

EARTHQUAKE STRENGTHENING OF OLD MASONRY WITH REFERENCE TO THE AUCKLAND FERRY BUILDING

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ABSTRACT:

The paper describes Auckland's seismicity and the history of the Auckland Ferry Building completed in 1912. Strengthening of the structure's foundation as well as non-structural components is described with emphasis on internal drilling and steel placement techniques. Fire and Egress requirements are also considered.

SECTION 1

AUCKLAND SEISMICITY:

1. We refer for background to several diagrams/graphs presented by D.S.I.R. Geologist, W.D. Smith, at the 1981 Napier Conference - "Large Earthquakes in New Zealand".

Figure 1. Shows the expected Mercalli intensities with a mean return period of 50 years.

Figure 2. Shows the mean return period (years) for Mercalli intensities VIII and greater.

Figure 3. Shows return periods or frequencies against Mercalli intensities.

The solid lines show the 1981 tentative new results.

These show the adjustments to the local expected seismicity from the latest research. However Smith tempers all his work with caution due to the inherent inaccuracies in all seismic assessments. Refer N.Z.N.S.E.E. Bulletin June 1982 (Vol. 15 No. 2).

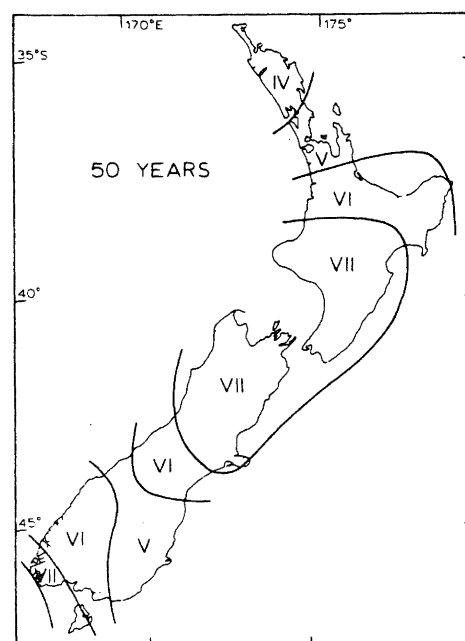


Figure 1

THIS RETURN PERIOD FOR A NORMAL BUILDING IS TAKEN ON 50 YEARS BEING A GENERAL BUILDING LIFE. THIS CURRENTLY IS TOO SHORT!

"Experts must advise to the best of their ability, but it is up to society or their elected representatives and perhaps others to decide on the acceptable risk. I also believe that those who decide on the acceptable risk must be prepared to take the responsibility for that decision. Unfortunately the apparent willingness by some to accept higher risks before an event is not matched by the same eagerness to come forward and take responsibility when things go wrong." -

QUOTE O.A. GLOGAU - LATE CHIEF STRUCTURAL ENGINEER, MINISTRY OF WORKS.

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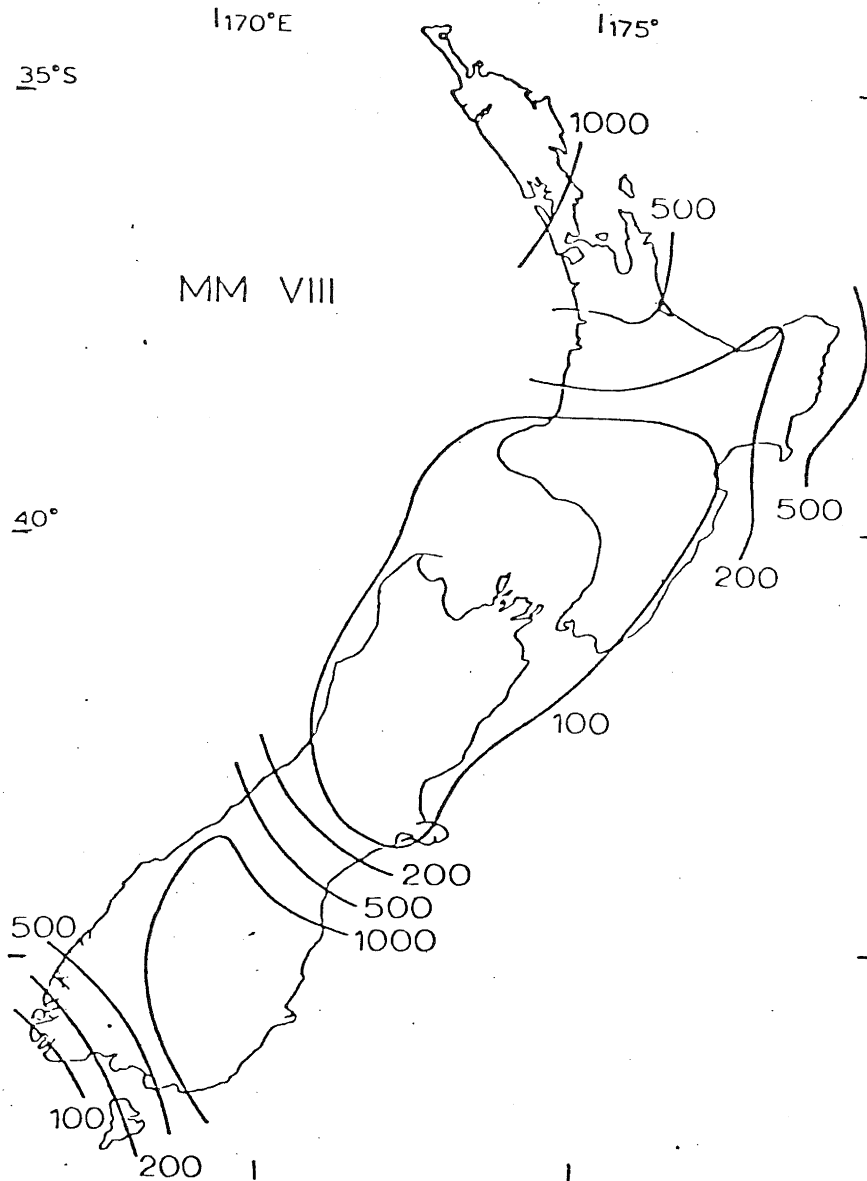


Figure 2

With respect to restoration and conservation of buildings, earthquakes with intensity of MM7 or greater are of interest. We note that the predicted mean for Wellington is 20 years but the last MMVII has been in 1942 - 37 years ago. Before that there was one in 1914 - 28 years previously and then in 1904 - 10 years before. Another earthquake 9 years earlier almost reached MMVII.

It must be concluded that quiescent periods evidently vary with periods of greater intensity.

