4. RESTORATION OF OLD AUCKLAND CUSTOMHOUSE

P. R. Boardman*

This building was built in 1890 and designed by Thomas Mahoney, who, with his father William, was also responsible for the Auckland City Art Gallery.

It was occupied until the early 1970's by several government departments in addition to the Customs Department and this explains the cellular construction. There are three reasonably distinct areas on each floor.

Restoration Commission:

In 1980 a scheme to refurbish the building for use as a shopping and entertainment centre was commissioned. Jasmad Group Limited were appointed Architects and Holmes Wood Poole & Johnstone were appointed Consulting Engineers. Construction was carried out in 1981.

Description of Building:

The building has a total floor area of 2800m^2, including a part basement, ground floor, and two suspended floors. It is U shaped in plan and the inside of the U is covered at ground floor forming the present theatre. The walls are brick, up to 800 mm thick, plastered inside and out, with elaborate plaster and stone cornices, parapets and general ornamentation.

The floors are timber, with occasional steel beams where spans are large. The roof is supported on timber trusses which span about 10 metres. An unusual feature is the brick strongrooms which have tied barrel vault roofs made in brick. The foundations are brick and mass concrete carried down to the Waitemata Series sandstone approximately three metres below ground. (Refer to figures 1 to 3).

Engineering Design Philosophy:

The basic engineering approach has been to make use of the existing elements to as great an extent as possible. Thus the number of new structural elements required is kept to a minimum, as are costs and a significant improvement in seismic capacity is achieved.

Floors are converted to effective diaphragms by connecting them to the walls, and walls and floors are strengthened as necessary to achieve the seismic design levels.

Local appendages and features, worth preserving because of their architectural merit, are strengthened as necessary and tied back to the main structure.

Calculations are necessarily approximate but sufficient to ensure overall resistance is adequate. Connections are generally designed on the drawing board with check calculation only when these are judged to be necessary.

Investigation and Analysis:

Introduction -

In designing the strengthening system there were two areas of considerable doubt as to numerical values to be used; the strength of the materials from which the building is made and the loads to which it may be subjected. Some tests can be done to ascertain material strengths, but the results are often misleading.

At Customhouse, diamond drilled cores from the brickwork were tested in shear giving results of up to 0.5 MPa (70 psi). These are good results but have little bearing on the strength of large sections of wall particularly as thick walls will contain incomplete mortar planes, and as well, the mortar can vary in quality from dust to something akin to concrete. A design shear strength value of 0.07 MPa (10 psi) at ultimate was assumed which is the value specified in the current masonry code. The brickwork was assumed to have no tensile capacity.

The earthquake loading adopted after discussions with the Auckland City Council was that specified by the Municipal Corporation Act viz. 50% of the value specified in NZS 1900 Chapter 8, 1965 and in this case a base shear coefficient of 0.04 g.

Various factors of up to 4 were applied for design of local elements.

The seismic load, generated primarily in walls is transferred to walls at right angles to the load, by the floor diaphragms. The distribution of load to each resisting wall is based on the contributing area of the floor which is connected onto it. No attempt is made to distribute loadings because of differing wall stiffnesses, as the timber diaphragm is flexible in comparison with the in-plane stiffness of the wall.

The walls were analysed as simple cantilevers with no account being taken of the strength or stiffness of lintels over openings. Gravity loads provide precompression to resist tension caused by bending. When this analysis showed that there was insufficient compression to balance the tensions various means were used to give the lintels in the wall bending strength and change the walls from a series of cantilevers to a deep membered frame.

Walls -

Generally the internal walls were sufficient to carry the loads required without additional strengthening. A continuous steel angle was provided full length of each wall at each floor level to ensure that no concentrated loads occurred in the brickwork, and that each cantilever section of brickwork participated in resisting the lateral load.

* Associate, Holmes Wood Poole and Johnstone, Auckland.

