CASE STUDIES: EARTHQUAKE RISK BUILDINGS

1. NEW PLYMOUTH GIRLS' HIGH SCHOOL STRUCTURAL STRENGTHENING AND UPGRADING OF THE 1926 BLOCK

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INTRODUCTION:

The building forms the apex of a two-storey L-shaped classroom block, the 'legs' of the L having been added later and including walls of reinforced concrete. This section was built in 1926 and is of cavity brick construction with reinforced concrete bands provided at first floor and eaves level. Floors and roof trusses are of timber.

The L-shaped block as a whole is not of great architectural importance and the suite of buildings in the school grounds have no consistent architectural form. Consequently, thought had been given to demolishing the 1926 building of the classroom block and replacing its facilities elsewhere in the grounds. However, because toilets and main stairway servicing the L-shaped classroom block are all located in the 1926 building, it was decided it should be retained.

An initial feasibility study had considered the possibility of tying the 1926 building to the adjacent 'legs'. Unfortunately, the system of horizontal ties at first floor level would have been seriously interrupted by the stairwell. Furthermore, such a solution relies on the brickwork being able to withstand face loading by spanning without protection between floors.

The 1926 block also contains four classrooms of about 47 m² each. The total area per floor of the two-storey block is about 250 m². A lightweight decramastic roof has replaced the former concrete tile roof in recent times.

Inevitably, upgrading of other aspects of an old building are considered once the decision to strengthen the structure is taken. Accordingly, both plumbing and electrical services were upgraded, and the building interior finish was completely redone.

The structural cost was about half the total value of the strengthening/refurbishing operation.

This article concentrates on structural matters.

LEVEL OF DESIGN LOADING AND STRENGTHENING SOLUTION ADOPTED:

The layout of the 1926 block is shown in Fig 1 'Structural' brick of up to 14 in. (356 mm) is present. It is evident that, by bricking up some of the window openings in the interior wall to the

corridor, reasonable symmetry would be achieved in the north-south direction; symmetry is already adequate in the east-west direction.

It was decided initially to consider two levels of design seismic loadings for the 1926 block, viz 'full code (NZS 4203:1976)' and 'two thirds code'. Design proceeded first of all using full code in both directions -

$$C_d = C \quad I \quad S \quad M \quad R$$

$$= .125 \times 1.3 \times 2.4 \times 1.0 \times 1.0$$

$$= 0.39 \text{ g}$$

for flexure.

An over-strength factor of about 50% was used for shear design.

The solution adopted in the end for protection of walls achieved this level of strength for in-plane loading as well as satisfying face loading requirements with generally the minimum percentage of reinforcement required by NZS 3101:1982. Hence a redesign to two-thirds of code was not considered warranted, an observation noted by Garrett⁽¹⁾ in the case of strengthening of Hastings Boys' High School.

Initially, diaphragm action at first floor and eaves level was designed in steel bracing using channel sections. These bracing planes were to be tied into the reinforced concrete bond beams at each level. However, after costing, this system was rejected and instead, an overlay of particle board was used at first floor and ceiling level. Neither solution was sufficiently stiff to act as a diaphragm in the normal sense of the term, i.e. where 'infinite' stiffness ensures that each element of wall receives applied shear in proportion to its stiffness. Thus, each section of wall strengthening was designed to resist load accruing from respective tributary areas of floor, plus loads resulting from their own inertia as well as that of tributary face loaded walls. The timber diaphragm therefore served primarily to tie face-loaded walls to nearby orthogonal walls. Such an approach is justified provided that the layout of walls is reasonably symmetrical. Boardman⁽²⁾ used a similar approach to design.

In order to obtain relative costs, a variety of solutions for strengthening a typical piece of brickwork of the 1926 block were designed. In order of economy (cheapest first), the solutions considered were as follows:

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FIGURE 1

1926 BLOCK & PART 1936 BLOCK AT SOUTH END.

1926 BLOCK & PART 1940 BLOCK FROM EAST SIDE.

- 1 GRC plaster coat to two faces
- 2 Steel braced frame
- 3 Post-tensioned brickwork
- 4 Reinforced hollow masonry
- 5 150 mm reinforced sprayed concrete
- 6 150 mm reinforced concrete

In spite of its relatively low place in the above ranking, 150 mm reinforced sprayed concrete was selected for all walls. This was influenced by the fact that more experience was locally available for using this method than for the other solutions. Reasons for rejecting the other possibilities were:

- . insufficient laboratory experience with GRC plaster was available at that stage, and, in any event, we were not prepared to use a thin structural plaster as a shear cum face load resisting system when only one face could realistically be plastered. This would apply in the case of external walls where the exterior brickwork appearance had to be retained;
- . the steel braced frame was only seriously considered as a replacement for the transverse (east-west) non-gravity load bearing walls. If used alongside brickwork, there would have remained the problem of protecting the adjacent wall against face loading. In addition, the difference in stiffness between frame and brickwall could have reduced the threshold of damage under in-plane loading. The solution was not used because it was decided to retain the same strengthening method for all walls;
- . in the case of the post-tensioning solution, although we understood that vertical holes through several metres of brickwork could be drilled with precision, we were concerned about loss of prestress due to creep in the medium to long term. More research is needed in this area, but the concern seems justified in the case of old brickwork;
- . reinforced hollow masonry was considered for the same locations (east-west walls) as the steel braced frame and as replacement for existing brickwork. It was not used in the end in order to retain one strengthening method for all walls.