W.D. SMITH CURVES

LARGE EARTHQUAKES IN NEW ZEALAND
CONFERENCE - NAPIER 1981

GROUND ACCELERATIONS	MM7	0.05 - 0.10g
	MM8	0.10 - 0.20g
	MM9	0.20 - 0.40g

W = Wellington
C = Christchurch
A = Auckland - - -

NOTE
THE

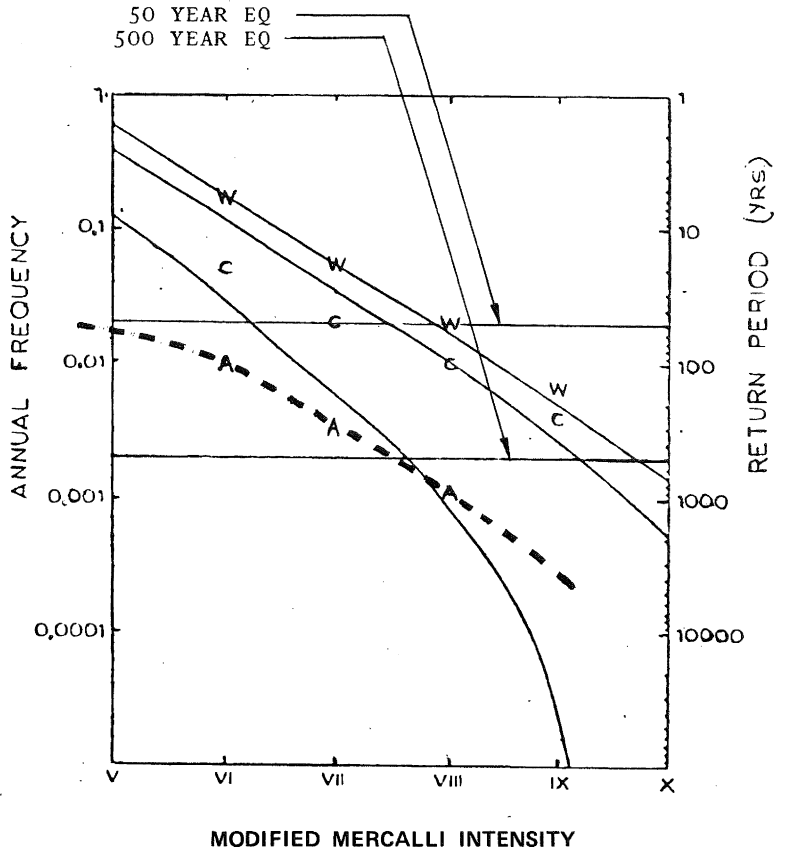


Figure 3

2. The risk of a particular Mercalli level earthquake occurring is as above, however the chosen level of acceptable risk in New Zealand is based on work embodied in a Californian Code, ATC III.

Also public surveys in the United States has indicated a public acceptance of an involuntary risk as being necessarily 100 to 10,000 times smaller than any other voluntary risk. In practice generally this means reducing the life threatening non-structural damage to less than 1 in 10 during the life of a structure which for a new building is considered as 50 years.

Using statistics

$$p = 1 - e^{-T/R}$$

which is graphed as -

We can assess for an extended design life the return period of the earthquake to be considered. For a new building this implies consideration of a 500 year Earthquake hence for an extended design life the return period could be construed as even longer.

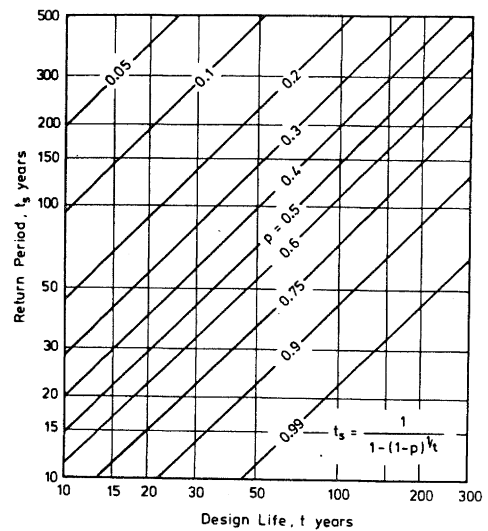


FIG. 4 : RELATION BETWEEN RETURNED PERIOD t_s , DESIGN LIFE t , AND PROBABILITY OF EXCEEDENCE P .

3. To accumulate the information to make the above assessment, requires major effort on behalf of client and engineer.

A list as suggested by one engineer could be as follows:

The Brief:

What are the owners

- intention; e.g. for extensive renovation?
- interest in the building e.g. commercial only?
- responsibilities for the building/land - moral obligation only? or

(statutory - as owner only? Subject to Council requisition?)

- options (does owner have option to offer for sale to local council)

What are the facts of the present situation?

What is the present condition and status of the building?

What are the objectives of the proposed work?

- limited extension of useful life? what desired use/occupancy of building?
- preservation of building (or parts) for future generations? (implies ability to survive future damaging earthquake with masonry features at least repairable).
- priorities should be assigned to historic features, if building is historic. Not all features should be seen as worth retaining.
- Is the owner able to consider and eventually set objectives or is this a matter for some other authority, e.g. city planning proposals?

What is NZNSE clarification?

What is Historic Places Trust category - what are its historic architectural features?

Discussion with a client to clarify this type of information early in any project we suggest will be very difficult and should not be necessary. We do not deny that it would be ideal if the data is readily available.

4. Legislation/Codes

Looking at any Typical 1910 Building:

- i. If a building is considered under the District Planning Scheme as having both architectural and community significance it can also be expected that the Historic Places Trust would consider it under Category B., i.e. "Those buildings which merit permanent conservation because of their very great historical

significance and architectural quality". Being generally constructed in unreinforced masonry the building could also be the subject of requisitions issued by the Council under authority of the Local Bodies Amendment Act 1978 Section 624, which act gives the council authority in the interests of public safety to require a building which does not comply with current by-laws to be improved to meet specified requirements.

- ii. The standards required by this council regulation are not onerous. They generally look for half the standards of NZS 1900 i.e. 'The standards applying for new buildings in the late 60's and these standards in the Auckland area are a good deal less severe than the current standards for new buildings (i.e. NZS 4203/76) and less severe than some comparable overseas standards, e.g. Los Angeles City Ordinance "Earthquake Hazard Reduction in Existing Buildings". These L.A. standards appear to us to be say 1/2 to 2/3 as severe as NZS 4203/76 whereas the Auckland City Council requirements can be as low as 1/16 of the requirements of NZS 4203/76. SO, AS IN L.A. THE UNDERLYING PHILOSOPHY OF THE LOCAL REQUIREMENT IS TO REDUCE DANGER TO LIFE WITHOUT ATTEMPTING NECESSARILY TO PROTECT THE BUILDINGS CONCERNED FROM DAMAGE.

i.e. Earthquake design so called, is design work (on drawing board) TO PROTECT THE OCCUPANTS OR PEOPLE.