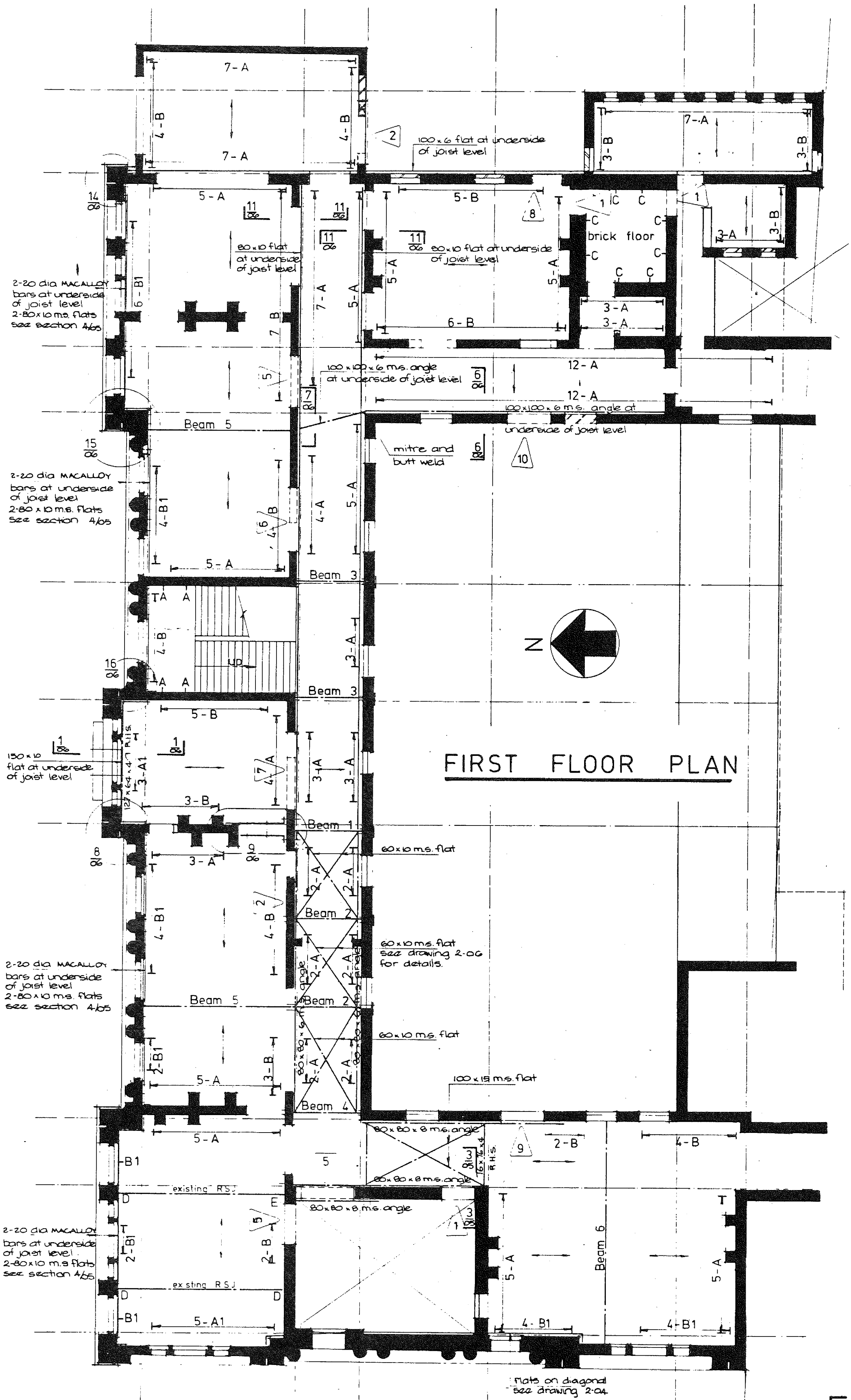

FIRST FLOOR PLAN
N
brick floor
100 x 6 flat at underside of joist level
80 x 10 flat at underside of joist level
100 x 100 x 6 m.s. angle at underside of joist level
mitre and butt weld
2-20 dia MACALLOY bars at underside of joist level 2-80 x 10 m.s. flats see section 4/05
150 x 10 flat at underside of joist level
60 x 10 m.s. flat
60 x 10 m.s. flat see drawing 2-06 for details.
100 x 15 m.s. flat
80 x 80 x 8 m.s. angle
existing R.S.J
flats on diagonal see drawing 2-04
Beam 5
Beam 3
Beam 1
Beam 2
Beam 4
Beam 6
UP

