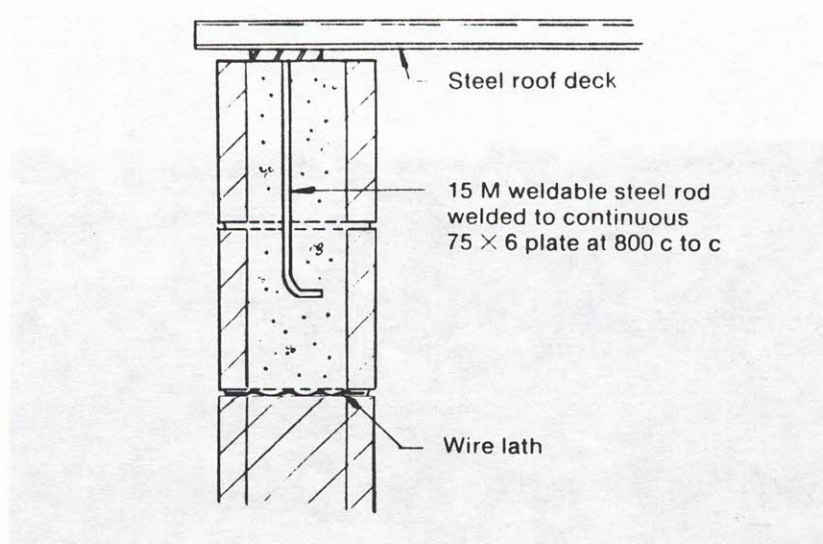


Figure 7. Properly anchored steel roof deck.

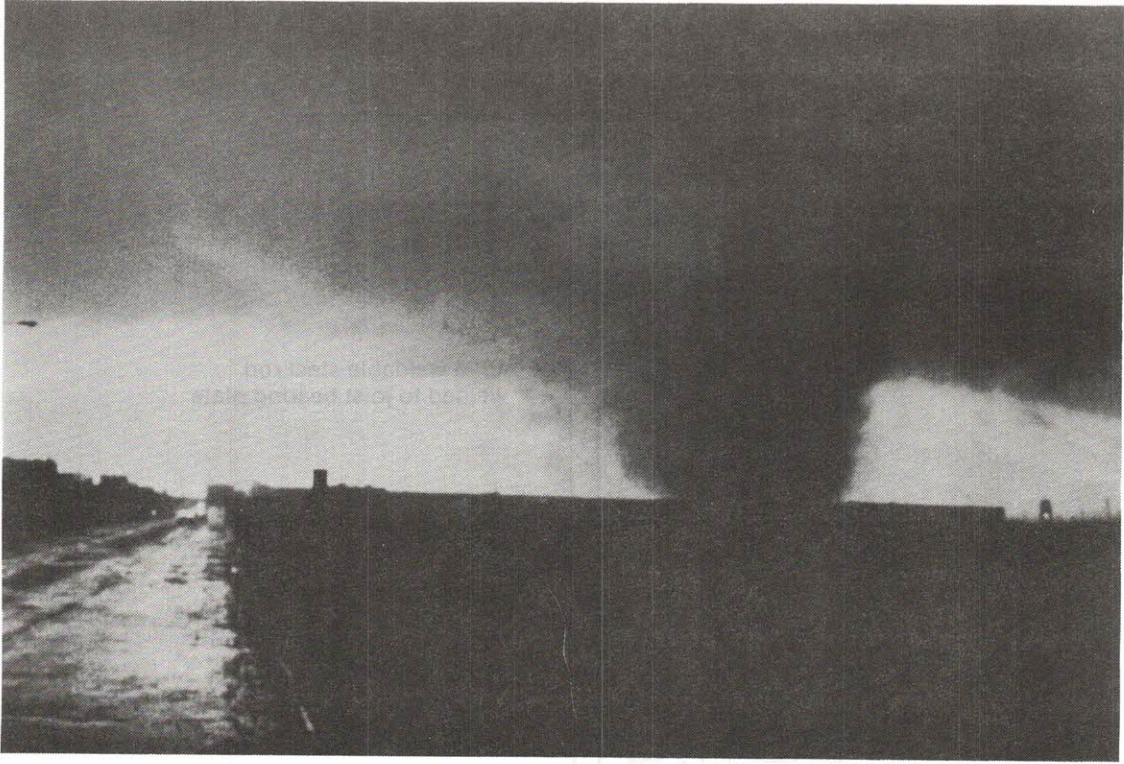


Photo 1. View of the tornado as it traversed through the outskirts of the City of Edmonton.



