WHY DID THIS BUILDING NOT COLLAPSE?

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ABSTRACT

In 1997 a condominium building was constructed in Winnipeg, Manitoba, Canada. Some two years later, one of the tenants, a retired engineer, noticed some small concrete fragments on the floor of the underground parkade and, looking up, saw some cracked reinforced concrete beams. This was promptly reported to the building manager, who in turn commissioned an engineering evaluation of the situation. A structural investigation disclosed that some of the main transfer beams supporting load-bearing walls had only about 20% of the required reinforcement. The investigation identified an exceptionally serious problem. In fact, there was some difficulty in determining why the building was still standing! Immediate attention was required to prevent collapse and to restore the required strength and, in the process, to find out why collapse had not already taken place. This paper explains why the building did not collapse and the remedial steps taken to ensure safety.

Key words: Masonry, Under-Reinforced, Cracking, Repair, Grouting, Prestressing

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INTRODUCTION

Failure of structures can be due to any one or more of the following factors: environmental effects such as freeze/thaw, inferior materials, poor construction practices, faulty structural design, and poor detailing. According to the information from insurance records in the U.S., the repair market has risen from about twenty five percent to more than fifty percent of the construction activity over the last ten years; eighty eight percent of the repair cost being due to design errors and poor construction practices (Mailvaganam 1992). In the present instance, the culprit very clearly was a faulty structural design.

This paper describes the problem of serious cracking observed in reinforced concrete beams supporting the main floor of a five-story condominium built in Winnipeg, Manitoba, Canada, in 1997. It soon became clear that the cracking was the result of gross under-reinforcement (20% - 25% of that required) of some transfer beams. Once the cause of the cracking became evident, there was the need for some urgent remedial work, and the interesting task of determining why the building had not collapsed. The paper describes the structural mechanisms that prevented collapse and the remedial work undertaken.

STRUCTURAL LAYOUT

The building is a five-story condominium with 200 mm load-bearing masonry walls supporting 200 mm precast concrete hollowcore floors and roof, non-load-bearing exterior walls being steel stud with brick veneer. There is one level of underground parking, and the foundation system is precast concrete driven end-bearing piles. The building is located close to the Red River with a geotechnical requirement that no additional load be placed on the riverbank. Figure (1) shows schematically a plan of the relative placement of the tower (34 x 27m) and the basement (42 x 37m), and the general layout of hollowcore and bearing walls.

The main bearing walls all run in one direction and support the hollowcore roof and floors. Balcony slabs span their long direction and are supported either on the bearing walls or on short lengths of masonry wall. As shown in Figure 2, the bearing walls are supported at the main floor by a system of substantial reinforced concrete transfer beams (B1) and the transfer beams in turn are supported on a series of reinforced concrete columns. There are a series of minor beams B2 located under non-load-bearing walls, such as the exterior veneer walls. These beams have about 20% of the bending resistance of the main transfer beams B1.

THE PROBLEM

At this point in the description of the structural layout a serious error becomes apparent: namely, that there are a few transfer beams B1 supported not on columns or transfer beams but on lesser beams B2. These lesser beams B2 in the locations A1 and A2 outlined in Figure (2) clearly should have been more substantial transfer beams of a typesimilar to B1. It is, of course, remarkable that this error was not identified during the various checks that are made during the design, checking and construction processes.