Figure 2 shows the floor layout and the extent of wall treatment for the reinforced concrete/sprayed concrete solution used. The tender documents did not rule out the possibility of conventional reinforced concrete being offered. However, the successful tenderer (Fletcher Construction) offered sprayed concrete.

Analysis of the system of walls considered them as deep membered frames

and, in general, two layers of 665 HRC mesh were used, staggered by 75 mm to minimise shadowing from the process of applying sprayed concrete. Apart from trimmers around openings, additional bars of deformed steel were used at ends of some panels to increase flexural strength.

The ground level was approximately 1 metre below ground floor level, and the original wall construction in this area consisted of mass concrete in excess of 300 mm thick and generally 600 mm thick. Hence no tying of walls to a diaphragm at ground floor level was made but the sprayed concrete continued to footing level. Transfer of seismic load into existing foundations was achieved by epoxying bars into existing concrete. A similar means was used to transmit shear into the new walls at first floor and roof levels. Fig 3 gives details. Performance and proof testing was required of epoxied bars.

As mentioned earlier, tying of brick walls at first floor and roof level was achieved by an overlay of particle-board to floor and ceiling. In the case of the first floor level, two 12 mm sheets were overlaid, staggered relative to each other, and glued together with a gap-filling adhesive, Expandite WB. Only one layer was used at roof level, and there, internal joints were made by gluing and screwing to blocking - Figure 4 gives details.

The means of connection to ring beam/brickwork at the perimeter of diaphragm sections was more difficult. In the end, 20 m cuphead bolts at varying centres, but as close as 100 mm, were designed to provide a load path via steel angles bolted into the ring beam or sprayed concrete. Bearing strength of the structural particle-board is critical and no published information was available. An onsite modification resulted in a third 12 mm layer being glued as an edge strip to allow countersinking of the bolt heads with no loss of bearing strength. The countersunk head was also surrounded by epoxy. Figure 5 shows typical details.

An area of particular uncertainty during design was the question of protection of veneer, against outwards collapse. Examination on site during the design stage indicated that the cavity behind parts of the veneer had been mortared up and that there was effectively no adhesion between 'structural' brick and veneer in such instances. In addition, although pointed with a cement mix, the mortar to veneer was generally sand-lime only. Some butterfly ties were present but their frequency was not known. It was decided that the expense of tying the veneer to structural brick through additional ties was not warranted; this was in part influenced by the fact that the main walkway outside the building (the eastern side - see Fig 1) was separated from the building exterior by a fairly extensive planted area. It was also decided to give some protection to the veneer by grouting the cavity where it existed. The grout contained

Legend

- Ⓐ WALL TO BE DEMOLISHED AND REPLACED WITH 150mm R.C. WALL
- Ⓑ 150mm R.C. WALL ADDED TO EXISTING WALLS. PLASTER TO BE REMOVED FROM EXISTING WALLS.
- Ⓒ AS FOR B EXCEPT 265mm R.C. WALL AT 1st FLOOR
- Ⓓ EXISTING GR'D FLOOR WALLS CUT BACK 150mm FROM BEAM FACE. FIRST FLOOR WALLS CUT BACK 115mm. REINSTATE WITH 150mm R.C. WALL.
- Ⓔ EXISTING GR'D FLOOR WALLS CUT BACK 115mm FROM BEAM FACE. NO CUT BACK AT FIRST FLOOR. REINSTATE WITH 150mm R.C. WALL. PLASTER TO BE REMOVED AT FIRST FLOOR.
- Ⓕ WALLS TO BE DEMOLISHED AND REPLACED WITH TIMBER AT FIRST FLOOR ONLY.

an expansive admixture to enhance bond, the cavity was flooded beforehand to saturate brickwork and a careful watch kept for signs of movement or cracking in the veneer during the grouting operation. In addition, we were worried about the possibility of efflorescence but, encouraged by the experience at Hastings Boys' High School⁽¹⁾, we decided to proceed. In the event, no movement of the veneer brickwork was observed and to date efflorescence has only appeared as a fine dusting in some areas.

CONSTRUCTION:

Core tests of sprayed concrete showed its variability, not only in terms of strength (core strength of 20 MPa at 28 days was specified) but also in terms of evident lack of bond to the wire reinforcing mesh on occasions. Cylinder strengths of core samples taken from separate test panels consistently exceeded the specified strength. However, strengths of a limited number of cores taken from the permanent works were less encouraging: here the 28 day strength generally ranged from 15 - 28 MPa with most test results in the 20 - 25 MPa range. A value as low as 9 MPa was recorded.