- iii. This is achieved by taking advantage of the fabric of the building to absorb the earthquake energy. This is fine with new materials which can be built with some toughness and resilience and so the damage incurred can be localised and depending on the acceleration and amplitude of the earthquake - minimised. But older structures of masonry are brittle, they have no toughness (little resistance in walls of brick, often minimal foundations, built on unconsolidated fill and timber floors that too often burn out and hence remove support to the already cracked masonry walls).
- iv. There are no current regulations to enforce the keeping of a building intact after an earthquake. If your design concept is for the building to stay elastic, the chances of damage in the design earthquake will be reduced. However, this ignores the risk imposed by the greater than design level earthquake. Many buildings are being strengthened by adding walls reinforced for ductility but the rigid "historic features" are expected to "ride along" and somehow during the wall degradation, avoid breaking down. Hence even though life is protected a strengthened building in a future event is highly likely to suffer physical damage to the fabric.

v. The Ministry of Works and Development, M.W.D. have always acted immediately on any worldwide research they thought relevant and hence they have appeared to work to a higher standard than the private sector and this no doubt has been encouraged by the fact that Government buildings are not insured. Hence if as technical advisor to the Local Authority Loans Board they are asked to comment on a loan application, they often are asking for higher standards than what the private sector would use.

vi. HOWEVER, when considerable sums of ANY money is considered for the use of preserving or restoring an old building a logical attitude must include the concern that a high degree of protection shall be given to any 'historic features' on the building i.e. it is not just the removal of the risk of collapse but any work done should prevent physical damage and "loss of that visible characteristic which is the reason for its preservation and even listing as a historic place". Hence the legal requirement for earthquake protection is not enough to protect a historic building. Preservation of any monument or something to be valued and passed on to future generations will cost a lot more than NORMAL costs.

vii. If strengthening is not practical, then accepting the risk presented by the unstrengthened (and even un-refurbished) building (perhaps with some main hazard reduction measures only) may be the better way in the long run.

Normal economic pressures of course will continue to be at work to ensure that the buildings life is not indefinite (not forgetting the gradual breakdown of any building fabric, without maintenance. However, we would suggest that with the legal requirements for strengthening being pitched as low as they are, the above is a NON-option.

The San Fernando quake of 1971 showed how almost any minimal measures improved tremendously a building's resistance to the earthquake hence work done with fire and general weather upgrading will often be enough to satisfy the Local Bodies Amendment Act 1978/Section 624.

5. CHOICE OF A DESIGN EQ FOR THE FERRY BUILDING

The building is:

- In Zone C
- A low squat structure (base period (excluding tower) .2-.3 secs)
- An inherently brittle structure (ductility of one).
- The expected damping (10%) hence a higher amplification could occur.

- On a filled site. (Sedimentary rock at depth but piles penetrate 6 m of unconsolidated fill).

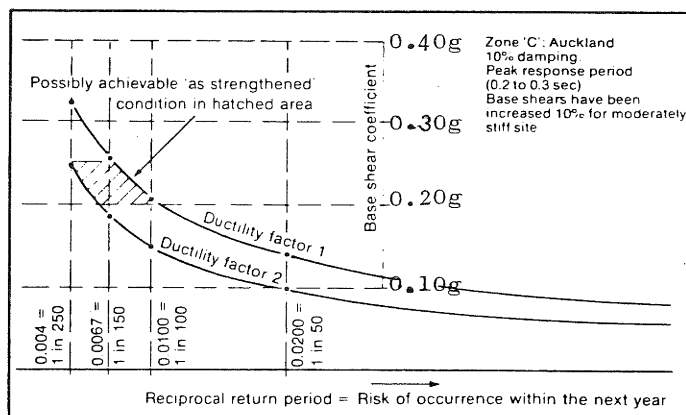


Figure 5

W.D. Smith's work produces estimates of risk related to Mercalli Intensity and these, in turn, can be related (not at all precisely) to ground acceleration.

The total dynamic force on the building is determined by the base shear coefficient. As compared to ground acceleration, the base shear coefficient will be:

- increased by dynamic amplification through the building itself
- reduced by any ductility (the ability to tolerate damage) in the building.

Base shear will be somewhat larger than ground acceleration.

Figures 4 and 5 are based on work by the Group on Seismic Design of Bridges of the N.Z. National Society for Earthquake Engineering. (NZNSE Bulletin September 1980).

In later sections of this paper, we discuss the order of strength which seems achievable by inserting new reinforcing steel into the fabric of the Ferry Building. It does seem to be possible to strengthen the building so as to bring it within the hatched area of Figure 5.

SECTION II

HISTORY OF AUCKLAND FERRY BUILDING:

1. 1853 Sir George Grey - 1st open boat ferries.
- 1863 Holmes Bros. - Steam Service initiated.
- 1872 Auckland and North Shore Ferry Company formed.
- 1881 Devonport Steam Ferry Company amalgamation.
- 1904 First R.C. Piles driven to combat TEREDO.
- 1907 Commencement of new Queen Street wharf Design underway for new shipping office - The Ferry Building.

W.H. Hamer M.I.C.E., Engineer to the A.H.B. reports:

"The building will be an architectural addition to the waterfront and provide excellent accommodation in an admirable position".

Architect Mr A Wiseman appointed.

- 1908 Foundations completed by A.H.B. 5 storey scheme reduced to 4 storey. Builders commenced on site - Messrs Philcox & Sons.
- 1910 Completion of the new Ferry Tees.
- 1912 Completion of the Ferry Building.
- 1913 Completion of Queens Street Wharf.
- 1915 Reclamation of Mechanics Bay.
- 1917 Reclamation of Freemans Bay.

2. SPECIFICATION/ALLOWABLE STRESSES:

The Ferry Building commenced construction in 1909 and the historical background seems to indicate that there was fairly extensive use of brickwork in major buildings in Auckland at that time. Most of the trade skills and many of the tradesmen were of British and Australian origin. The specification is a thorough, professional document and implies an experienced architectural practice. It calls for:

- * Appointment of a Clerk of Works with clearly limited discretion responsible to the Architect.
- * GROUND LIME MORTAR - One part lime, and three parts cleaned red scoria, very finely ground. The whole to be ground in a mortar mill erected on the site, driven by an engine.
- * LIME - Auckland Hydraulic Stone or other approved manufacture, free from dirt and impurities.
- * CEMENT MORTAR - One part cement, and two parts washed sand (unless specified richer elsewhere). "All piers supporting girders. All 9" parapet walls".
- * BRICKS - All bricks, whether as facings or backings to be good, hard, sound, square, and well burnt, even and uniform in shape, free from cracks, flaws and other defects, giving a clear ring when struck. Backing bricks to be machine pressed Auckland bricks, of approved make and size, and equal to sample to be seen at Architect's office. Facing bricks to be approved red pressed bricks even in colour. Allow £6.50 (six pounds five shillings) per 1000 on Auckland wharf.
- * H.B. BONDING - Build in to strengthen the walls of single storey wings, tower and internal brick walls of Main Building.

H.B. Bonding (Cooke & Buddle, Agents). Allow for one strand to each half brick in thickness and two rows to each floor (three rows to brick walls of tower) to be well jointed and lapped at angles.

