Masonry Saved This Under-Reinforced Building

After finding cracked RC beams, a structural investigation revealed that some main transfer beams had no more than 20% of the required reinforcement

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In 1997, developers erected a five-story condominium building 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. He promptly reported this to the building manager, who in turn commissioned an engineering evaluation of the situation.

During their structural investigation, the engineers discovered 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, the engineers had some difficulty in determining why the building was still standing! Immediate attention was required to prevent collapse, restore the required strength, and, in the process, find out why collapse had not already taken place.

Structural layout

The condominium's load-bearing walls are 200-mm concrete masonry units. They support the 200-mm-thick, precast-concrete, hollowcore roof and floors. The non-load-bearing exterior walls are steel stud with a brick veneer. One level of underground parking sits above a foundation system of driven precast-concrete 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 27 m) and the basement (42 x 37 m), and

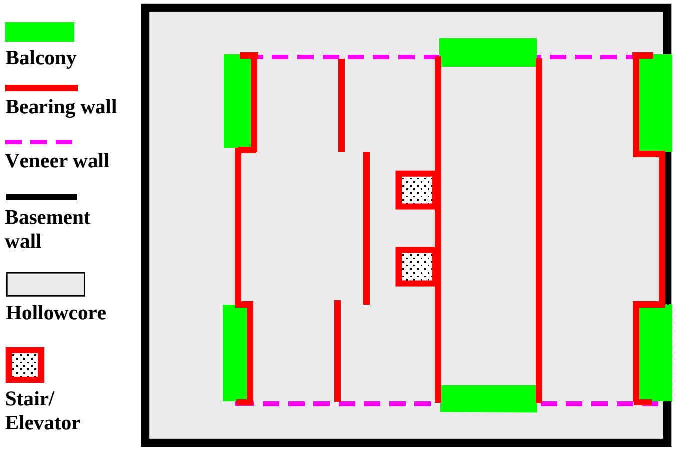


Fig. 1: Relative placement of basement and tower

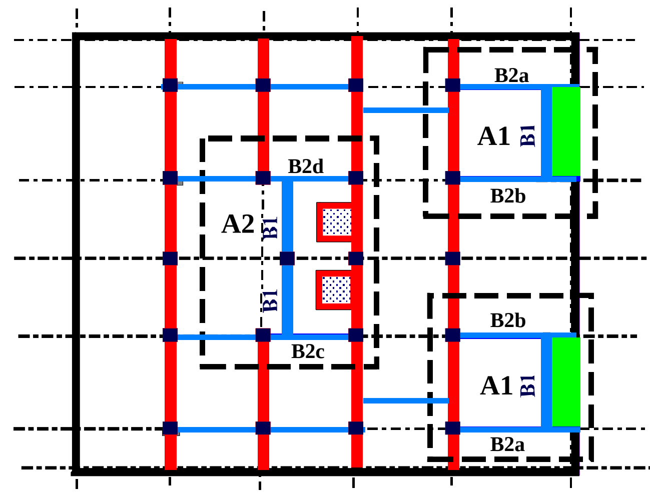


Fig. 2: Locations of concern A1 and A2

TABLE 1: Sectional properties and moments of resistance of B1 and B2 beams

Beam designation	Dimensions (mm)	Reinforcement			M_r (kN-m)	
		Bottom	Top	@ Hollowcore	(positive)	(negative)
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 (positive) (kN-m)	M_f (kN-m)	M_r/M_f (%)	As_{req} (mm²)	As_{act} (mm²)	As_{act}/As_{req} (%)
B1	2172	2000	110	7000	7700	110
B2a/b	385	2035	18.9	7900	1500	19
B2c/d	385	1520	25.3	6000	1500	25

the general layout of the bearing walls and the hollowcore floor.

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

The problem

At this point in the description of the structural layout, a serious error becomes apparent: a few B1 transfer beams, are supported not on columns or other transfer beams, but on the lesser B2 beams. These B2 beams in the A1 and A2 locations outlined in Fig. 2 should have been more substantial transfer beams similar to the B1 beams. Remarkably, this error was not identified during the various checks made during the design and construction processes.

The general configuration of the overloaded B2 beams is shown in Fig. 3. They support the heavily loaded B1 beams at about one-quarter of the span length from one end. Their spans are 7.5 to 10.0 m long, and they are supported at their ends either on the exterior basement wall or on columns. When investigating engineers ran the review calculations, it was clear to them that not only were these beams seriously overloaded, but they actually should have collapsed under the tributary dead load.