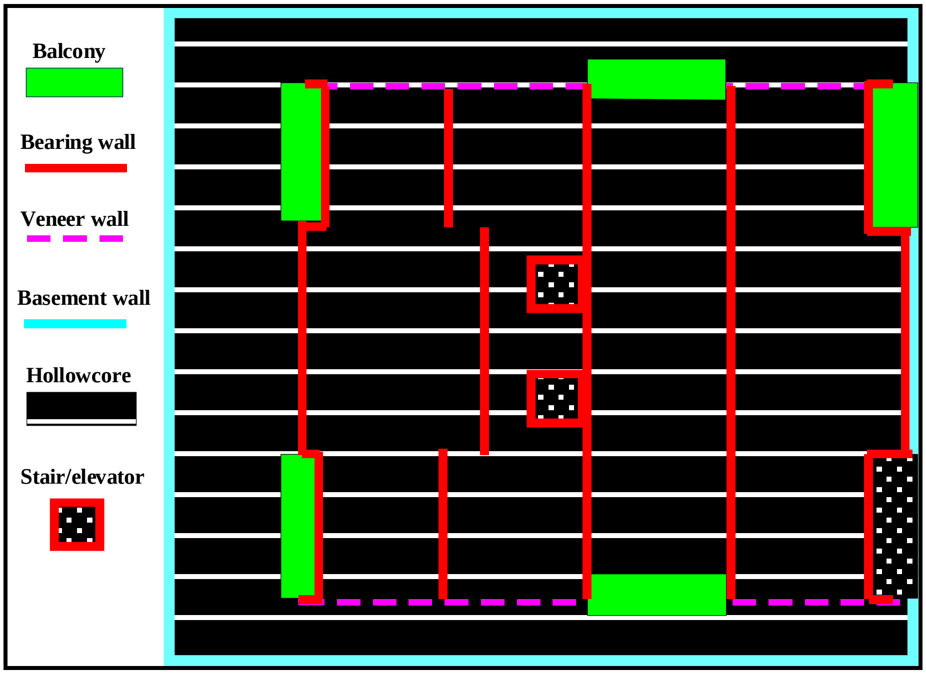

Figure1: Plan of Relative Placement of Basement and Tower.

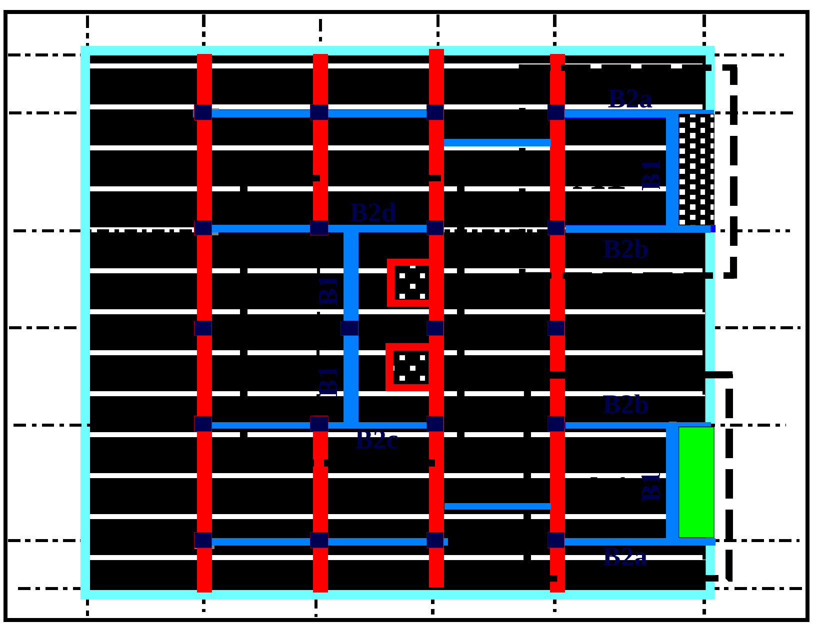

Figure 2: Locations of Concern A1 and A2.

The general configuration of the overloaded beams B2 (B2a – B2d) is shown in Figure (3). They support the heavily loaded beams B1 at about one-quarter of the span length from one end. Their spans are, variously, in the order of 7.5m to 10.0m, and they are supported at their ends either on the exterior basement wall or on columns. When the review calculations were made, it was clear that not only were these beams seriously overloaded, they actually should have collapsed under the tributary dead load. The expected behavior was the simple collapse mode shown in Figure (3). However, collapse did not take place, and a site inspection showed cracking consistent with serious overloading, crack widths being in the order of 2-3mm, about ten times the normal crack width.

