VOL. 34, NO. 9 : 15 SEPTEMBER 1979

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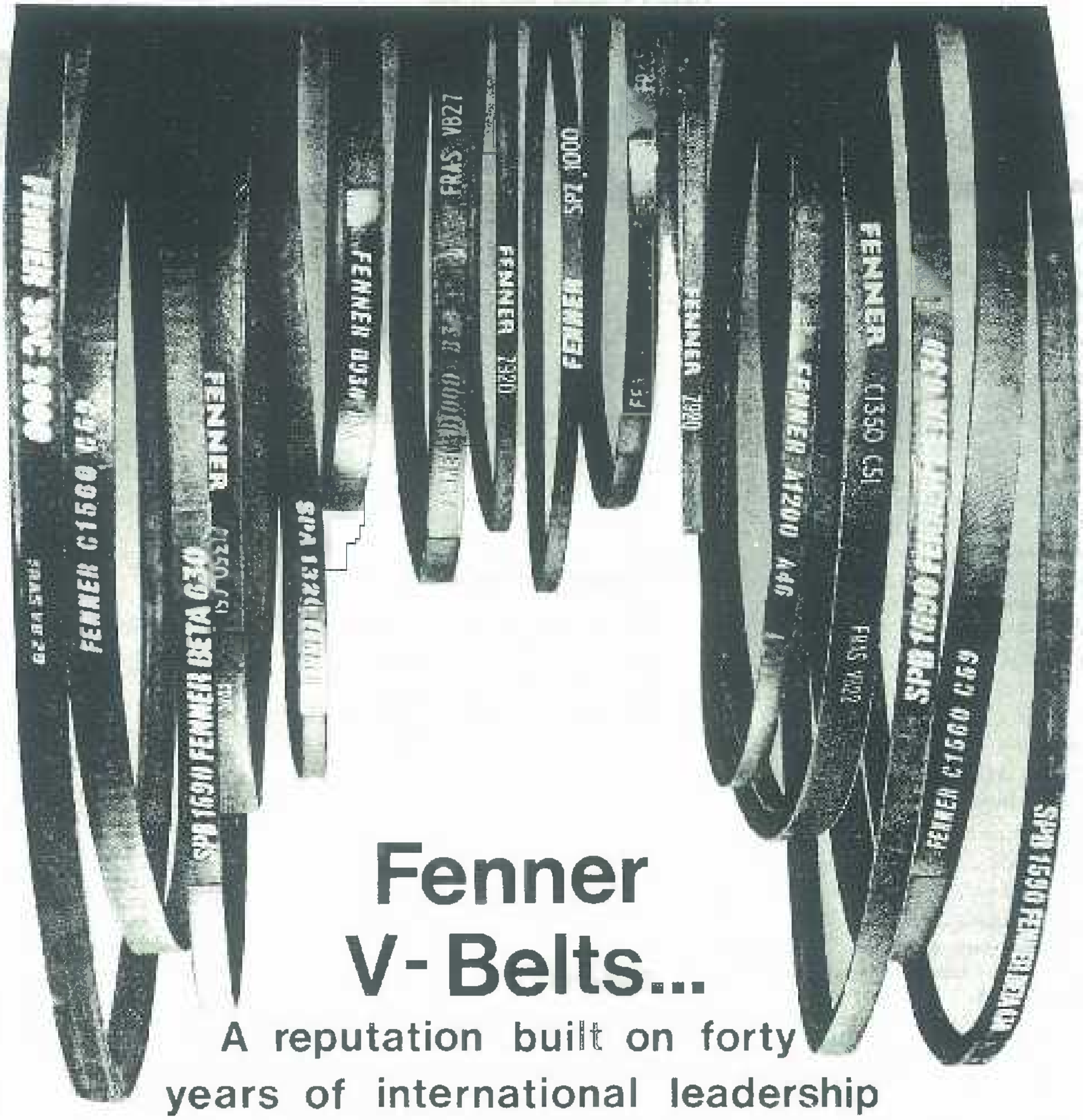
Cover picture

The ANZ Bank building in Dunedin, built in 1874, after renovation and strengthening as described in the paper on p. 198.