The simple collapse mode shown in Fig. 3 would have been the expected behavior. Collapse did not take place, however, although their site inspection showed cracking consistent with serious overloading. Crack widths were 2 to 3 mm wide—about ten times the normal crack width.

Diagnostic study

Using the provisions of the Canadian Standards Association's (CSA) concrete standard A23.3 (1994a), and the CSA's masonry standard S304.1 (1994b), the engineers used a limit-states strength analysis of the B1 and B2 beams and the masonry walls to determine their moments of resistance. The section properties and moments of resistance of the B1 and B2 beams are summarized in Table 1. A comparison between the calculated moments of resistance (M_r) and the factored design moments (M_f), and between the actual (As_{act}) and required (As_{req}) reinforcement is given in Table 2.

The B1 beams are generally 1000 x 900 mm and reinforced with 11 30M bottom bars and three 30M top bars, with four 20M bars at the level of the hollowcore ledge. The positive moment of resistance of the B1 beams was 2172 kN-m, and their negative moment of resistance was 808 kN-m. The lesser B2 beams are 450 x 900 mm and reinforced with three 25M bottom bars and three 20M top bars, and their positive and negative moments of resistance were 385 and 154 kN-m, respectively. In other words, the B1 beams were about four to five times as strong as the B2 beams.

The locations of concern are identified as A1 and A2 in Fig. 2, with four types of B2 beams, namely B2a and B2b in location A1, and B2c and B2d in location A2. In all instances B2a through B2d, the beam type B1 would have been appropriate, and this was likely the intent of the original structural design. In all four cases, the B2 beams had about 20 to 25% of the resistance required to prevent collapse under the tributary dead load.

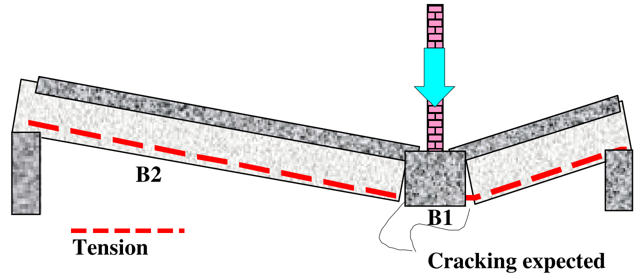


Fig. 3: Configuration and expected behavior of B2 beams

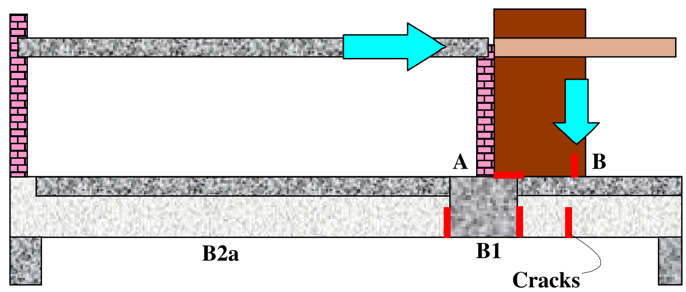


Fig. 4: Redistribution of loads on B2a

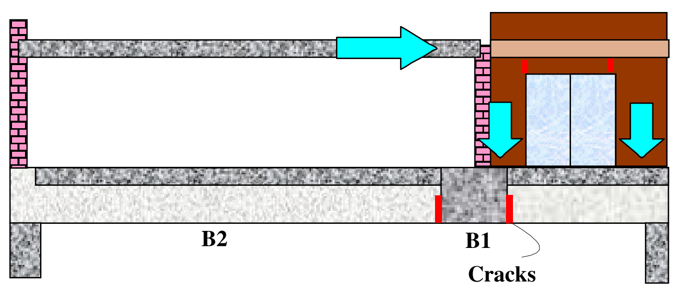


Fig. 5: Redistribution of loads on B2b

Why this building did not collapse

Location A1

The B2a and B2b beams in location A1 support the heavily loaded B1 beam. The factored design load from B1 was in the order of 850 kN, but the factored resistance to load of B2a and B2b at this location was only about 250 kN. The B2a beams, in addition to the load from B1, support a short length of balcony bearing wall. B2a was, of course, under strength, and the field inspection showed cracking consistent with serious overload and the onset of collapse. To collapse, the beam would have to deflect. As the beam deflected, however, the balcony wall above could not rotate because of the horizontal constraint at the floor levels. This horizontal constraint was 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 B2a beam at location A and was hung up on the beam at location B (Fig. 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 the B1 beam out towards the end of the balcony wall, significantly reducing the moment in B2a.

