CASE STUDY: CHRISTCHURCH GOVERNMENT BUILDINGS STRENGTHENING AND REFURBISHMENT

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This paper was presented at the 1996 NZNSEE Technical Conference.

SUMMARY

The Government Building in Christchurch was purchased by the Christchurch City Council in 1992, to avoid possible demolition. After a period in which no developers could be found, the building is to be strengthened and refurbished for use primarily as apartments. The strengthening scheme adopted involves demolition of two areas of the building which are otherwise difficult to use, and replacement with new reinforced concrete shear wall structures. This has resulted in more useable space, and has allowed strengthening to full code loads. Work will be completed in late 1996.

INTRODUCTION

The Government Building was designed by Maddison [1] in 1909 to accommodate the offices of many of the government departments in Christchurch. Situated immediately to the east of the Christchurch Cathedral in the square, the building is a striking example of Palladian architecture (see Figure 1), and has a category 2 listing with the Historic Places Trust.

It is a large building, approximately 78m x 21m in plan area, of three storeys plus a basement. Architectural features of the building are the long internal corridors, the main stairwell with leadlight windows, and the north and west facades. The facade had been altered significantly in 1960, when the original parapet was removed and replaced with a lower precast concrete one, in a different style. Otherwise, when the building was vacated in the late 1980's it was in reasonably original state, albeit with some deterioration, most notably at the west end, where a series of large cracks had opened up.

In 1991 the Christchurch City Council purchased the building from Government Property Services, in the face of public outcry following threats of demolition, as no developer/purchaser could be found. The Council adopted a position as interim owner until a suitable development plan could be found which would retain the important features of the building.

Following the collapse or rejection of some early schemes, the building was marketed formally, commencing in late 1992 for expressions of interest. No feasible proposals were forthcoming, in part due to the economic climate at the time, and the building was tendered in 1993. As an added "sweetener", the adjacent former Transport Board building (recently vacated) and bus park were included for possible joint development.

After a long period, three similar proposals were received later in 1993. All generally proposed use of the Government Buildings primarily as accommodation, with retail/service at ground floor, use of the Transport Board building for a hotel, and a carpark for the bare ground at rear. Essentially the City council would be required to contribute to varying degrees to the cost of structural work required before the refurbishment and fitting out could begin.

To allow comparison between the different tenders, the City Council engaged Holmes Consulting Group to prepare and cost a strengthening scheme which would be compatible with most of the tendered proposals.

Subsequently, a proposal was accepted, and work on the site commenced in June 1995. The strengthening work began in October 1995, and the development will be complete in September 1996.

PRELIMINARY DESIGN

The structure of the building was reasonably typical for its age. Walls, including most internal walls were unreinforced masonry, generally brick. Floors were reasonably heavy timber, except vaults and toilets, which were concrete, 450mm or 250mm thick respectively. The roof was 250mm thick lightweight concrete, with a bituminous membrane, leaking badly. Aside from the basement, floor to floor heights are high, up to 5700mm. Walls were unusually thick, up to 900mm for exterior walls, and 1100mm for interior walls.

The west facade is heavily modelled, with a set-back of the main wall line, and heavy stone columns spanning two storeys to support the roof.

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FIGURE 1: Government Buildings, 1956

Founding is on large concrete foundations on closely spaced precast driven piles. The soil profile shows typically silts, sands and peat down to gravel at 6.5 to 7m depth. The watertable varies considerably along the site; there is evidence that the basement (1300mm below ground level) has flooded in the past, at the east end only.

The 1960 replacement parapet had deteriorated considerably due to concrete cancer, but in any case failed to meet heritage considerations.

Because the proposal entailed a change of use, and because of the requirement to give the highest level of protection possible to the building, as nearly as practicable to full code strengthening was required.

Seismically, there were a number of concerns. The thickness and concentration of masonry walls yielded a seismic mass well in excess of a modern building of equivalent area (approximately double). Although the wall area was significant, the window heights and spacings resulted in a rocking mode of failure for the external walls, at a relatively low level of load. A preliminary analysis had indicated that an overall failure might occur at a load of approximately 25% of full code loading to NZS4203:1992 [2]. Even though spans were low, the existing diaphragms were only capable of approximately 15% of full code loading. Consequently, strengthening was likely to require significant new bracing elements, and upgrading of diaphragms. A possible solution by base isolation had been proposed at an early stage, but was

ruled out due to cost, and difficulty of achieving clearances and a rational isolation plane.