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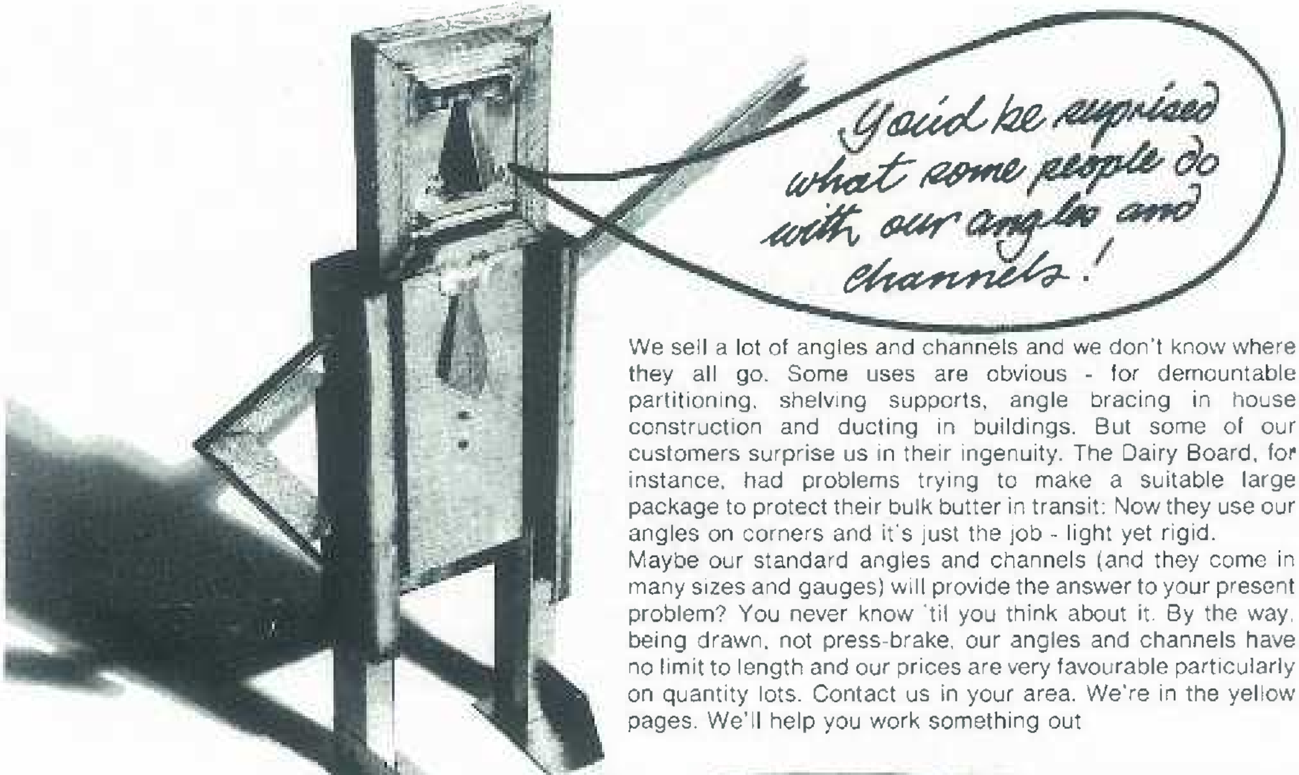

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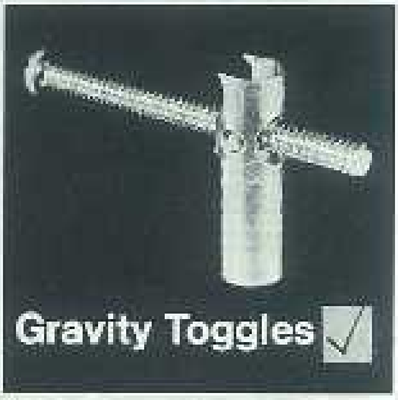