It is reasonable then, to assume the materials and workmanship were of a standard comparable to those of British and Australian buildings of similar importance built at the same period. Typical service load basic COMPRESSION stresses used in design in those countries in the 1950's and 1960's

were:

- New buildings: Cement or cement/lime mortar: 1.6 MPa (234 psi)
- New buildings: Hydraulic lime mortar: 1.2 MPa (174 psi)
- Old buildings: Cement & cement/lime mortar: 1.0 MPa (145 psi)
- Old buildings: lime mortar: 0.6 MPa (87 psi).

No doubt these figures are quite conservative because they were intended to cover a wide range of buildings with different degrees of control on workmanship. The figures for old buildings would have incorporated some allowance for typical degrees of weathering, settlement, damage etc.

Generally, however, a careful physical appraisal of the building showed the brickwork to be sound and built using good techniques. Joints were tight, approximately ¼" and all joints viewed were solid filled even in the thick 39" tower walls. There was no sign of a marked quality reduction of either bricks or brickwork in the middle of the walls. Most brick-work is in Flemish and English bond and a retired Master Tradesman, Mr Alf Way commented "the whole job gave him a good feel of a job well done".

3. STRUCTURAL PROPERTIES - as sampled - refer Section IV - 5

SECTION III

REINFORCED MASONRY DESIGN:

1. The first priority in the design of reinforced concrete and reinforced masonry is to ensure that the adjective 'reinforced' is indeed warranted. This is so whether or not earthquake resistance is an issue.

A plain (unreinforced) concrete element has some definite strength which is determined by the cross-section geometry and by the material rupture strength. But this is a brittle 'single-shot' strength. If rupture occurs the element is thereafter useless.

2. There is then a threshold or minimum steel content implication in the term 'reinforced'. The yield strength of a lightly reinforced element depends on the cross-section geometry and on the yield strength and disposition of the steel. If the yield strength provided is much less than the rupture strength then the element should not be called 'reinforced'. For it will

still be prone to brittle 'single shot' behaviour.

3. If the yield strength as defined by the steel is about equal to the rupture strength as defined by the concrete (or masonry) cross-section, then the 'first shot' strength will be about the same as for an unreinforced section but it will remain intact and available for many subsequent shots. This defines the 'minimum steel content' design.
4. In the design of new buildings it is fairly common to use relatively fine but heavily reinforced elements. Steel content per square or cubic metre may be 10 to 20 times the minimum value defined above but this is done so that quite small elements can carry heavy loads. For new buildings these authors think that this tendency may have been carried to an extreme. Buildings with relatively larger masonry elements can contain less steel in toto, are stiffer, easier to build and offer enhanced thermal storage capacity.
5. Old buildings, such as the Ferry Building, already have many massive masonry elements.

If these can be converted from 'single-shot' brittle elements by the inclusion of minimum content steel, then that alone may confer a respectable level of strength on the building as a whole.
6. There are, of course, other issues along the same philosophical lines as the 'minimum content' issue. Buildings should have 'integrity'. That is, they should behave as single entities and not as collections of 'stacked-up rubble'. This depends on the connections between elements and it is sensitive to the detailing of those connections.

Steel must be placed so that it can be effective. Bond and anchorage are crucial - perhaps, even now, we do not fully understand just how crucial.
7. There are a number of approaches to the 'strengthening' of historic buildings:-
 - (a) The 'particular hazards' approach.
 - (b) The 'new primary structure plus tiebacks' approach.
 - (c) The 'knit the existing fabric' approach.
8. The 'particular hazards' approach simply removes (often by demolition) specific 'particular hazards' such as high chimneys and parapets. Of itself, it does nothing at all for the bulk of the building. It may, for some buildings, be an appropriate short-term expedient

pending finance for a more thorough approach but it is then necessary to take care that it does not compromise future options.

9. The 'new primary structure' approach is the one that seems to have generally been contemplated in seismic areas around the world.

It is indeed tempting to 'cut the Gordian knot' by inserting some wholly new reinforced concrete or steel structure designed to the best modern standards. One can then be fairly confident that the new primary structure will itself remain standing after an earthquake. But what of the fabric? Is it enough to -

- a) design a fine new well-engineered primary structure

AND THEN

- b) regard the historic fabric as architectural icing to be tied-back to the new structure with occasional bolts epoxied into drilled holes?

10. Certainly in the case of the Ferry Building, the present authors think that such an approach would not be enough. The historic/aesthetic value of the Ferry Building is almost entirely external. If the elaborate and heavily moulded facades are lost along with the tower then everything is lost. The authors take the view that the first priority must be to mitigate the risk of collapse of the facades and, as near as possible, to provide them with a degree of integrity approaching what one would aim for in a new building of comparable scale and detail.
11. This leads to the 'knit the fabric' approach which is dealt with in more detail below. This approach has been extensively used in Europe for the repair of the great Gothic cathedrals and, subsequently, on many famous historic structures including bridges as well as buildings.
12. Philosophically, this requires that one regard the existing masonry as a predetermined outline to be reinforced as well as can be done, in accordance with modern understanding. In particular, one will seek to provide 'minimum content' steel, to tie elements together and to suppress bond and anchorage failures.
13. This requires a great deal of careful detailing at a local level. This work is, in the first instance, calculated to solve local problems i.e. to provide just the minimum amount of steel required to keep elements in place tied to each other. However, if the totality of such work is sufficient to lift the overall strength of the building up to some figure approaching modern standards, why then, add some new primary structure?