Photo 2. Complete destruction of a steel structure.



Photo 3. Totally destroyed masonry structure.



Photo 4. Failure of masonry wall under suction.

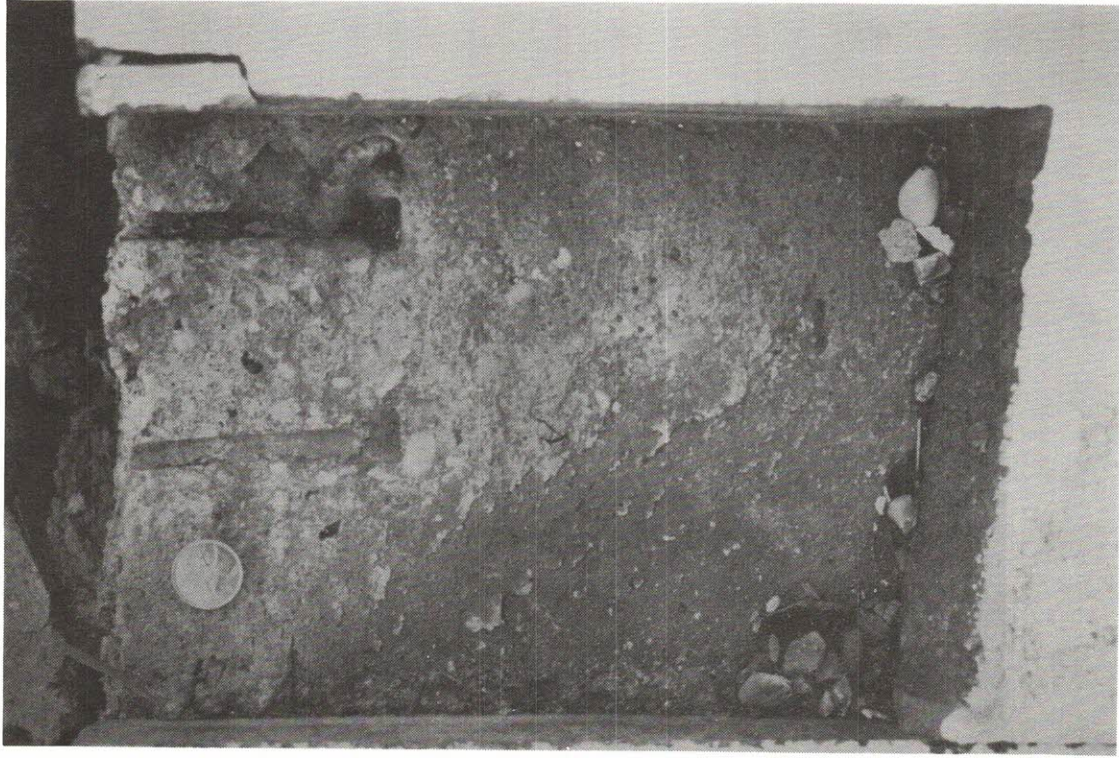


Photo 6. Location of studs within the bond beam.