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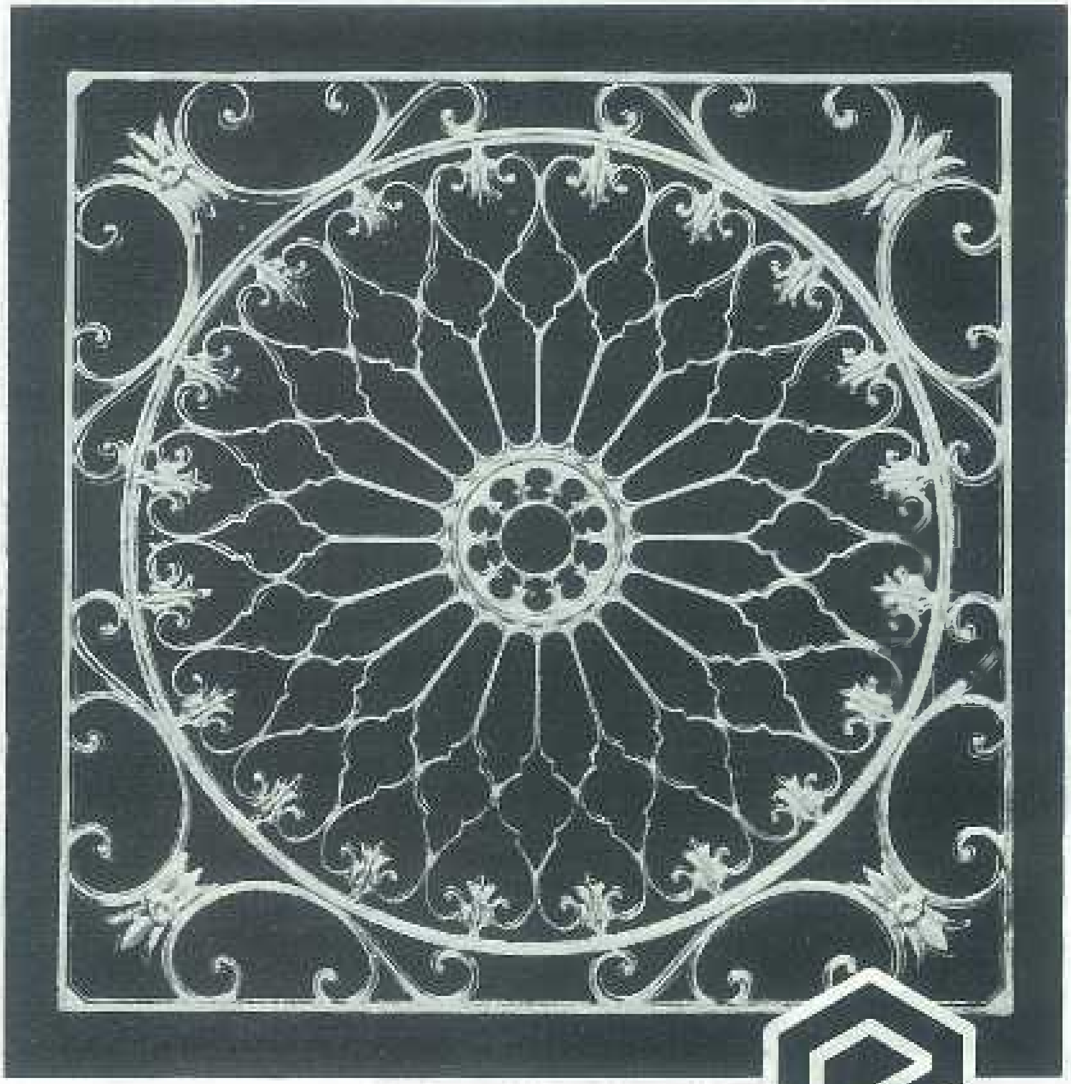

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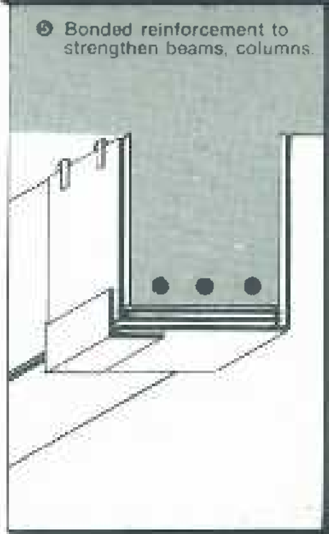
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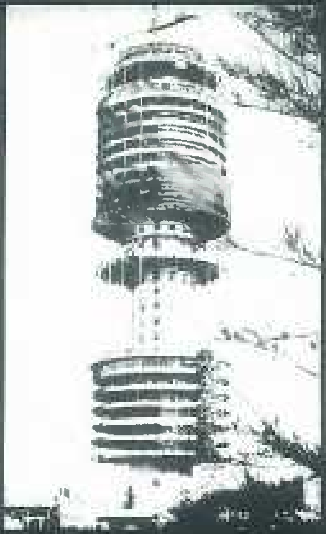