14. This does not mean that there are no new concrete or masonry elements in the proposals for the Ferry Building. Rather new elements are added to solve specific local problems.
15. The Ferry Building has both massive facades and also an internal 'egg-crate' arrangement of load-bearing masonry walls. These amount to shear walls with generous flanges. It seems obviously essential that wall/wall and wall/floor inter-sections be positively held together. Traditional masonry bonding is not enough so small reinforced concrete elements will be used to enclose and confine the intersection area. In these areas the appropriate content of secondary steel should be, at least, of the same order as that used in gravity loaded concrete columns.
16. Elsewhere there are locations where the egg-crate arrangement contains serious structural omissions. One obvious example is in the ground level facade at the South East corner where there were originally large shop-front windows. Most of these would be infilled to leave a fenestration pattern comparable to the rest of the building.
17. The current reinforced masonry codes generally seek minimum steel contents of the order of 0.1% (1 in 1000) of the gross masonry area in each direction. This is the sort of figure we have aimed for. There are some walls where, with an eye to strength/stiffness relationships for the building as a whole we have budgetted for two to three times code minimum content. Nevertheless, the overall average content for the whole building would be decidedly closer to code minimum content and, of course, much less than would generally be used in a new ductile frame building of comparable scale.
18. Nevertheless, minimally reinforced masonry can provide a lot of strength if there is a lot of masonry. For the Ferry Building our calculations suggest that it is possible to develop a base shear strength of the order of 0.7 MPa (100 psi). There is scope for much argument as to whether this is an appropriate design figure, bearing in mind all the esotericisms of the 'new' loadings code and current seismological thought. However, the point is, that it is not a figure which we prescribed in advance but rather an outcome of working through the building element-by-element looking at just what details seemed appropriate to provide reasonable integrity.
- The outcome figure can be expected to vary somewhat in the process of refined detailed design but it cannot be drastically changed within the context of the present approach.
- To set a much lower overall target figure would be of marginal effect unless combined with a policy decision to leave a substantial proportion of the masonry unreinforced.
- To set a much higher target would require much more new primary structure. It would also have major foundation, functional planning and cost implications and, in our opinion, would likely prove a self-defeating exercise.
19. Most of the structural calculations relevant to overall capacity were done at about the time the draft of the new masonry code was published. We did consider assigning some fraction of the gross shear (say 0.2 MPa out of 0.7 MPa) as 'carried by the masonry' although we would consider it inappropriate to use this approach to reduce steel below the minimum content figures. Rather it may be considered as an argument to modestly increase the calculated shear capacity resulting from a given steel content. We recognise that this is a somewhat artificial argument brought about by present code shear provisions which are based on masonry strut action at 45°. It seems obvious that strut angle will vary with steel content to the effect that minimally reinforced sections will be stronger per unit of steel provided than will be heavily reinforced sections. We look forward to the day when it will be possible to calculate this in some rational fashion.
20. There has been a good deal of laboratory testing of new reinforced masonry in shear recently. Much of the results are quite heartening. However, the reports are usually content just to tabulate the gross shear stress developed as an average over the whole area together with photographs and/or qualitative descriptions of the crack pattern. Given the expense of testing, we would have thought it possible to glean a good deal more understanding by constructing quantitative equilibrium models of the major forces in the steel and the masonry at 'ultimate' load. There are a number of reasons for this:
- Strut angle and reinforcement efficiency may well vary with steel content as mentioned above.
 - Shear strength may also be related to global boundary conditions. Aspect ratio is obviously important but perhaps flanges and floor slab diaphragms are just as important. Present knowledge does not consider such factors.
 - The lack of equilibrium models may be clouding our understanding of the role of bond and anchorage and hence the importance of detailing styles. Perhaps better detailing would provide better strength and ductility at given reinforcement content.
21. Certainly there is a great need for more

testing and, in particular, for testing situations typical of those arising in the restoration of historic buildings. These should use the historic masonry materials (under fired clay bricks from demolition sites, fat lime mortars etc.) combined with practicable modern methods for inserting reinforcing steel.

22. There has been extensive testing of the brickwork cut from the Ferry Building, including panels up to 2 ft. (600 mm) square. Details are in laboratory reports issued by the University of Auckland Applied Research Office and our own reports. These are, in the first instance, confidential to the Harbour Board, but no doubt suitable arrangements can be made through ourselves or through the Chief Engineer to the Board. Some of the conclusions are summarised in an article in New Zealand Concrete Construction of April 1982:
- a) Crushing strengths are surprisingly high; 12-20 MPa.
 - b) Young's modulus is surprisingly low at 1 to 2 GPa.
 - c) Strain capacity is high - probably in excess of 1%.
 - d) Creep is low as compared to elastic strain.

The stiffness modulus is about 10% to 20% of that expected of modern brickwork and seems to be influenced by both the bricks and the mortar. It does seem to be consistent with qualitative observations made by various authorities working on European buildings. (Fondedile, Ove Arup).

23. There has also been some work on field survey techniques for assessing variability of masonry quality over the extent of the building. These were based initially on procedures developed jointly by SEAOSC and Los Angeles City Council.
24. There has not yet been any experimental work on the effect of inserting reinforcement into existing fabric. At one level this would involve basic research which cannot reasonably be charged to one particular project. However this need not preclude some future testing of specific reinforcement details unique and/or crucial to the Ferry Building.

IV RESTORATION OF FACADE:

- 1. This is a major area in its own right and it is not possible to do justice to it within the limits of the present paper. In budgetary terms, it is a substantial proportion (say 25-30%) of total costs.
- 2. The embellishments and decorations of the facade and the tower are of

Pymont 'yellow block' sandstone shipped from Sydney specifically for the Ferry Building. These elements are often massive e.g. the 5 feet wide window sills and the 3 ft diameter columns. The colour can be seen in places beneath peeling paint and is about the same as Sydney Town Hall.

- 3. The building has always been in a saline environment but it seems likely that this is a good deal less significant than the fact that for the first 40-50 years it was surrounded by coal-burning ferries and locomotives. The combustion products would have included sulphur oxides and, in wet weather, sulphuric acid and various sulphate salts would have been deposited.
- 4. With aging and indifferent maintenance the weathering envelope (flashings, gutterings, drains, coatings) has deteriorated and dampness has penetrated extensively into the stone and brick fabric.
- 5. Various salts, particularly the sulphates mentioned above, are carried in solution through the body of the masonry. When dampness reaches evaporation surfaces these salts are precipitated and disrupt the stone to a depth typically of the order of 20 mm in much the same way as ice forming beneath a road disrupts the surface.
- 6. About thirty years ago some of this damaged stone was cut back and repaired with cement plaster. The extent and scope of these repairs are not fully known. All of the stone was then painted with a proprietary 'waterproof' coating.
- 7. The use of any such coating needs to be approached with great care. In general there is a very real danger that coatings can aggravate problems. Unless the coating has superb elastomeric qualities, it is inevitable that water will enter through various cracks and then be trapped behind the coating. The stone will not be able to breathe and (more important) salts accumulating on surfaces will not be flushed away by rain.
- 8. There are many similar buildings in Sydney with many similar problems. Over the last 5 to 10 years there have been more than a dozen major stone facade restoration projects. The care, skill and attention going into these has been of the best available quality and budgets have run to several million dollars per project. In the process a great deal of expertise has been built up in the building professions and trades. With regular maintenance, the refurbished facades are not expected to require any major attention for another hundred years.

9. On the Harbour Board's instructions, we have retained Dr George Gibbons of Sydney to advise us on the Ferry Building. He has spent about a week carrying out an extensive detailed survey and presented an extremely thorough and detailed report complete with recommendations.
10. It seems that the Ferry Building is about in the mid-range of deterioration as compared to similar buildings in Sydney. The present coating should certainly be stripped and it may then be possible to leave the stone permanently exposed with new coatings limited to near horizontal surfaces such as sills and copings. Deteriorated stone would be cut or faired back and the stonework flushed with water to remove salt accumulations. Rain-water provisions would be cleaned out, replaced or amplified. It is not generally proposed to reconstruct the original stone profiles in their original positions or to bring in any substantial quantities of replacement stone. Buildings, like people, can be expected to show their age and some may find this part of their charm. Where the original profiles are considered architecturally vital, it may be possible to reconstruct them with ground stone mixed with resin.
11. However, much of this depends on the extent and degree of the work done 30 years ago and it is not possible to finalise matters until the existing coating has been stripped. At the worst, a new coating over some vertical surfaces may prove unavoidable.
12. A final caution to those contemplating the use of cement pointings or plasters on old masonry. Cement is, of course, strong and impervious and, where there is an historic accumulation of salts, this tends to redirect and amplify the attack on adjacent stone and some bricks. The Pyrmont/Balmain area of Sydney is very close to White Bay Power Station and one of the authors has seen many cases where cement pointing has worn down adjacent stone 30-40 mm within a few years. Lime mortars with appropriate carefully-chosen sands seem to act as sacrificial agents and attract salt concentrations. In this way it is possible to regularly replace lime pointings and plasters as they break up. Initially this can be quite rapid but over, say, a five year period with several applications, it is possible to remove the bulk of salt concentrations which may have taken a century or more to build up. Water flushing, of course, accelerates the process and may be appropriate when it is otherwise acceptable.