The various options were reduced to two alternative schemes:

1. A conventional strengthening scheme involving sprayed or poured concrete skin walls, and plywood overlay diaphragms. New skin walls would be reasonably evenly distributed to minimise diaphragm loads, and impact on foundations, the strength of which was unknown. Virtually the full length of the perimeter walls would be covered, at all levels. Uncertainties related to achieving continuity of walls through floors, particularly at the areas of existing concrete floors. A load level of $\frac{2}{3}$ of full code loading was used, as a practical limit on reviewing impact.
2. A more innovative approach of partial demolition, followed by reconstruction of new stiff, strong elements to take up the majority of the load, with minimal new skin walls. By removing the toilet and vault areas either side of the stairwell, the mass of the building could be significantly reduced, and the area could be opened up, allowing better utilisation of space. New concrete walls would be constructed to form large shear cores, which the rest of the building would be anchored back to. To minimise diaphragm stresses, some new skin wall elements would be added on a reduced scale at each end of the building, to the exterior walls. A load level of full code loading was adopted as being reasonably achievable.

Both schemes contained provision for strengthening of the columns at the west end, and for replacement of the parapet. In addition, allowance was made to place additional piles under the west end, in response to the settlement which had apparently occurred.

Preliminary pricing proved very interesting. Option 1 was estimated at \$3.89m, an Option 2 was estimated at \$3.94. Despite the extra work involved in option 2, the easier access for the main structural work and reduction in internal skin walls meant that the extra cost was not as much as expected. Given the benefit of the significantly increased useable area, and the increased protection available, there was a definite preference for option 2. It should be noted that the estimates also included a number of restoration items, common to both schemes, that were additional to the strengthening.

Eventually in late 1994 a commercial agreement was struck between the City Council and a joint venture known as Symphony Group/Pacific City Life Joint Venture. After some further refinement to the preliminary design, the agreement to purchase included an undertaking by the two parties to share the strengthening cost of \$3.1 million.

FINAL ANALYSIS AND DESIGN

The final documentation was undertaken in the early part of 1995. The decision was taken to use the new version of the concrete code [3], initially in its draft form. Initial analysis was done on ETABS 5.0, but there were reservations about its use, as elements of the diaphragm are flexible, particularly in the area adjacent to the main stair. The preliminary scheme had allowed to upgrade this diaphragm with steel plate or flat, but it was felt it may be possible to avoid this if the flexible diaphragm could be modelled adequately. Consequently the building was reanalysed using COMBAT, which allowed flexible diaphragm elements. This indicated that the reduced diaphragm through the main stair area had little effect on the overall building stiffness and shear distribution, but there was a disturbing lack of correlation between the two analysis packages when analysing the rigid option. Finally, the analysis was completed using ETABS 6.0, the new version with flexible diaphragm capability. A sensitivity analysis was completed to verify the distribution before settling on a model which included a light flexible diaphragm only in the central area at the main stair. Further refinement of wall thicknesses was required to allow for the proposed architectural treatment to the south wall, and the final shear distribution, which meant that the outer core walls needed to be thicker.

The final model included new concrete shear walls at the south of the corridor, and running across the building. The south walls of the new concrete cores devolved to frames at the upper level, with solid wall elements below. Skin walls were modelled as frames with composite properties to reflect the enhanced stiffness provided by the thick masonry behind the concrete. In all cases, property values were reduced in accordance with the recommendations of NZS3101:1995 [3], to allow for cracking. Typical existing and proposed floor plans are shown in Figure 2.

The new concrete core walls attract considerable load due to their much greater stiffness relative to the existing walls. Even though the demolition lightened the building considerably,

shears are high and consequently the new walls are very thick. Design was to the limited ductility provisions for $\mu=3$. As required by the code, a capacity design had to be undertaken. In some cases, advantage had to be taken of the new concrete code's increased allowable maximum shear of greater than 6 Mpa.

The foundations were of considerable concern, due to the lack of knowledge of the existing piles, some of which would be required to sustain loads under the new shear walls. New piles were allowed in the design, some of which would be required to take tension loads, always difficult in Christchurch's soil conditions.