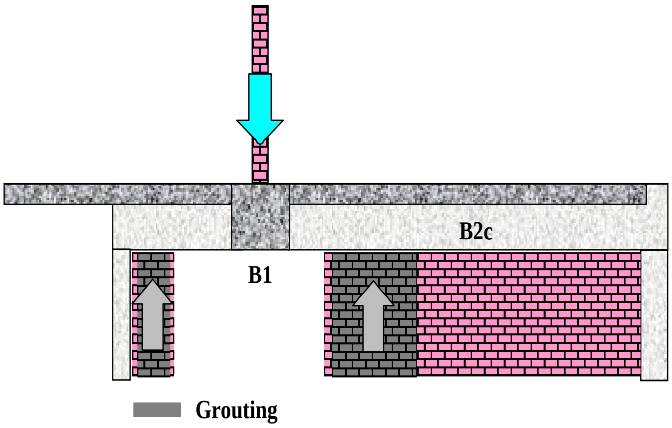


Fig. 6: Redistribution of loads on B2c

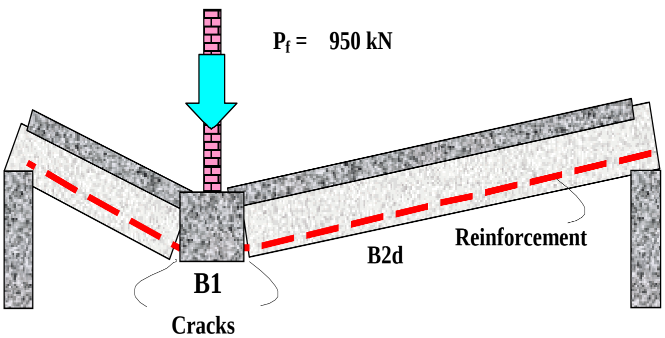


Fig. 7: Loading on B2d

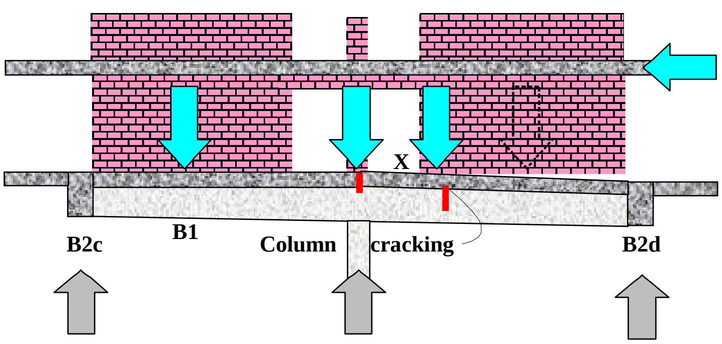


Fig. 8: Redistribution of loads on B1 at location A2

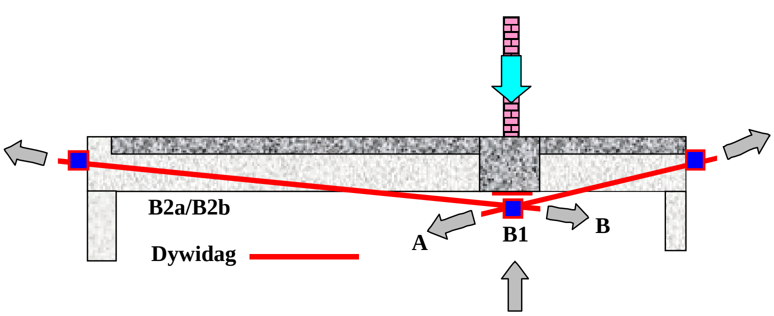


Fig. 9: Remedial work for B2a and B2b beams

The situation for the B2b beam was rather similar in that, again, there was a balcony wall above the beam. This time the balcony wall extended further, but had a large patio-door opening. The balcony wall was prevented from rotating by the floor and roof diaphragms. The walls in this instance redistributed the load by a different mechanism—mainly through the masonry lintels over the door opening (Fig. 5). This was borne out by the presence of cracking in B2b at B1, and by the lintel cracking in the balcony wall. The result was 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. Interestingly, 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.