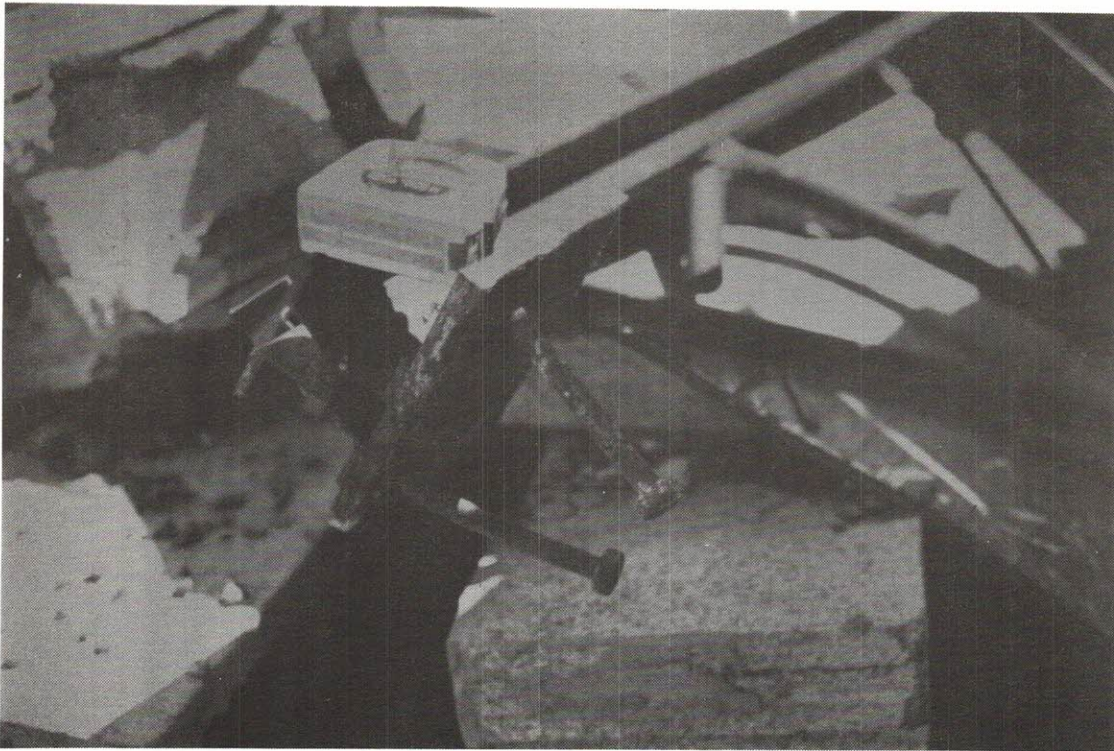


Photo 5. Open web steel joist and connection plate.