SECTION V

STRUCTURAL DETAILING:

A. The Adaption of Fondedile Techniques To Achieve R.M.

1. Introduction -

The Fondedile S.p.a. of Naples has been involved in the repair of ancient buildings and monuments in Europe for about twenty years. It has a British subsidiary, 'Fondedile Foundations Limited' which is associated with the British contractors, Sir Robert McAlpine Limited of London. The portfolio of work carried out by the Fondedile organisation is very impressive and we must assume that it has established an excellent reputation with European governments and other responsible organisations such as 'national trust' authorities.

2. Engineering Philosophy -

Many of the structures Fondedile has dealt with have been massive ancient masonry structures which were 'in extremis' or close to serious collapse at the time. They have reached this condition with the accumulation of weathering, foundation settlement damage and site instability movements over hundreds (or thousands) of years. These effects have often been substantially accelerated in recent times by the construction of other adjacent works and the effect of these, for example, on local geological and geohydraulic conditions.

The underlying philosophy for the design of remedial works involves an acceptance that the existing masonry fabric together with the complex underlying geological materials constitute a structure which, self-evidently, is able to carry normal loads with a factor of safety larger than (but probably only just larger than) one. These materials then are regarded as pre-existing assets not only in an aesthetic sense but also in a strictly structural sense even though their engineering characteristics may not be quantifiable. The aim of strengthening is to supplement them adequately but with delicacy and finesse disturbing the pre-existing state of stress as little as possible.

THE FONDEDILE APPROACH is to install a three-dimensional lattice of reinforcing steel bars grouted or cemented into small drilled holes raking at various non-perpendicular angles through the body of the superstructure and the underlying ground.

There are subtle issues involved. For example, the complex non-perpendicular reinforcement arrays are a good deal more difficult to appreciate in quantitative design terms than the simple perpendicular arrays used in new buildings.

This does not necessarily mean that they are less effective. Indeed, one has a 'hunch' that they may often be more effective.

3. Typical Reinforcement Of Internal Walls to Ferry Building -

Reinforcement of these walls is directed at three objectives -

- a) Walls function as webs linking major masonry flange elements and with these, constituting massive vertical cantilever beams of 'I' and 'C' configuration to provide primary earthquake resistance. Horizontal steel serves as shear reinforcement for such cantilever beams.
- b) The webs must be restrained from out-of-plane buckling under the effects of in-plane shear and/or out-of-plane seismic shaking.
- c) Even if the walls are severely damaged by in-plane shear during the duration of shaking they should still preferably retain capacity for carrying gravity loads at, perhaps, a reduced margin of safety.

The Fondedile approach has been adapted and combined with spaced reinforced concrete ribs. The purpose of these are:

- a) To provide a zone where lap-splices of horizontal steel can be expected to perform with a higher degree of reliability during shaking.
- b) To stiffen the wall against lateral buckling and out-of-plane shaking.
- c) To provide a more reliable path for gravity loads in the event of severe damage to the intervening lengths of wall.
- d) To provide somewhat distributed 'shear-lugs' to assist in suppressing bed-joint sliding.

This internal reinforcing was subsequently replaced with bars in a gunite curtain for reasons of cost. However, the general intention of strengthening was still retained.

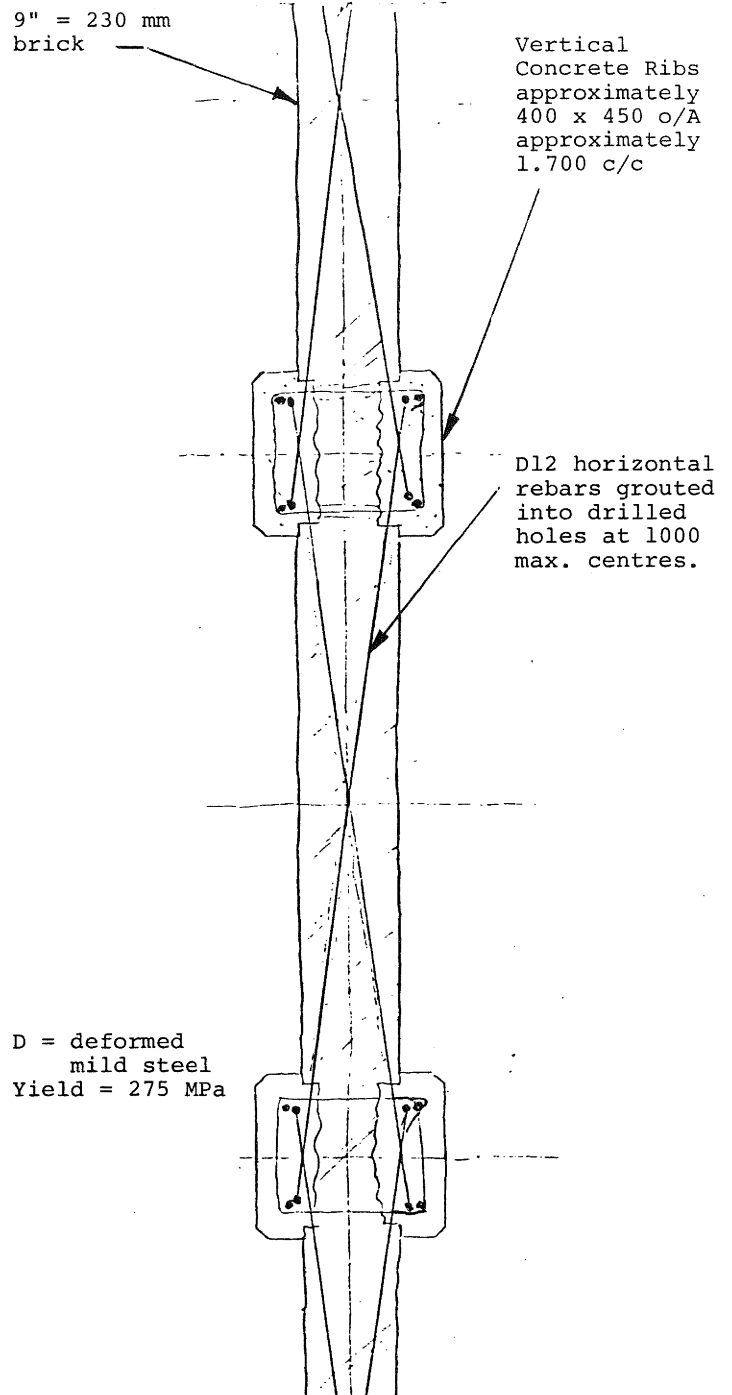
4. 'Typical' External Walls -

In this content the word 'typical' is somewhat questionable because the external walls are very complex and variable.