Of particular concern was the evident lack of compaction over the depth of the 400 mm floor joist at first floor level. It is in this region that both new gravity support to joists and lateral support to faceloaded walls exist. Considerable remedial work was necessary here.

Details of trimming reinforcement were originally set the same as is normally used for reinforced concrete but were changed on site to reduce the possibility of shadowing effects. In addition, the specified D10 ties epoxied to structural brickwork were changed to an equivalent area of R6 ties because of the greater ease of drilling a smaller hole in often brittle brickwork. Figure 6 shows a typical area of wall.

It was found that supervision of the epoxying operation was critical: initial fixings produced without supervision had virtually no strength. However, given proper supervision, results were such that bar yield was achieved in performance tests, and proof tests were also satisfactory. Figure 7 shows the range of results. Unfortunately, due to a lack of readily available equipment, continuous proof testing (a minimum of 5% of fixings were required to be tested by the specification) was not carried out. It has been hoped that a simple fulcrum arrangement might have been possible, but this was not feasible and so a hydraulic jack with direct pull was used, with the crew being brought in from outside the city.

The contract drawings showed vertical dowels passing right through the first floor ring beams and thereby connecting adjacent levels of sprayed concrete. However, this was changed on site to L-bars (Figure 3) epoxied horizontally

into the ring beam, because the horizontal reinforcement in the beams behind which the vertical dowels would have to pass could not be located.

As mentioned, grouting of the cavity did not give rise to the problems feared, notwithstanding the weakness and brittleness evident in the bricks. However, the grout did not fill all the cavity available, and in the end, the system of check ports specified in the documents was amended. The bottom two courses was not effectively filled due to the presence of mortar droppings from the time of original construction. During remedial work in one area to these bottom courses, it was apparent that excellent adhesion of the grouted cavity to brick above these courses had been achieved.

The laying of the particle-board diaphragms proceeded quite satisfactorily. However, sheets tended to be cut in order to abut the adjacent inner sheet, giving rise to a gap at the junction with brickwork and loss of bolt edge distance. The lack of squareness of room spaces also often resulted in one edge losing significant edge distance, particularly at ceiling level. Figure 8 shows the edge detail.

SUMMARY AND OBSERVATIONS:

- 1 In a low-rise building of this type, the achieving of a level of seismic strength in accordance with the current loadings code for new buildings (NZS 4203:1976) is not onerous.
- 2 Half of the total cost of the contract was consumed in non-structural areas. Consideration needs to be given to reducing the non-structural component; alternatively, it should be recognised that complete refurbishing is being achieved together with an increase in seismic strength to a level of loading applicable to modern buildings.
- 3 Although the level of risk to life is greatly reduced to approximate that applying to new buildings following the addition of strengthening measures such as that described above, it is unlikely that the susceptibility of the strengthened brickwork to damage has been reduced to the level of that of a new building because of the retention of brittle materials, particularly the exterior veneer. However, the degree of damageability compared to that of a new building is simply not known.
- 4 There is a need for development of methods of connection of timber diaphragms to surrounding brickwork. While such diaphragms are economically much more attractive than steel, it is evident that the method of connection used in this project could be refined through laboratory testing.

- 5 Epoxy connections of reinforcing bars is extremely workmanship and supervision dependent and a situation of no strength is all too easily achieved. Site testing is essential.
- 6 Sprayed concrete is very quickly applied but the quality of the product deteriorates rapidly in areas where the compactive effort of the application is absorbed. Conventional cast in situ reinforced concrete must be used in such locations.

REFERENCES:

- 1 I.J. Garrett: "Hastings Boys' High School Administration Buildings, Hastings, Hawke's Bay, New Zealand: Case Studies : Earthquake Risk Buildings" Bulletin N.Z. National Society for Earthquake Engineering, Vol. 16, No. 1, March 1983.
- 2 P.R. Boardman: "Restoration of Old Auckland Customhouse : Case Studies : Earthquake Risk Buildings": Bulletin N.Z. National Society for Earthquake Engineering, Vol. 16, No. 1, March 1983.

FIGURE 3

===== 2 SHEETS OF 12mm THICK 'STRUCTEX'
 ===== EXISTING TONGUE & GROOVE FLOORING.

FIGURE 4

FIGURE 5

FIGURE 6 TYPICAL WALL JUNCTION
REINFORCEMENT.

FIGURE 7 TWO EPOXIED BAR TEST SPECIMENS:
UPPER SPECIMEN HAD ADEQUATE
STRENGTH, THE LOWER HAD
INSUFFICIENT EPOXY STRENGTH.

FIGURE 8

BOLTS TO STRUCTEX FLOOR AT DOOR OPENING.

DIAPHRAGM VIEWED FROM BELOW.