The new concrete skin walls were also designed to $\mu=3$. In some areas a column hinging mechanism was adopted due to the relative proportions of the existing masonry piers and spandrels to the exterior walls. Diagonal reinforcing has been used where the shear stresses exceed code limits for ordinary shear stirrups.

Typically the existing walls step back by a brick thickness at each floor level, which can lead to difficulties with torsion due to misalignment of the new walls. This has been alleviated by removal of bricks below the floor to the window head level. This allows a uniform width reinforcing cage in the beams, and the column cages can crank across at a reasonable angle.

Diaphragms typically comprise heavily nailed plywood overlay flooring to existing timber t&g floors, with 100mm topping to Dycore in the core areas. The timber diaphragms are securely tied back to the main cores with heavy steel flats running through the floors into the main body of the walls. (Some further adjustment was made to the overall model to reflect the reduced stiffness of the timber diaphragms, relative to the concrete.)

At roof level, the increased diaphragm and gravity loads are to be resisted by a reinforced concrete topping, bonded to the existing slab. A concrete bond beam is to be cast around the perimeter of the roof, with steel uprights cast in to support the new parapet. The parapet is to be constructed in lightweight material which replicates the form of the original. With a height of up to 5m lightweight construction was necessary to minimise seismic loads.

The stone columns at the west face are to be strengthened by diamond coring full height and casting in prestressing cables, which are to be lightly stressed, to control cracking and provide sufficient additional strength.

Face loadings have been checked using the energy methods proposed by Priestley [4]. Due to the thickness of the existing walls, and the vertical load in them, there are no problems, even with the relatively high loads demanded by the loading code.

To take advantage of the floor to floor space available, the architectural brief called for partial mezzanine floors, set back from the windows. This required insertion of new steel beams into the existing floors. Where possible, these were used as floor ties to distribute wall anchor forces into the diaphragm.

At the uppermost level some difficulties were encountered with head height due to the steel beams supporting the roof. This

e x i s t i n g f i r s t f l o o r p l a n

Existing Concrete Floor

n e w f i r s t f l o o r p l a n

- ① New concrete skin walls 200/250 thick
- ② New concrete shear walls
- ③ New steel floor ties
- ④ New 200 Dycore floor, 100 topping
- ⑤ New plywood overlay to ex t&g flooring (typical)
- ⑥ Prestressing cables to centre of existing masonry columns

FIGURE 2 : Existing and proposed floor plans

FIGURE 3 : *View of main corridor showing longitudinal tie member*

FIGURE 4 : *Excavation around piles proceeding in new core area*

was solved by forming upstand beams over the top of the existing steel beams which could then be cut back.

CONSTRUCTION

At time of writing, construction is approximately 40% of the way through the strengthening programme. Figures 3 and 4 show the extent of some of the work already carried out. In the manner of all strengthening work, the design details have been varied considerably in many areas, but have remained true to the original concept.

Following demolition some of the existing piles were tested, to refine the design. It was found that the existing piles were well reinforced, but with a compression-only connection to the existing mass concrete foundations. The new foundation beams and pile details were adapted to pick up the full capacity of the existing piles, and hence minimise the number of new piles required. Some difficulty was encountered in driving new piles close to existing foundations, but a satisfactory result was able to be achieved with minimal variation to the design details.

The design required considerable numbers of grouted-in anchors, to bond new concrete to the existing masonry walls, and for conventional floor to wall ties. Testing of these anchors has revealed that the capacities given in the NZNSEE

“Red Book” [5] can easily be met with the embedment depths available, and the quality of the existing masonry.

CONCLUSION

The strengthening of this building was made more difficult than normal due to the peculiarly heavy construction of the building. By reviewing the needs of the developer, and the conservation requirements, it was possible to develop a solution that would offer a high degree of protection and retain the best heritage characteristics of the building.

Strengthening is always a vastly more complicated and expensive procedure than securing, but should give considerably greater protection. In this case, the final cost of the strengthening only is projected at approximately \$350/m. This equates nearly to the equivalent structural cost of a new building and is of equivalent strength.

The need to strengthen buildings such as this can be argued, but the end result is a building of high heritage value being reused in a way that will ensure that its future is assured for many years to come.

ACKNOWLEDGMENTS

The cooperation of the Christchurch City Council and Symphony Group Limited is gratefully acknowledged.

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