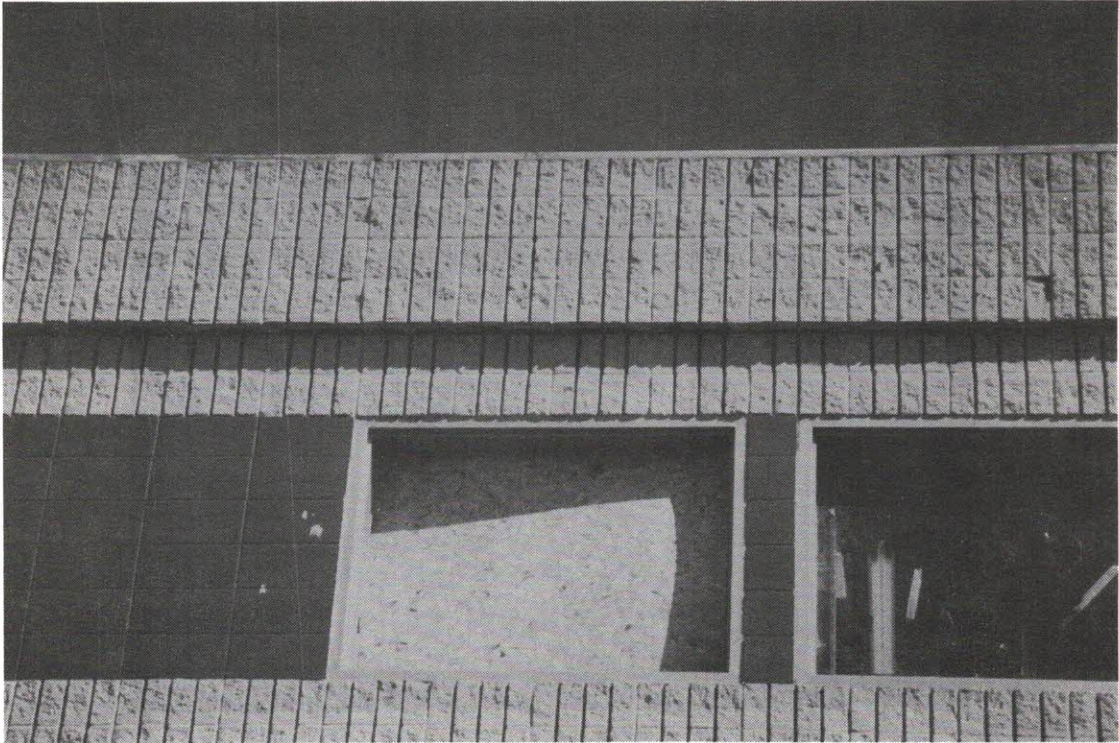


Photo 7. Separation of bond beams from the rest of the wall.

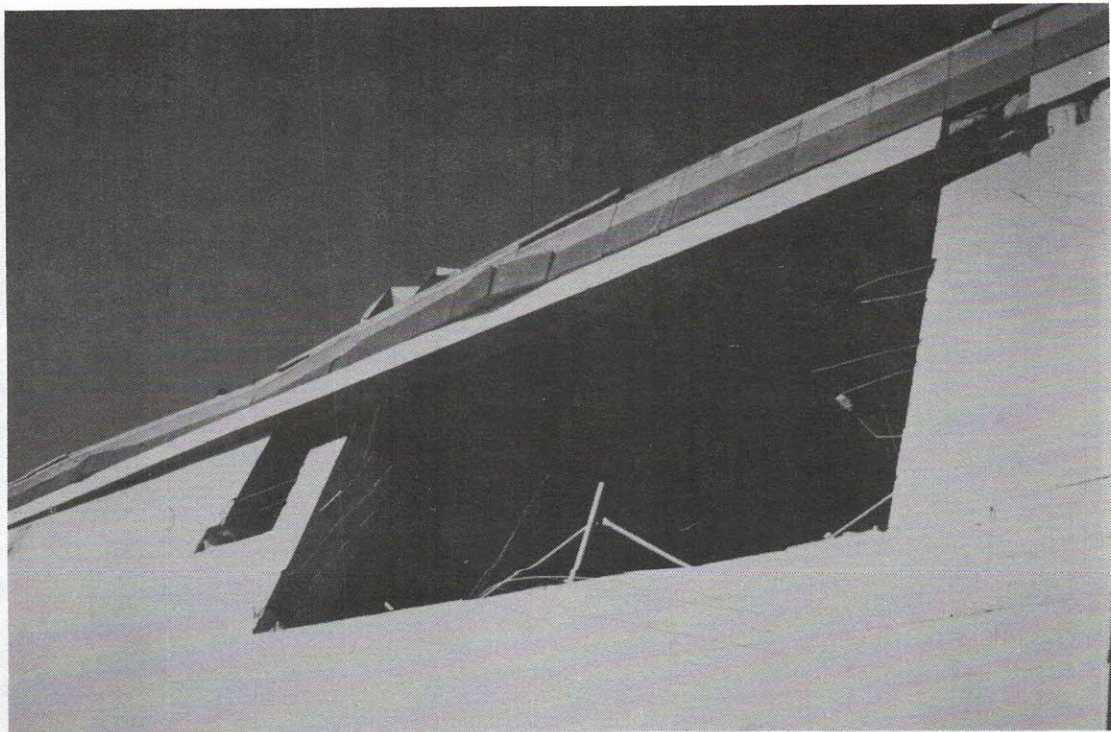


Photo 8. Collapse of wall caused by improper anchorage to the bond beam.



Photo 9. Failure of concrete walls under the action of wind loads.