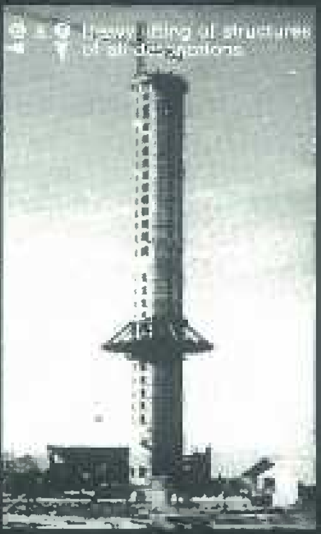
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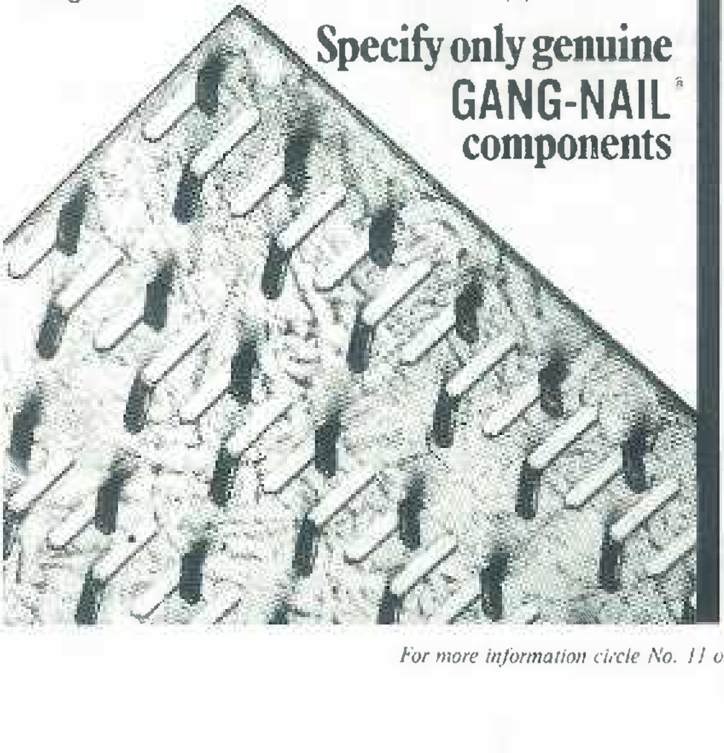

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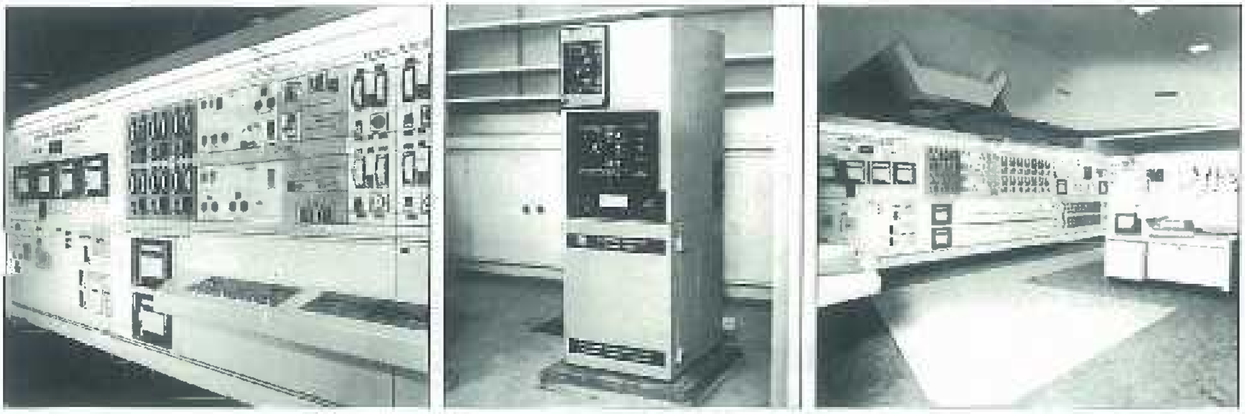

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NEW ZEALAND
Engineering
THE JOURNAL OF THE N.Z. INSTITUTION OF ENGINEERS* VOL. 34, NO. 9, 15 SEPTEMBER 1979

The advancement of engineering science

ACCORDING to our rules, advancement of engineering science is our primary goal. All our institutional organisation is orientated to it, be it qualification, practice, technical papers, standards activities, groups, divisions, branches, continuing education, engineer in society, or monitored training.

But are we "pursuing" the goal, driving forward? Is there a sharp edge to our endeavour, related to a clear perception that, at bottom, the only status and standing worth having rest on the quality of our performance as engineers.

From John Smeaton's "Society of Civil Engineers", founded more than 200 years ago, the institutions have existed to foster discussion "between men of practical science". The vehicle has been, mostly, the technical paper — a careful and thorough exposition, as befits an engineer, for the considered discussion of fellow engineers. Paper and discussion together provide the step forward, or the reference which will assist and benefit other engineers. Preparation of a paper is in itself a valuable experience in the arts of communication so essential to everyday work with the profession's clients.

The art of the technical paper seems now to be declining. Whether this is in spite of, or because of the amount of technical activity is not clear. Some of our technical groups meet for discussion. This is in the tradition. It advances engineering science, and provides the social benefit which should never be underestimated. It also provides immediacy, and our present problems may indeed have to do with the stately and extended progress of a technical paper from preparation to branch presentation, to the reviews and decision-making of the publications committee, and on to annual conference and publication.

A discussion meeting is fine for those who are able to attend. Increasingly, the smaller, and particularly the more remote branches have become disadvantaged as the technical range of the large urban branch programmes has become elaborated by the developing activities of the technical groups.