Location A2

Both the B2c and B2d beams were grossly under strength 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 (Fig. 6).

The B2d beam, on the other hand, had some large cracks indicative of yielding steel and effective structural failure. The resistance of the beam to vertical load was 350 kN, compared with a factored design load of 950 kN (Fig. 7).

The reason the B2d beam did not collapse can be seen from Fig. 8. Once B2d started to deflect, the wall above B1 became hung up at X, reducing the load transmitted to B2d, but overloading the B1 beam and the foundation under the column.

Remedial work

B2a and B2b beams

Underpinning the B2a and B2b beams was not feasible for two reasons: any columns would interfere with parking, and any underpinning would have to be installed to bedrock at a depth of about 15 m. The engineers considered attaching steel plates to the underside of these beams, but the number of anchors to be installed was excessive for the space available. Besides, the seriously overloaded masonry walls would not be relieved of any load. For the same reason, strengthening with fiber reinforced polymer composite was not feasible.

The engineers elected to post-tension the under-strength beams with high-tensile Dywidag bars. The repair contractor stressed each 36-mm reinforcing bar to about 700 kN (Fig. 9) to provide the required upward force on each side of each beam. The contractor drilled the concrete walls or beams at each end to receive the bars, which required substantial steel box hardware at each end to sustain the post-tensioning forces. The cost was about \$6700 US (\$10,000 CAN) for each beam.

B2d beams

Because B2d was not accessible for the remedial solution used for B2a and B2b, and because headroom was limited for pile installation, the engineers decided a jack-in pile should be installed under B1 about 1.0 m from B2d to relieve the load (Fig. 10). This involved using B1 as the reacting structure while the repair contractor jacked 1.2-m lengths of 300-mm-diameter steel tube one after the other into the clay and welded them together. The cost of the remedial work was about \$13,500 US (\$20,000 CAN).

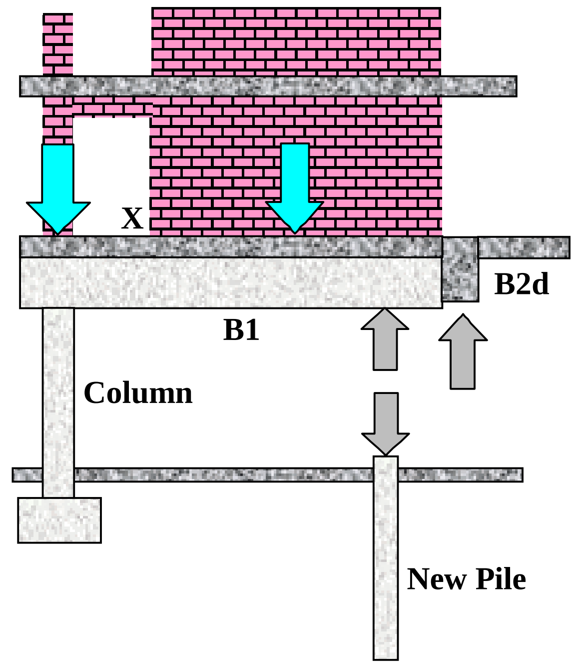


Fig. 10: Remedial work for B1 at location A2

Final result

A serious error was made during the design of this building. 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 eventual collapse was not far off.

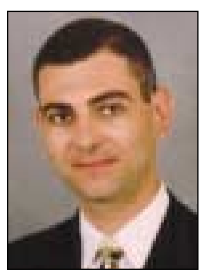
Of the six under-strength beams, four were strengthened by post-tensioning, one had a masonry wall underneath that required grouting, and one had its load relieved by installing a jack-in pile nearby. Visual inspections immediately afterward revealed that crack widths were reduced by more than 50%. They had practically disappeared about two months later. The total cost of repair was \$40,000 US (\$60,000 CAN), a fraction of what the cost would have been had a retired engineer not been vigilant.

References

Canadian Standards Association, 1994a, “Concrete Design for Buildings,” Standard A23.3- *M94*, Etobicoke, Ontario, Canada.

Canadian Standards Association, 1994b, “Masonry Design for Buildings (Limit States Design),” Standard S304.1-94, Etobicoke, Ontario, Canada.

Selected for reader interest by the editors.



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