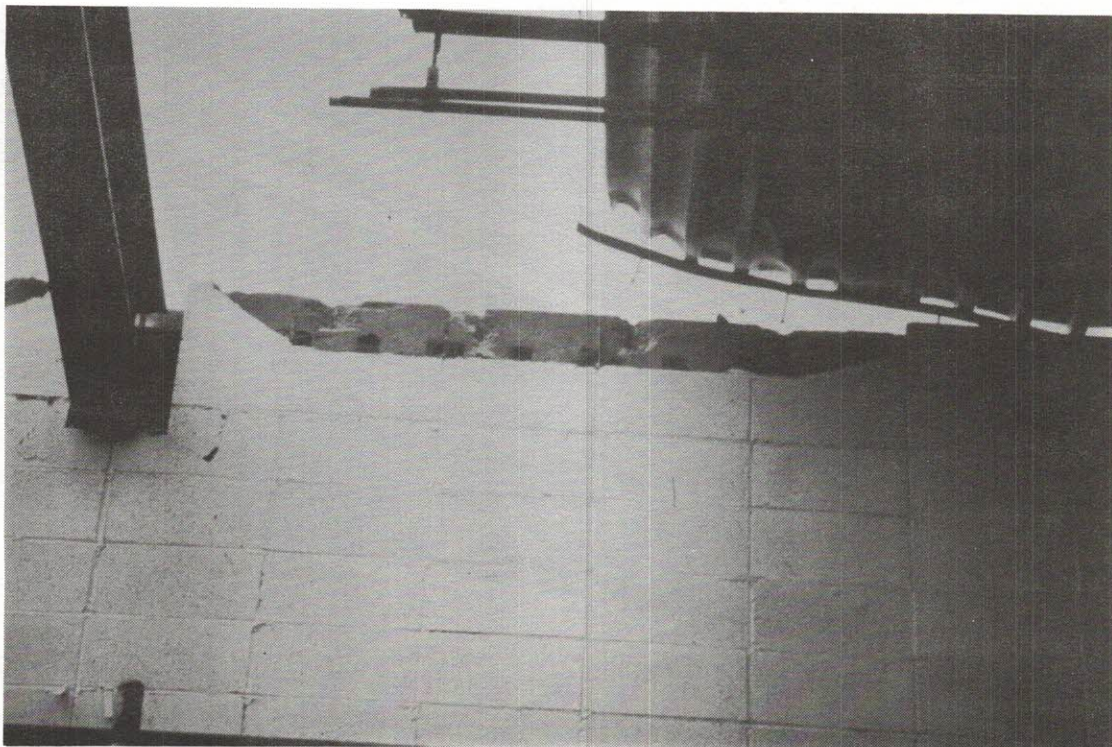


Photo 10. Failure of roof due to uplift.