What do we need to do, then? What changes do we need to make? The experience of the prepared technical paper, its record, its objects, benefits, communication, are all too good to be thrown away. Clearly we must re-establish the art. Let us solve the problems of immediacy, disadvantage, and sharing. Engineering science committee's current consideration of a decentralised programme committee to arrange a couple of well prepared and relevant papers, with the interests of the more remote branches in mind, may well have the right direction.

Immediacy, and sharing, are questions which can be addressed in the further working out of policy for the new trilogy of journal, transactions, and proceedings which will accompany the emergence of a new *New Zealand Engineering*. Thus, equipped anew, let us ensure that the path ahead continues to be at once tested and illuminated by the high standard of the individual technical contribution of members. ▽

Victorian building reinstated ANZ Bank, Dunedin

DAVID G. COX*

M.E., F.I.E. AUST. (FELLOW)

This original building, designed by R. A. Lawson, architect for many of Otago's notable early buildings, was built in Princes Street, Dunedin, in 1874. It is one of the finest examples of early Victorian architecture which has done much to influence the character of present-day Dunedin. It had been occupied continuously by the Union Bank, later to become the A.N.Z. Bank, for a century before being vacated to allow the recent reinstatement.

1. INTRODUCTION

THE exterior of light-coloured Oamaru stone cladding and dark grey Port Chalmers basalt base had generally weathered well, but the interior had deteriorated and the double-pitch internal valley slate roof and framing were in a poor state of repair. The first-floor ceiling showed evidence of severe water damage.

Furthermore, the construction of load-bearing masonry and timber floors offered poor seismic resistance. The original lime mortar, commonly used in early masonry construction, had deteriorated and calcified, with areas of local fretting and failure.

The structure exhibited some signs of cracking throughout, although the north and west walls were generally sound. Cracking was particularly severe in the rear or east wall, where the building is at its highest. This wall was virtually separated from the adjoining side walls owing largely to a lack of bonding around corners.

The existing layout was inefficient and ill suited to modern banking requirements. Recognition of this, together with its inferior structural condition, led the bank in the early 1970s to consider demolition and redevelopment in conjunction with the adjacent site to the east. Bank staff were very conscious of the quality of the building and its importance to Dunedin's heritage, but they were growing increasingly concerned at the risk to occupants under severe earthquake conditions, and also to pedestrians on adjacent footpaths.

When strengthening and renovation were first seriously considered as an alternative to demolition and redevelopment, the client queried the degree of upgrading which could reasonably be achieved in terms of modern seismic code requirements. Following a careful analysis of previous earthquakes in the Dunedin area which indicated a maximum felt intensity of MM5 in the previous 20 years, a preliminary examination of possible structural techniques was undertaken. On this basis, the client was advised that while avoidance of damage could not be guaranteed under moderate to severe earthquake conditions, it was considered that sufficient structural integrity could be achieved to ensure the safety of occupants and pedestrians in earthquakes up to approximately MM8. This was considered acceptable in view of the lower degree of seismic activity in the coastal Otago region (Zone C) compared with other more active parts of the country. It was further recognised that in the improbable event of an earthquake of greater intensity than MM8 occurring within the extended life of the building, protection of staff and public could not be assured. However, the relative degree of risk would still be significantly less than for many other buildings throughout the city.

This proposed level of strengthening was in line with the then recent amendment to the Municipal Corporations Act (301A) which required building owners to upgrade existing buildings to at least 50% of current code loading requirements. This amendment had not in fact been adopted by the Dunedin City Corporation, but it was used as a basis for assessment and later discussions with the corporation.

In April 1974, Dunedin experienced its most severe earthquake for some 40 years, reaching a felt intensity of MM7. By coincidence, this occurred during the period when the feasibility of reinstatement was being examined. A subsequent detailed inspection revealed that the structure had suffered no obvious further damage, although some minor movements had occurred along the lines of existing cracks. This was most encouraging, and became a significant factor in the decision which was taken soon afterwards to proceed with reinstatement.

2. DEVELOPMENT OF STRENGTHENING SCHEME

In collaboration with the architects, Stephenson & Turner, a variety of alternative structural proposals for strengthening were investigated. One major constraint was the timber floor construction, since the code requirements

Fig. 1: Elevation from Princes Street.

---

* Principal, Brickell, Moss, Rankine & Hill, Consulting Engineers, Dunedin.

This project was awarded a Certificate of Merit in the 1978 A.C.E.N.Z. Merit Award Competition. It was first received for publication on 22 September 1978 and in its present form on 23 May 1979.