(i) Flange/Web 'Haunch Ties'

The first priority must be directed towards the risk of separation of the facades (considered as flange areas) from the internal walls. A 'typical' situation involves an external concrete rib placed in the space between double sandstone columns. The size of the external rib should be the minimum consistent with the placing of dense, compact concrete in this critical exposed location. Vertical bars should be properly lapped and anchored into the massive cornices below first and third floor levels.

This seems to be the minimum

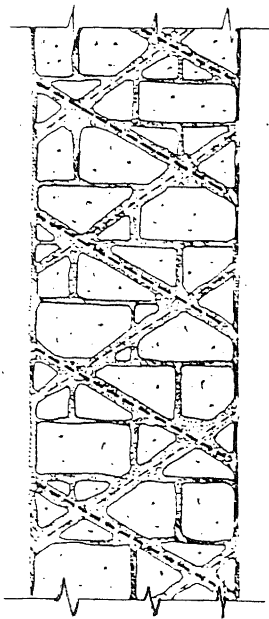


FERRY BUILDING PLAN

Figure 6

structurally acceptable intrusion of concrete onto the outer face of the Ferry Building. If one accepts the view that tie-back to some internal structure is not, on its own, sufficient for such massive facades then it seems that a 'minimum cost' structural solution might require a good deal more concrete on the outer face. There must be some content of in-plane steel somewhere near the outer face and such steel can only be contained in new concrete or in the existing masonry.

(ii) In-Plane Vertical Reinforcement of Existing External Walls -



15

Figures 6 and 7 show typical reinforcement grouted into drilled holes at an angle to the vertical. The bars are not envisaged as having any particular positive anchorage. This curtain provides several effective reinforcing bars at each point over the height of the walls with bars lapped in staggered pairs. The bars can combine with a 'lazy tongs' effect and these curtains of steel can be selectively positioned in the facade and will be effective:

- a) As flexural and shear-friction clamping steel for primary earthquake resistance in the plane of the facade.
- b) In providing resistance to vertical face load bending moments.

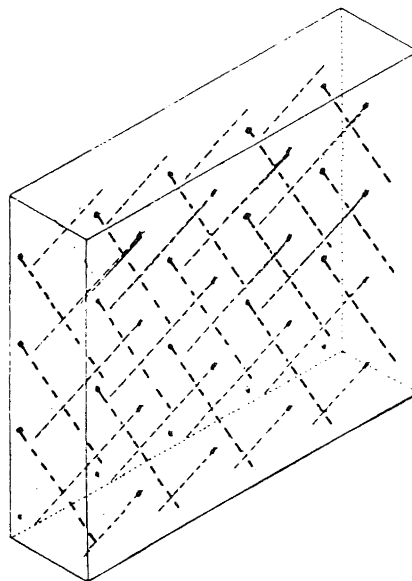


Figure 7

TYPICAL FONDEDILE 'CURTAIN'

(iii) In-Plane Horizontal Steel -

The Ferry Building does have a regular pattern of openings occupying about one half of the total height. Opening reveals, sills and heads all provide potential for drilling access and advantage can be provided by two curtains of bars drilled through from one reveal to the next. Each curtain should be located close to a wall face and each should contain at least one half of the code minimum content steel: i.e.

$$0.05\% = 230 \text{ mm}^2/\text{m} = \text{D16 at 900 say for 430 wall}$$

$$\text{or D20 at 1200 say for 430 wall}$$

$$= 460 \text{ mm}^2/\text{m} = \text{D20 at 700 say for 920 wall}$$

$$\text{or D24 at 1000 say for 920 wall}$$

These bars should have some positive

anchorage to secure the (full thickness) stone window jambs back to adjacent walling. But their main function is as shear reinforcement to wide piers and we do not believe that the calculated strength as shear steel needs to be reduced to account for threading at the ends.

The inside face horizontal bars could be contained within a minimal thickness (say 50-60 mm) gunite curtain. The cost of such gunite would, to some extent, be offset by the reduction in drilling and grouting costs, and the incidence of damage to windows and joinery would be reduced. Such a gunite curtain would provide better resistance to out-of-plane shaking. It would also provide for any design increases in shear steel over and above 'minimum content steel' without any further drilling. Eccentricity effects would not seem significant even if the inner curtain is substantially heavier than the outer because of the greater material stiffness of the gunite.

Finally, there are the existing 9 inch x 7 inch (225 x 175 mm) rolled steel joists which support the floors and span at right angles to the facade often onto spandrels above window openings. If these are dislodged by severe spandrel damage then the floors may lack gravity load support. This risk can be substantially reduced by incorporating local stiffening ribs and lintels into the gunite curtain (minimum practical size say 400 x 200 including gunite thickness).

(iv) Horizontal Steel In Depth of Spandrels -

Outer face horizontal steel in the depth of the spandrels serves to reduce damage to the spandrels themselves and also as shear steel within the width of piers.

The approach of Section ii is, in principle, applicable but needs substantial local adaptation to deal with the intricacies of the outer face.

Alternatively we would detail a curtain of bars close and parallel to the external face but angled on elevation (i.e. in the vertical plane) so that the holes can be drilled from openings. This approach does seem to be a practical one below the first floor cornice because of the many arched openings there. It does not seem to be practical above that level, simply in terms of drill access and length.

(v) Vertical Bars in Spandrels -

Provide straight bars in outer face only (inner face gunite curtain) drilled through sill to head. Positive anchorage each end.

0.05% = D16 at 900 or D20 at 1200 -
460 wall
= D20 at 700 or D24 at 1000 -
920 wall

(vi) Delamination Bars -

Drilled and placed straight through walls at right angles to prevent separation of components (sandstone brick, gunite) and encourage composite action. Identify individual and reasonably substantial stone blocks (say larger than 0.05 cubic metres) and secure individually wherever possible.

B. Masonry Drilling - By Fondedile

An experienced drilling organisation such as the Italian or U.K. Fondedile firms have much expertise in placing reinforcing bars into masonry.

Their grouting technology, anti-corrosion and drilling techniques are 1st class and much of their equipment after local adaptation is capable of higher performance than its maker's originally schemed.

e.g.

Van Moppe - hydraulic drive
jig mounted
diamond cylindrical bits
water lubricated
non impacting
original capacity 10' now
drilling through the St Pauls
masonry 40' in depth
(horizontal)

The drilling is either (a) CORE DRILLING electric or hydraulic drive.
Diamond bits, wet or dry lubrication

or

(b) HOLE DRILLING

tube drills, generally jig mounted to the scaffold 10-30 hp drive
impacting or non impacting
wet or dry or no lubricant
and air, electric or hydraulic driven

Their capacity varies from:

Atlas Copco Rock drills (20-40 mmø) -
16 m/drill/day

to

Diamond, non coring, non impacting
(small holes) 6 m/drill/day

and the Atlas Copco rate of course would increase for smaller diameter holes and vica versa.