FIG. 1

Fig 2 NORTH WALL ELEVATION

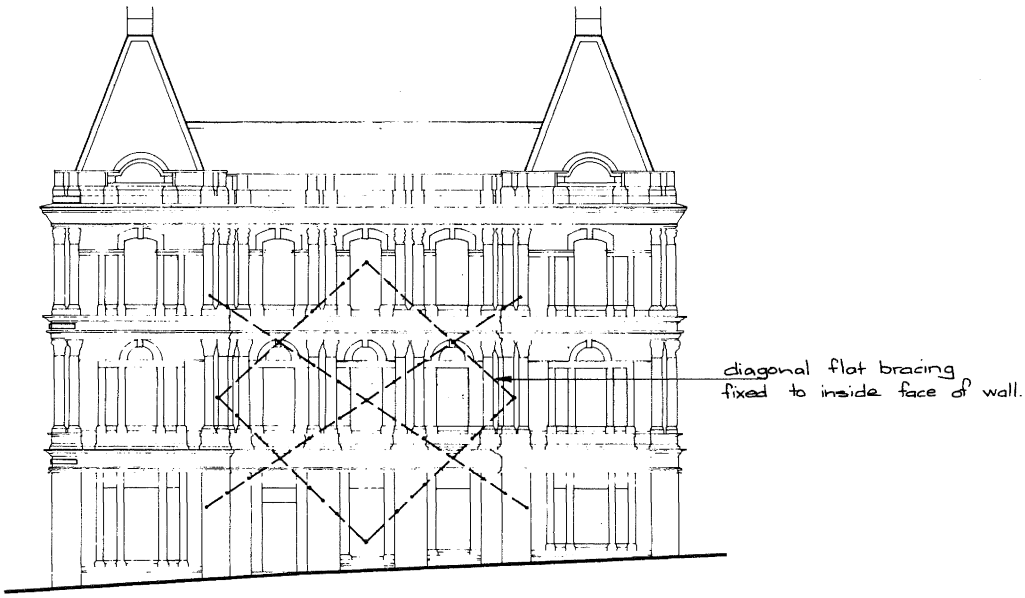


Fig. 3 WEST WALL ELEVATION

On each facade additional strengthening was required, because the walls were very heavy in comparison with the other walls as a result of the ornamentation built in and thus attracted large seismic forces.

On the north wall stressed strand provides compression in the lintels over the windows which enables them to carry bending moment and so the wall acts as a deep membered frame. The stressing strand is stressed to about 50% of the fully stressed level. The compressive forces in the walls are low, and no problems with cracking in them or axial shortening were experienced. The strand will provide extra bending resistance above the design level, as the lintel can act as a normal cracked section reinforced concrete beam would, using the reserve strength in the strand, after precompression loads have been exceeded. (Figure 2).

On the west face this was not possible because of steps in plan in the wall, so cross bracing in steel flat was applied through the lintels to give a similar action to a coupled shear wall with diagonal reinforcing. (Figure 3).

Face loads were checked, but with the thick walls having sufficient compression in them from their own weight no strengthening work was required.

Openings -

Because Customhouse was originally built as a series of cellular offices, and was now to be used as an open shopping complex, a large number of new openings were required. Some calculation and some judgement was required to finalise the extent and location of them. Where possible they were positioned vertically above each other to form the cantilevers referred to previously, and were limited in extent so as to keep the shear stresses in the brickwork remaining to the value of .07 MPa previously chosen. Where this was possible simple lintels were provided consisting of steel angles or reinforced concrete flat arches. In other cases where openings were essential in structurally important positions, a four sided frame in reinforced concrete was cast into the opening. This was designed to replace the strength of the brickwork removed, again based on .07 MPa shear stress.

Some existing openings were closed with brickwork or lightly reinforced block, where structural continuity was desirable.