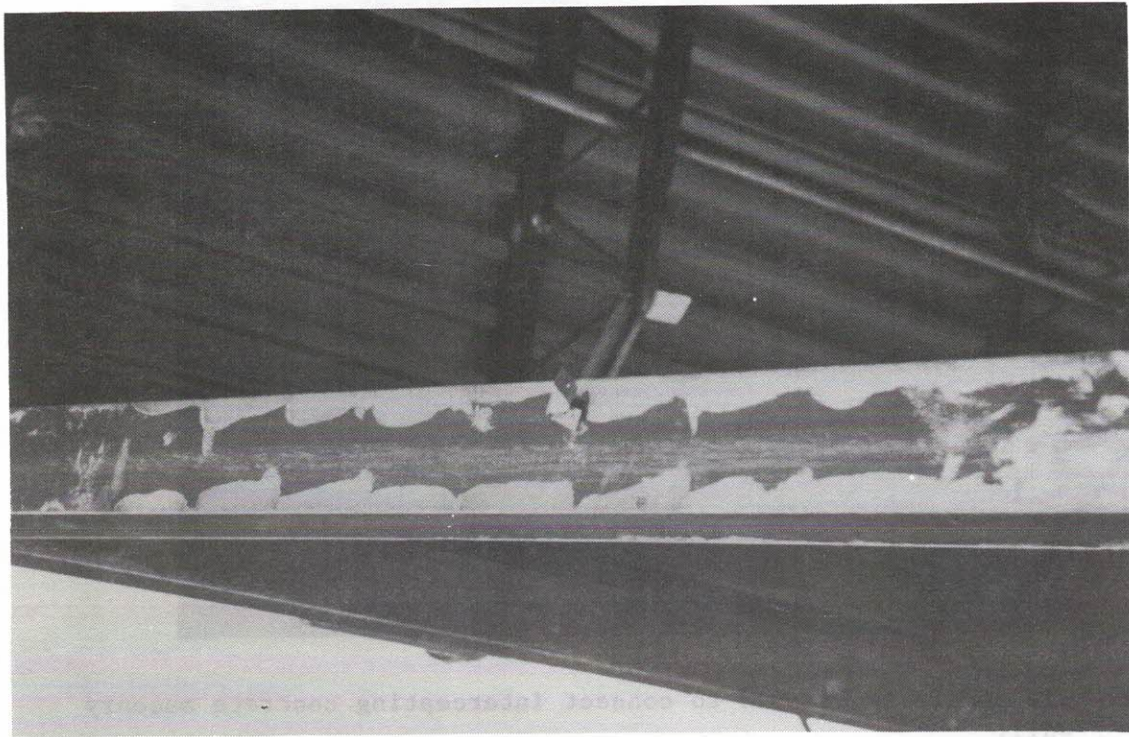


Photo 11. Infill panel connected to steel column by means of 28 gauge corrugated strip.

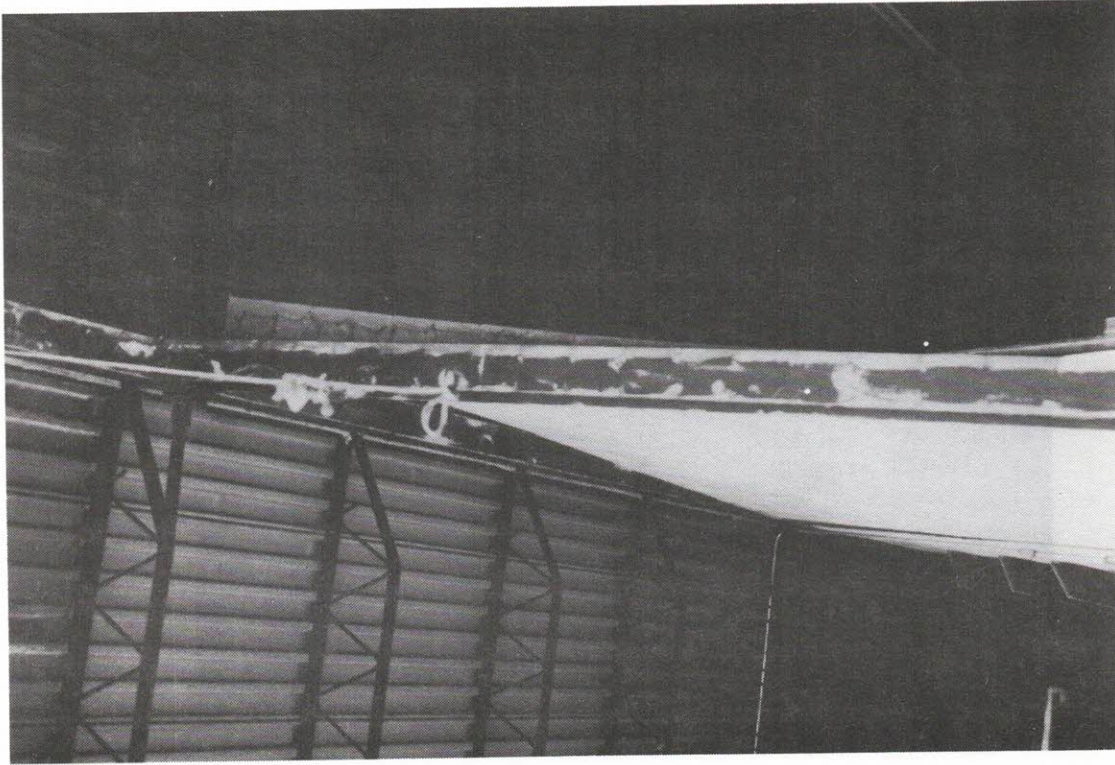


Photo 12. Concrete block wall shifted from the supporting steel column because of improper anchoring.