Fig. 2: Pediment and columns to front facade.

for fire-resistance limited such construction to two storeys only in the central commercial zone. It was considered neither practical nor economic to introduce fire-rated concrete floor slabs into the existing building, with corresponding extra supporting structure. This would also have added further heavy loads to be catered for, which was in turn undesirable. Since the area of the combined sites approached the maximum code limitation for this type of fire compartment, it was agreed to maintain the basement and ground floors only of the existing building, extending these levels into a new extension on the adjacent site. Thus the existing first floor could not be reused, which proved to be a distinct structural advantage, allowing loads to be reduced at that upper level, and making available the full height of the upper storey for part of the strengthening system.

Some additional floor area was required by the client at a later stage in the design development, and this was accommodated by including a limited area mezzanine between ground and first floor levels. This provided an additional intermediate level which could be utilised to obtain extra support to front and side walls. It had been decided at an early stage to demolish the rear east wall, since it was obviously beyond reinstatement.

The ground floor was in reasonable condition, requiring only local strengthening and an integration of local stiffening members.

Internal walls below ground level were generally load-bearing brick, and these, combined with the heavy basalt walls externally, indicated that the structure could be considered relatively stiff over this lower storey. Architectural requirements necessitated partial removal of two of these internal walls.

Particular hazards were identified as:

(1) The massive pediment overhanding the front facade in Princes Street.
(2) The four free-standing columns beneath the pediment.
(3) The parapet and cornice to the north and west elevations.

3. PHILOSOPHY OF DESIGN

The above architectural and client requirements, combined with preliminary structural aspects, established the outline parameters around which the eventual strengthening scheme was finalised.

The overall philosophy thus aimed at a practical and realistic reduction of earthquake risk, while at the same time providing reasonable protection for occupants and public. It was emphasised that this could not be proven in precise mathematical terms, since finite and detailed calculations of the ultimate strength of the building as a whole were considered to be of little practical value. Thus design concentrated on treating and stabilising particular hazards, reducing weight at upper levels, examining local elements and strengthening each of these in turn, and finally integrating all members into a bracing system to transfer loads eventually into the relatively massive and contained basement walls. The minimum basis required for specific calculations was 50% of the loads required by the then current code NZSS 1900:1965, Chapter 8, i.e., for Zone C, a seismic coefficient of 0.04. However, a coefficient of 0.08 was actually used, which, when multiplied by appropriate factors (2 to 6) when treating particular parts or portions, gave effective design coefficients ranging from 0.16 to 0.48.

4. OUTLINE OF FINAL SCHEME

The final scheme thus evolved comprised the following:

(1) The removal and replacement of the existing roof concurrent with the construction of a steel-framed diaphragm at roof level, together with the bracing of higher level pediment, parapet and cornices. The new roof structure was placed considerably higher than the original roof to provide maximum possible support to higher level features.
(2) The post-tensioning of the four front columns, front facade and pediment to achieve resistance to lateral seismic forces.
(3) The provision of a braced steel diaphragm immediately above the existing first floor, together with vertical members bracing the full height of the upper storey and the roof space. Diagonal bracing then created deep, lightly framed steel trusses in both directions, thus acting as a fully braced space frame. Individual members adjacent to wall elements were fixed at close centres to steel dowels drilled and grouted to maximum depth into the walls behind. (See Figs. 7, 8.)
(4) The reduction of all unnecessary heavy loads at higher levels.
(5) The replacement of the existing cracked east wall with a new reinforced concrete frame dowelled into and tying together the side walls, while also providing a junction and seismic separation joint with the future new adjacent building. (See Fig. 3.)

Fig. 3: New frame to east wall nearing completion; replacement of parapet commenced.

Fig. 4: Internal view from mezzanine showing exposed steel after completion.

(6) The construction of braced stiffening frames in precast reinforced concrete and structural steel inside the other three external walls.

(7) The construction of a new mezzanine floor between existing ground floor and new first floor diaphragm levels, together with associated exposed steel stiffening members around remaining walls. This created an additional intermediate level of support, since all perimeter members were dowelled into adjacent wall elements.

(8) The construction of a new concrete ring beam at ground floor level anchoring the base of precast wall columns, thus tying the whole system into floor and lower walls at ground level.

(9) The provision of two stiff reinforced concrete portal frames replacing masonry walls in basement, removed to accommodate revised planning requirements.

5. PARTICULAR FEATURES, CONSTRUCTION ASPECTS

5.1 Pediment

This is large, heavy and partly overhangs Princes Street (see Fig. 2). It was possible neither to reduce the weight nor to remove any part of it without seriously detracting from the external appearance. Accordingly, steel RSJs were fixed to anchor dowels drilled in and grouted to maximum depth within the pediment itself at both upper and lower levels. Upper steel beams followed the triangular shape and were then firmly braced back into the new roof structure.

5.2 Columns to front facade

The four sculptured columns to the front colonnade were composed of several large blocks of Oamaru stone stacked one on top of the other. Thus the whole front facade, together with the pediment over, relied solely on gravity for stability (see Fig. 2).