Floors -

Each individual area of existing floor diaphragm was checked approximately assuming an ultimate shear strength of about 1.5 kN/m. It was found that most areas of floor were either stressed well below this value, or well above, so detailed checks of diaphragm capacity were not necessary. In cases where this capacity was exceeded plywood overlays were provided, and in a couple of particularly severe cases horizontal steel trusses were built directly on the old floor under new raised areas of floor. Plywood is particularly effective, as well distributed connections to walls can easily be arranged. As a secondary benefit it also gives a superior surface to the old flooring on which to lay carpet or vinyl.

The floors were connected to every wall wherever they touched. Steel angles and straps are used in conjunction with dwangs and plywood inserts bolted through the joists and nailed to the flooring. Bolts made from lengths of deformed reinforcing steel, epoxied into the wall were used. These were drilled to within 100 mm of the outer face of the wall. The work was carried out from the top of the floor by removing continuous sections. Chipboard was used to replace the flooring removed to give good nailing around the bolted connections. (See Figure 4). Connections are frequent, generally at about 800 mm centres, to limit the pullout force on each bolt in the brickwork.

Roof -

The roof is sarked and provides an adequate diaphragm to distribute loads generated in itself to the roof trusses which transfer them into the ceiling diaphragm. The ceiling diaphragm is connected to the walls, which generally extend just above it, via epoxied-in bolts, various steel cleats, and horizontal plywood distribution beams. At the edges of the roof, substantial angle steel cleats joint roof, wall and ceiling, to transfer face loads from the wall into the diaphragms, and to connect roof diaphragm to ceiling diaphragm. See Figure 5.

The ceiling space was the most complicated to detail because of the geometrical complexity of the timber framing. In addition the east end of the building had been constructed by a different gang of workmen from the west, each of whom had their own ideas on details. Two sets of connection details were required. See Figure 5.

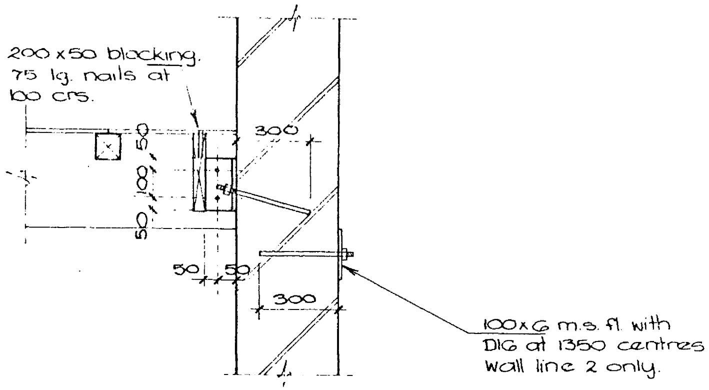
Central Tower -

The central tower was considered sufficiently important to warrant an independent lateral support system which does not rely on the brickwork for stability. This consists of a vertical steel truss on each of the four faces of the tower, extending from the top of the tower and continuing down into two major walls on which it rests. The steel members are exposed inside the tower, but have been arranged so as not to cross window openings. The trusses were designed for a load of 0.08 g. See Figure 6.

Details -

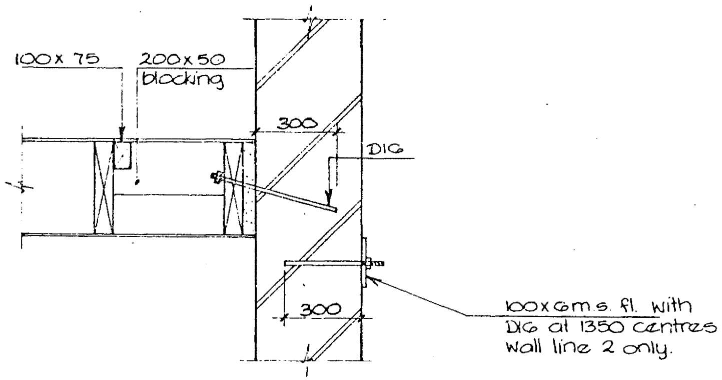
Other than the chimneys, which were behind the ridge line, parapets and other ornamentation were considered of sufficient architectural importance that they should be retained.

The chimneys were demolished down



connection A

- remove 3 boards of flooring.
- bolt 100 x 100 x 6 m.s. angle 200 lg. to joist with 2-M16 dia. bolts.
- drill into existing wall for 16 dia. deformed bar, length as shown.
- 100 x 100 dwang, make good floor with chipboard.