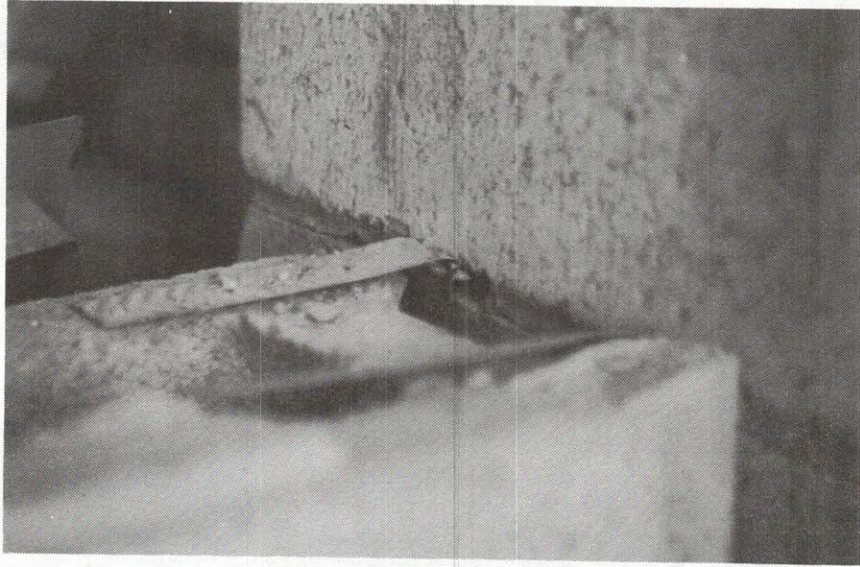


Photo 13. Connection detail of failed wall.

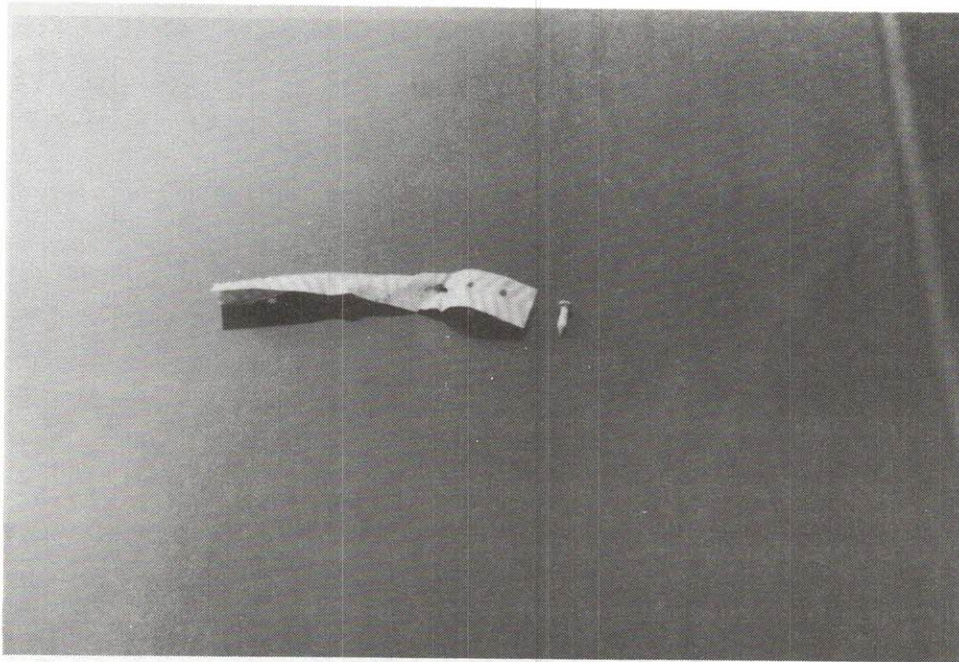


Photo 14. Tie and fastener used to connect intercepting concrete masonry wall.