Early ideas suggested drilling down through the centre of these into the basement buttress walls and post-stressing. This would have meant drilling some 13 to 15 m, maintaining precise verticality, while access at the top was extremely awkward. There was also the possibility of small steel dowels at the junctions of blocks which could have created severe disruptions to drilling operations.

This was discarded in favour of drilling down between columns and corresponding piers in the front wall. This required only 600 to 900 mm of drilling through limestone at the top, and some 1800 to 2400 mm into basalt boulders in the basement buttress walls below. The need for precise verticality was not critical, and stressing cables could be more easily installed. Another major benefit was that the whole front facade was by this means effectively post-tensioned, further enhancing its overall lateral resistance. Large holes were opened up in the basement walls and dead end anchors concreted into anchor blocks. High tensile Freyssinet cables were then encased in steel pipes and sleeved through new upper level concrete beams cast into the rear of the pediment and over the front wall. They were later stressed from the top and left ungrouted for a period of more than 2 years to allow actual stresses to be checked prior to final grouting. In fact, relaxation in the central two columns averaged between 10 and 19%, and these were retensioned.

The level of prestress aimed at an average of at least 690 kPa over the whole area of column in front and pier behind. It was noticed when stressing approached this level in the central two columns that some minor cracking appeared in the flutes at joints. Originally it was assumed that all joints between blocks were fully bedded, but subsequent holes drilled into the joints proved they were only about one-third full around the outside. Thus high stresses were being concentrated around the outer edges, particularly to the flutes.

Stressing was partially relaxed while joints were grouted up, and although this was done under pressure it was not wholly successful. The surface area to be grouted in any one joint was large, with very dry adjacent stone surfaces. Despite presoaking with water through several holes, grout lost its moisture, and thus mobility, very quickly. It was estimated after grouting that joints were at least three-quarters full, and this was considered acceptable. Cracking was not significant to the two outer columns, largely owing to a greater spread of load with a corresponding lower level of effective prestress. Relaxation of stress in these was virtually negligible.

5.3 Parapet and cornice

Original design required the upper portion of the parapet to be removed to reduce height and weight, and the remainder dowelled with large-diameter deformed bars at close centres grouted into oversize holes drilled vertically down into solid masonry below the cornice. After the old roof had been removed it became apparent that some of the brickwork above the cornice was severely eroded and

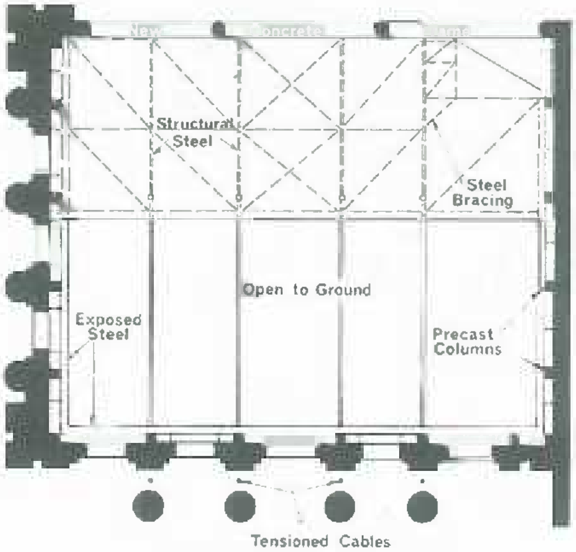

Fig. 5: Mezzanine floor plan.

weathering away. Lime mortar was also severely fretted at this level. Cornice stones were held in place solely by the dead weight of the brick and stone above them, and they were showing some cracking. Thus the whole cornice and parapet were on the verge of instability.

It was quickly concluded that the existing parapet would have to be removed down to cornice level, but not until the outer edges of the cornice stones had been temporarily supported from exterior scaffolding. Vertical dowels were then grouted down into the lower level masonry as planned, and a new reinforced concrete parapet was formed and placed. This in turn meant that it was possible to return the new parapet to its original height, which was most acceptable architecturally. An added structural advantage was that the new concrete parapet around the front elevation anchored on to each end of the pediment and also served as the anchor beams for post-tensioning cables over the two end columns. In this way the pediment derived additional stability.

5.4 Steel space frame

Steel supporting columns from first floor to upper roof level were first anchored to dowels grouted into walls. The main N-S trusses were fabricated off-site in one piece, being some 16.5 m long by 5.8 m high. They were transported to the site during a weekend and erected with the tower crane, which was by that time fully operational. This was a distinct advantage over erection in sections.

Owing to the complexity of joints where up to 10 members coincided, the structural steel fabricator elected to site-weld all connections for members in an E-W direction. Original joint details required some bolted and some welded connections, but fully welded joints with all rectangular hollow section members were readily acceptable. Thus the upper levels became a fully welded, braced three-dimensional space frame.