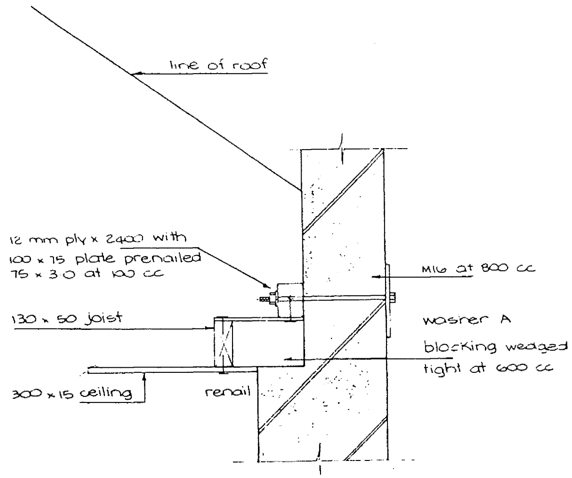


connection B

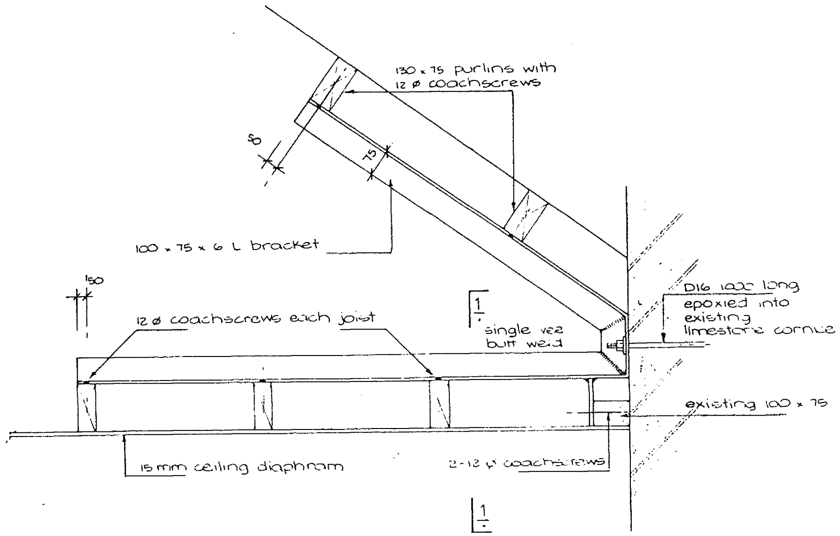
- remove flooring to second joist for 200 either side of bolt.
- 200 x 50 blocking.
- drill through joist into existing wall for 16 dia. deformed bar length as shown.
- concrete pack behind joist 400 lg. central on bolt.
- 100 x 75 timber make good flooring with chipboard.

TYPICAL FLOOR CONNECTIONS

FIG. 4



section 1



section 2

TYPICAL ROOF & CEILING CONNECTIONS

FIG. 5

to below the roof level and strengthened inside the ceiling space, with steel beams spanning between roof and ceiling diaphragms.

Parapets were secured with a continuous steel band fixed along the top and bolted through the parapet, via U shaped steel cleats which turned down each face of the top stones to provide positive anchorage. The steel band was braced back to each roof truss. Higher, walled parapets had a steelwork frame erected behind them and bolted through. Each parapet baluster is secured to the steel band with stainless steel wire to hold it from falling should it fall out of the relatively shallow pockets in which it rests. See Figure 7.

Over the head of major openings, particularly at egress ways, horizontal steel beams are provided to resist lateral loads. Vertical loads continue to be carried by the brick arch.

At corners and some wall intersections, long steel flats are provided each side of the wall, bent around the corner, and bolted together through the wall to improve continuity.

Construction -

The installation of the strengthening system was carried out without undue difficulty over a period of about 6 months. Despite much detailed measuring on site, prior to design, many did not fit so details required modification to suit the brickwork on site. This must be regarded as inevitable, and detailing needs to be flexible enough to cope with what is often quite unexpected.