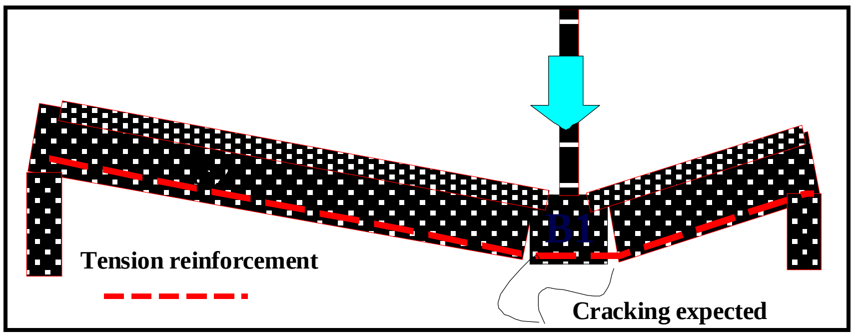

Figure 3: Configuration and Expected Behavior of Beams B2.

DIAGNOSTIC STUDY

Using the provisions of the Canadian Standards Association standard A23.3, a limit states strength analysis of beams B1 and B2 was used in determining their moments of resistance. The section properties and moments of resistance of beams B1 and B2 are summarized in Table (1). A comparison between the calculated moments of resistance and the factored design moment, and between the actual and required reinforcement is given in Table (2).

Beams B1 were generally 1000+/- x 900 mm reinforced with 11 -30M bottom bars and 3 -30M top bars with 4 -20M bars at the level of the hollowcore ledge. The positive moment of resistance of beams B1 is 2172 kN-m, and their negative moment of resistance 808 kN-m. The lesser beams B2 are 450 x 900 mm reinforced with 3 -25M bottom bars and 3 -20M top bars, with positive and negative moments of resistance of 423 kN-m and 254 kN-m, respectively. In other words, beams B1 are about four to five times as strong as B2.

Table 1: Sectional Properties and Moments of Resistance of Beams B1 and B2.

Beam Designation	Dimensions (mm)	Reinforcement			Mr (kN-m)	
		Bottom	Top	@ Hollowcore	(+ve)	(-ve)
B1	+/- 1000 x 900	11- 30M	3-30M	4-20M	2172	808
B2a/b/c/d	450 x 900	3-25M	3-20M	N/A	385	154

Table 2: Comparison Between Required and Actual Strength.

Beam Designation	M_r (+ve) (kN-m)	M_f (kN-m)	M_r/M_f (%)	As $_{Req.}$ (mm^2)	As $_{Act.}$ (mm^2)	As $_{Act.}$/As $_{Req.}$ (%)
B1	2172	2000+/-	110+/-	7000+/-	7700	110+/-
B2a/b/c/d	385	2035	18.9	7900	1500	19

The locations of concern are identified as A1 and A2 in Figure (2), with four types of beam B2, namely B2a and B2b in locationsA1, and B2c and B2d in location A2. In all instances, B2a through B2d, the beam type B1 would have been appropriate, and this likely was the intent of the original structural design. In all four cases the beams B2 have about 20% of the resistance required for the tributary design load, and only about 50% of the reinforcement required to prevent collapse under the tributary dead load. This, of course, raises the interesting question of why they did not collapse.

WHY THIS BUILDING DID NOT COLLAPSE

<u>Location A1</u>

The beams B2a and B2b in location A1 support the heavily loaded beam B1. The factored design load from B1 is in the order of 850 kN, but the factored resistance to load of B2a and B2b at this location is only about 250 kN. Beams B2a, in addition to the load from B1, support a short length of balcony bearing wall. B2a is, of course, understrength and a field inspection showed cracking consistent with serious overload and onset of collapse. The collapse mechanism requires the beam to deflect. However, as the beam deflects the balcony wall above cannot rotate because of horizontal constraint at the floor levels. This horizontal constraint is provided by the floor diaphragms transferring the required horizontal force back to the vertical service shafts. As a result of this restraint of wall rotation, the wall separated from the beam B2a at location A and was hung up on the beam at location B, as shown in Figure (4). The validity of this suspected mechanism was borne out by the presence of horizontal cracking at A in the masonry above, indicating separation, and vertical cracking at B, indicating serious overload of the masonry in compression. Fortunately, the masonry contractor, suspecting heavy loads in the short balcony walls had taken care to grout them fully. The result of this initiating collapse mechanism was to shift the load from B1 out towards the end of the balcony wall, significantly reducing the moment in B2a.