5.5 Internal columns

Internal concrete columns to wall elements between windows from ground to first floor levels were orignally planned to be placed *in situ*. However, the contractor requested that these be precast if possible. He advised that they would in fact be cheaper, since the confined working space made it very awkward to place concrete *in situ*. It was still essential that these be dowelled to the walls. Accordingly, sleeves were cast into the columns and corresponding dowels were grouted into the walls before precast columns were erected. These were done in two lifts for ease of handling, the intermediate junctions being covered by main structural steelwork at mezzanine level. Plate washers were placed over the dowels in recesses in the column faces, welded all round, cut off and ground down, and recesses were then plastered. The locations of dowels are not now obvious without careful inspection.

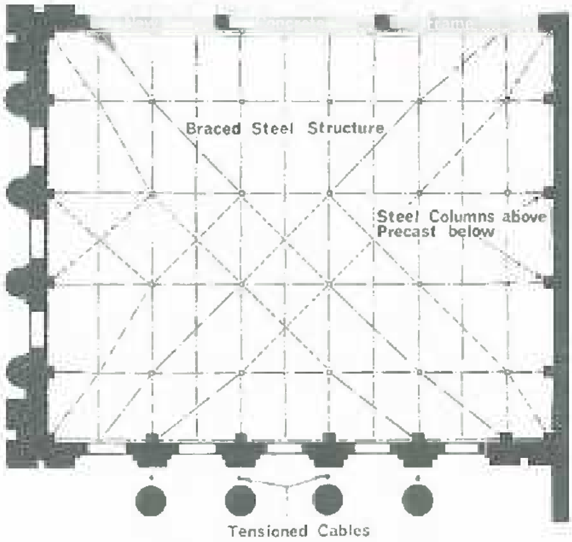

Fig. 6: First floor diaphragm plan.

6. CONTRACT CONSIDERATIONS

Naylor Love Construction Ltd, a local construction company, was selected as the contractor because of previous experience in the renovation of older buildings. The contract was negotiated on the basis of cost plus fixed fee, together with a total target price. Such work is highly labour-intensive, working within the constraints of existing structure and walls, and hence tends to attract higher than normal unit rates.

No specific construction difficulties were experienced apart from some of the inevitable unknowns connected with old buildings. It is essential to retain sufficient flexibility to modify details where necessitated by actual conditions once exposed. Likewise, a reasonable level of contingency is necessary for such contracts, since existing details often vary markedly from those indicated even on original drawings if these are still available.

The contractor elected to provide a tower crane on the adjacent site which proved far more satisfactory than frequent use of large mobile cranes operating from public thoroughfares. He also decided to retain the existing first floor until late in the contract period, which afforded him a convenient working platform for all upper level steelwork and roofing. This had the added structural advantage of providing some stability to external walls until the roof framing was virtually completed. (See Fig. 7.)

The overall scheme proved practicable and did not create unreasonable requirements of the contractor. It was recognised that this was a progressive upgrading and that, while each step added some strength locally, the scheme could not be considered satisfactory until all stages had been successfully completed. This is an inevitable consequence of this type of reinstatement. Local members or portions of the building may thus be more vulnerable over short periods, but when viewed in the light of a recent estimate that the return period for an earthquake of MM7 intensity in the Dunedin area is approximately 800 years,[1] it is a low risk. Consequently, the contractor was not required to go to elaborate lengths of temporary bracing for short periods until permanent members were fixed in place.

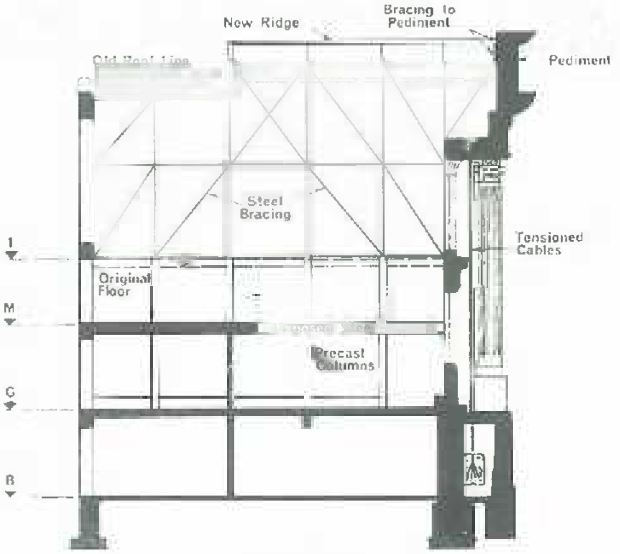

Fig. 7: Section.