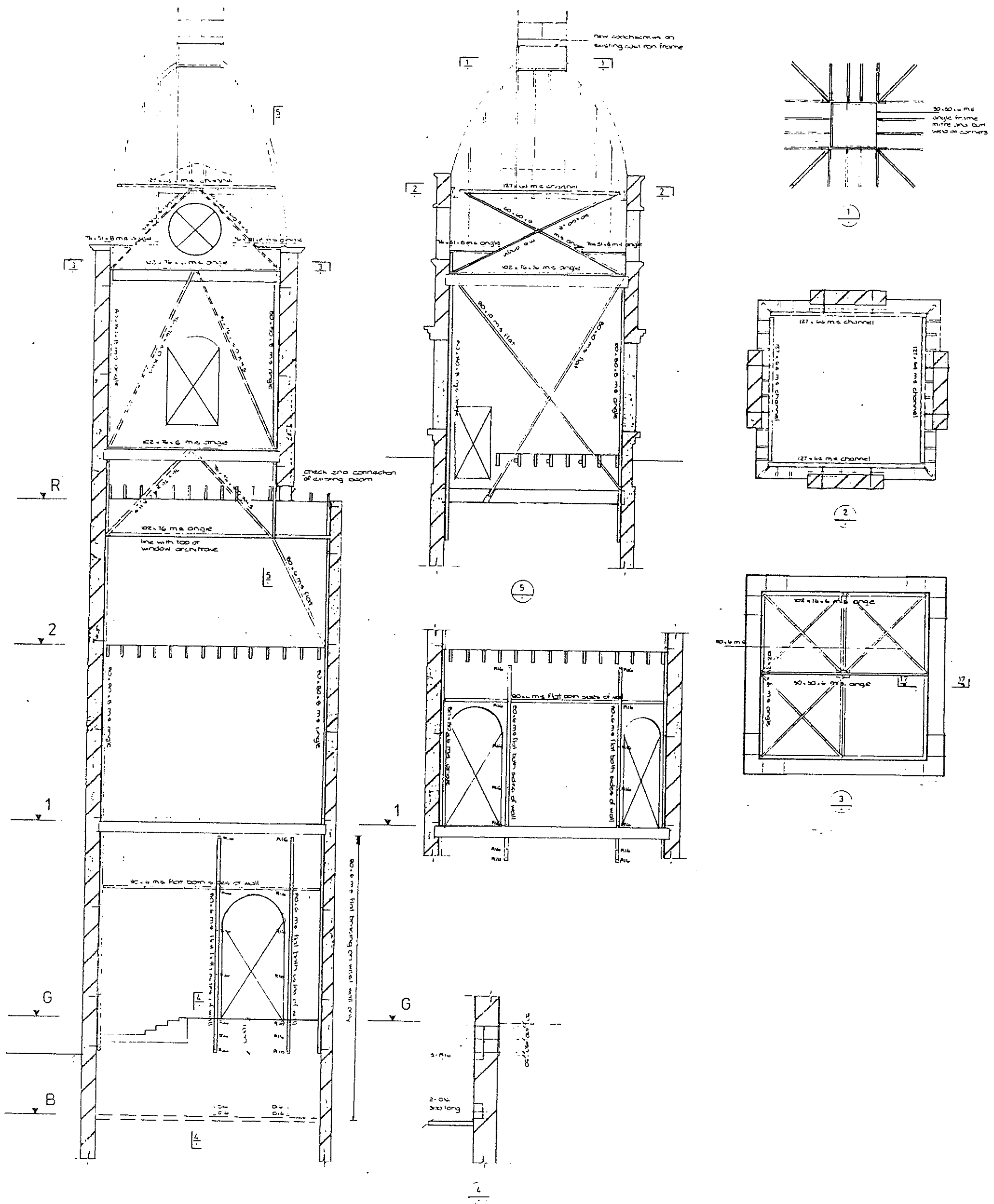
The work was carried out on a cost plus contract with agreed rates for labour and principal materials being set before the job started. Major items, such as structural steel work were tendered, with sensible contingency sums built into the contract.