The situation for beam B2b is rather similar in that, again, there is a balcony wall above the beam This time the balcony wall extends further but has a large patio door opening. Once again, the balcony wall is prevented from rotation by the floor and roof diaphragms. The walls in this instance redistribute the load by a different mechanism - mainly through the masonry lintels over the door opening as shown in Figure (5). This was borne out by the presence of cracking in B2b at B1, and lintel cracking in the balcony wall. The result is a redistribution of load from B1 toward the support, with a consequent reduction of moment in B2b. Once again, the good grouting of the masonry balcony walls prevented collapse. It is interesting to note that the masonry contractor cannot recall these balcony walls being bonded with the load-bearing masonry walls. In any case, there clearly was a transfer of load.

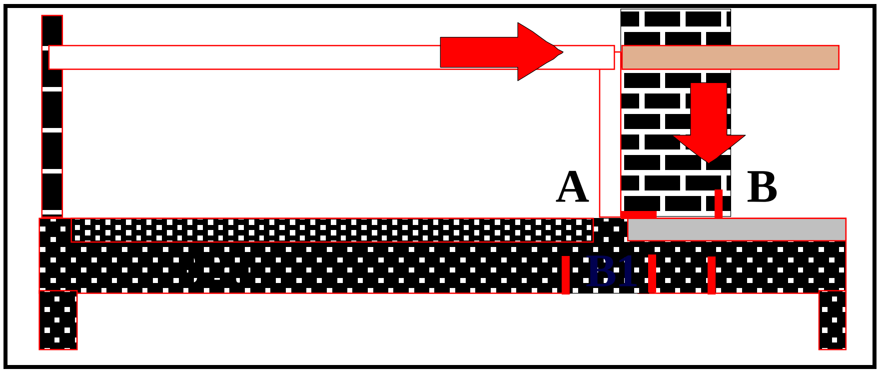

Figure 4: Redistribution of Loads on B2a.

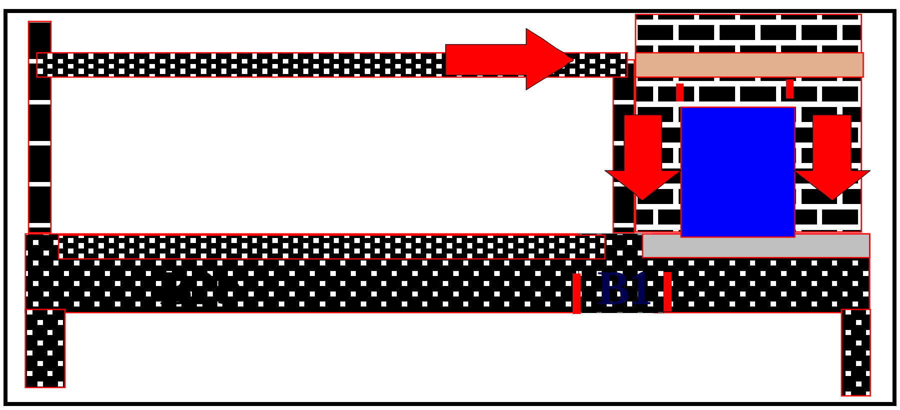

Figure 5: Redistribution of Loads on B2b.

<u>Location A2</u>

Both beams B2c and B2d are grossly under-reinforced for the reactions from B1. Rather mercifully, there is a masonry wall beneath B2c, which exhibited normal crack widths, and sufficient foundation capacity below. In this instance, it was merely a matter of grouting the masonry wall to sustain the unexpected load safely. This is shown in Figure (6). Beam 2d, on the other hand, had some large cracks indicative of steel yield and effective structural failure, the resistance of the beam to vertical load being 350 kN, compared to a factored design load of 950 kN, as shown in Figure (7). The reason B2d did not collapse can be seen from Figure (8). Once B2d started to deflect, the wall above B1 became hung up at X, reducing the load transmitted to B2d, but now overloading the beam B1 and the foundation under the column.