The final cost of the work was approximately $300,000 of which about $80,000 can be attributed to forming new openings in the walls. The cost of the strengthening in this case was approximately $80/square metre of useable building. This sum was within the original budget allowed, but took longer to complete than anticipated. The work was highly labour intensive and most work locations were cramped so only a limited number of men could usefully work in any one area.

Conclusion -

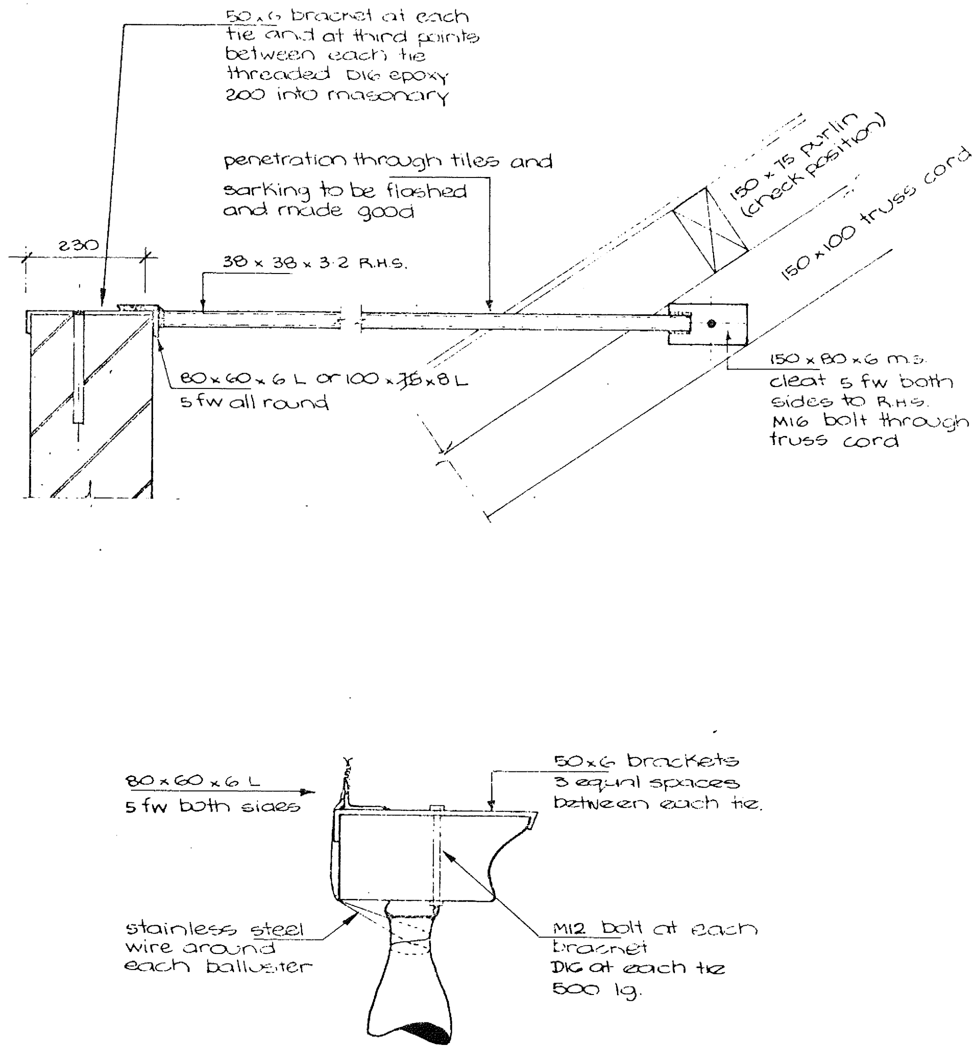
This case study has described the strengthening of an historic brick building to a standard which was acceptable to both the local authority and the owner. By making all of the available diaphragms effective, the building was changed from one which resisted lateral loads as a series of walls acting individually, to one where the loads are resisted by a series of interconnected boxes.

Had the building been designed to resist a lateral load in excess of about 0.06 g the system used would have been insufficient and additional concrete walls or cross bracing would have been required. The cost of such work would have been considerable. The level of strengthening provided gives the greatest increase in strength for the least cost, any further increases would have cost proportionally far more.



TOWER

FIG. 6



TYPICAL PARAPET CONNECTIONS

FIG. 7