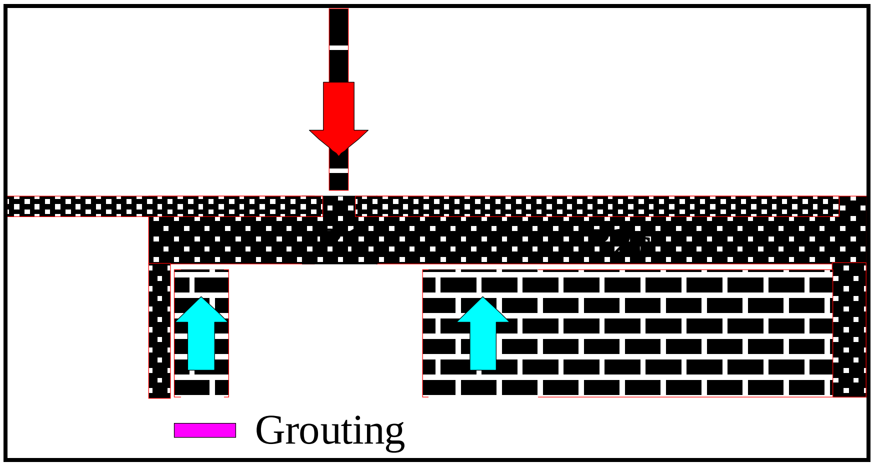

Figure 6: Redistribution of Loads on B2c.

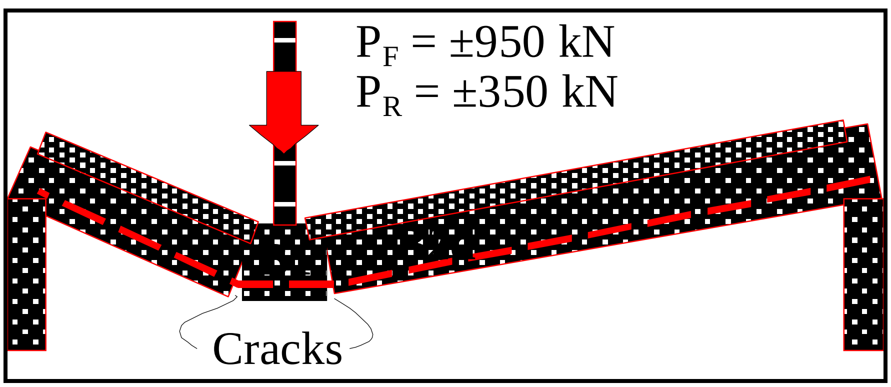

Figure 7: Loading on B2d.

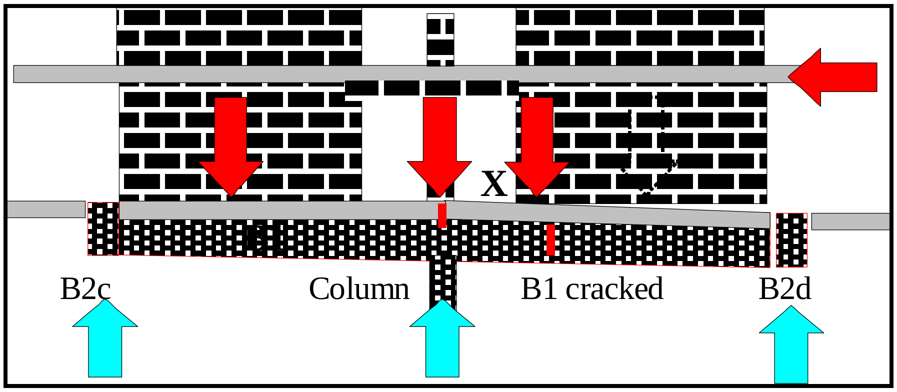

Figure 8: Redistribution of Loads on B1 at Location A2.

REMEDIAL WORK

<u>Beams B2a and B2b</u>

It was not feasible to underpin beams B2a and B2b, for two reasons: namely that any columns would interfere with parking, and also that any underpinning would have to be installed to bedrock at a depth of about 15 metres. Another solution considered was to attach steel plates to the underside of these beams, but the number of anchors to be installed was excessive for the space available. Besides, the masonry walls now so seriously overloaded would not be relieved of any load. For the same reason, strengthening with FRP (fiber-reinforced polymer) was not feasible. The solution selected was to post-tension the under-strength beams with high-tensile Dywidag bars. The Dywidag bars were 36mm, each stressed to 700kN+/-, one (cranked as shown in Figure (9) to provide the required upward force) each side of each beam. Concrete walls or beams at each end were drilled to receive the bars with substantial steel box hardware at each end to sustain the post-tensioning forces. The cost was about $10,000 CAN for each beam.

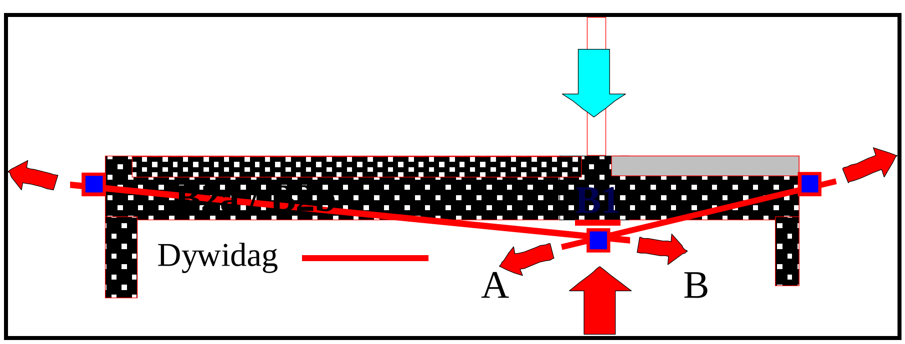

Figure 9: Remedial Work for Beams B2a and B2b.

Beams B2d

Since B2d was not accessible for the remedial solution used for B2a and B2b, and since headroom was limited for pile installation, a jack-in pile was installed under B1 about 1.0 m from B2d as shown in Figure (10). This involved using B1 as the reacting structure while 1.2 m lengths of 300 mm diameter steel tube were jacked one after the other into the clay and welded together. The problem here was that geotechnical specifications called for a jacking force of 150% of the maximum expected service load, while only about 60% of the service load was actually present. The structure was now required to support upward forces about 2.5 times the actual load, and this upward force had to be sustained for 24 hours before a steel column was installed between the pile and the beam. This procedure was conducted uneventfully. The cost of remedial work at this location was about $20,000 CAN.

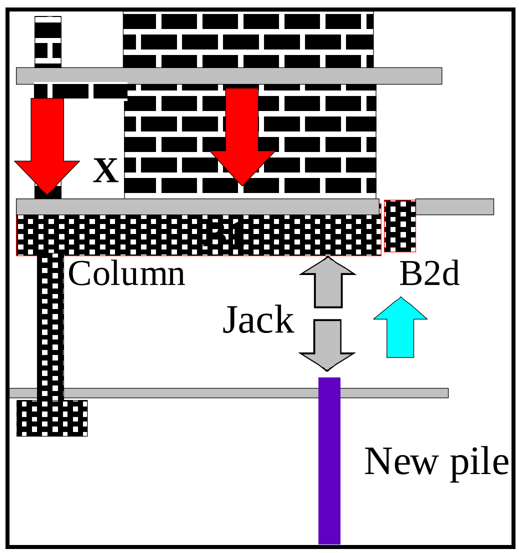

Figure 10: Remedial Work for B1 at Location A2.

SUMMARY

A very serious engineering error was made during the design of this building, one that could easily have led to a disaster. Beams supporting heavily-loaded transfer beams were reinforced with only about 20% of the required reinforcement. Fortunately, the presence of masonry walls, however short, was sufficient to permit a redistribution of load sufficient to prevent immediate collapse, although strength calculations for the masonry clearly showed that eventual collapse was not far off.

Of the six under-strength beams, four were strengthened by prestressing with Dywidag bars, one had a masonry wall underneath that required grouting, and one had its load relieved by installing a jack-in pile nearby. It is interesting to note from visual inspections that immediately after remedial action crack widths were reduced by more than 50% and had practically disappeared about two months later.

The total cost of repair was $60,000 CAN ($40,000 US), a fraction of what the cost would have been had a retired engineer not been vigilant.

REFERENCES

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