Some examples of drilling by the Fondedile organisation:

- i. Old 17th century stonework:
600 mm thick walls
20 m long/75 mmø/cylindrical drill
reinforced with 30 mm
MacAlloy bars. Cement grouting.
- ii. 16th century brickwork:
13½" brick walls
1.2 m long/30 mmø
reinforcing with 12 mmø m.s. bar
sheradized for anticorrosion
cement grouting
- iii. St Pauls Church, London:
Walls up to 200 mm thick
12-5 m long/75 mmø/cylindrical
diamond drills
- iv. Alexandra Palace:
Brickwork strengthening
(Nth London ex B.B.C. T.V.
studios)

Post Fire Damage: Piers to 1300 mm thick
holes .5 to
2.5 m deep/30 mmø
reinforcing 16 mmø
H.Y. ungalvanised.

.5m hole	14 mins.	Set up & drill steel, supply & fix, grouting.	£16.5 (\$39.30)
1.0 m hole	27 "	"	£22.0 (\$58.40)
1.5 m "	45 "	"	£29.5 (\$70.20)
2.0 m "	80 "	"	£42.5 (\$101.20)

We have no evidence that these costs could be achieved in New Zealand but these rates showing a productivity of 14-15m/day/ drill and 22m/day is good for the smaller holes.

v. Ferry Building

Some of the largest schemed holes using the 75 mmø bit for accuracy 14 m deep. 14° down from horizontal estimating, 10 m/day.

Set up and drilling time 1½ days
 Steel and grouting ½ day
 Tolerance: with a cone of 200-300 mmø

To allow for inexperience in this drilling:

the lack of specialist equipment and the greater difficulty of access,

all drilling on the Ferry Building was estimated at a more conservative figure than the Alexandra Palace and other estimates shown.

2. Reinforcing Steel - By Fondedile -

- use mild steel
- stainless steel staifix, staibond
- or mild steel sheradized (coating of powdered zinc)
- or mild steel galvanised
- or even glass fibre

Corrosion proofing is important and must be carefully assessed. If the masonry is porous, pre-grouting is necessary until the voids are sealed. Entry and exit points of the drill bit is important and bar anchorage location to be assessed to minimize the external impact. Grout staining to external finishes is to be avoided and removal of grout splash must be immediate.

Selection of hole entry points to coincide with jointing is advantageous and diamond coring for plugs is often done. Collection of the dust gives a material source for filling the plug ring.

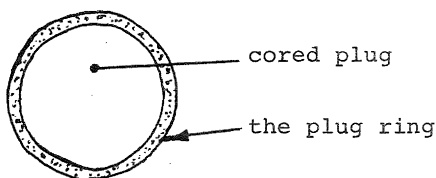


Figure 8

3. Masonry - Grouting By Fondedile

The use of cement grouting or epoxy grouting to seal voids and protect the reinforcing against corrosion. Nothing will cause quicker deterioration of the masonry than rusting of the internal reinforcing. However, the surface for pointing or plastering lime mortars to minimise any interference with the migration of water born salts are used at every opportunity.

SECTION VI

FIRE AND EGRESS COMPLIANCE:

(On the Board's instructions we retained MacDonald Barnett Partners, Engineers, to advise on this matter).

1. Compliance with NZSS 1900 Chap. V shows the following:

- i) Central Fire Risk Area
 Type 2 Construction
 Floor plan area 10,065 sq.ft. (935m²)
 Four stories

The fire compartment size complies with Div D.1. (Commercial) and D2 (shops, stores and restaurants).

- ii) Floor fire ratings of 1½ or 2 hours is achieved by the existing 150 mm thick under-floor concrete slab.

Wall fire ratings requirements are readily achieved by the massive brick and masonry facades. Window openings are not large and the allowable percentage of opening is satisfactory.

- iii) Partitions to inter-tenancies are sub-standard.

Smoke Stop Doors to exit ways need upgrading.

2. Corrective work on the above is generally non structural and would include:

- (a) Sealing of floor spaces around miscellaneous ducts to negate fire bridging between floors.
- (b) Some thickening of fire protection systems around steel beams.
- (c) Add fire rated boards to inter-tenancy partitions and wired glass to the high level windows.
- (d) Improve smoke stop doors.
- (e) Add roof space fire stops or expose the roof to the 3rd floor.

3. Primary Egress is by the main stairway located centrally around the lift well. This is augmented in the existing building by a timber stair at the West end and a strictly non complying "ships ladder" at the East end. To avoid "cul de sac" situations the East end ladder would need rebuilding as a stair and the West stair would need some winder treads rebuilt. The exit ways capacity is satisfactory for a usage:

- (a) Factory manufacturing
- (b) Workrooms (such as clothing manufacture)
- (c) Offices
- (d) Shops
- (e) Schools

however, any use such as restaurants, clubs, cabaret, dance halls or museums would require a widening or relocation of existing stairs.

4. In conclusion it is evident there are no major problems to be overcome in upgrading this building to meet acceptable fire and egress standards.

Whatever approach is adopted will to a greater or lesser extent, depend on the type of occupancies that are adopted and the overall economics of any total refurbishment and structural upgrading.

VII FOUNDATIONS:

1. Test bores and geotechnical investigation for the Ferry Building were done by Brickell Moss and Partners under the direction of David Hollands and Andrew Brickell. Brickell Moss also made available earlier reports on reclamation history of the Downtown area compiled for the Harbour Board at the time of the adjacent Downtown (Quay to Customs Streets) Scheme.

Subsequently Dr Tam Larkin of Auckland University provided a second opinion on liquefaction effects at the Ferry Building site.

2. Rock (Waitemata series) occurs at about 8½ m below Quay Street level and dips gently towards the harbour. There is about 5 m of non-compact clay fills overlying the original estuarine sands.

3. The Ferry Building foundations up to underside of brick wall haunchings were built by the Ferro-concrete Company of Australia at about the same time (1908) as that company was building Queen's Wharf. It was a period of intense major expansion of port facilities. Subsequently a separate contract for the building proper was let under the supervision of Architect Alex Wiseman.

4. The foundations comprise:

- (a) A stepped mass concrete seawall (base thickness 4 m) under the North wall and returning under the East wall and also on a line about 6 m outside the West wall.
- (b) A high level grillage of large reinforced concrete foundation beams supported on:
- (c) Driven precast concrete piles.

5. A diamond core taken from the sea-wall shows sound concrete cleanly seating onto hard siltstone. The bottom 400 mm of concrete is somewhat softer but it is not known whether this results from saline effects or from tremie placement. The base of the wall is about 4 metres below low tide level.

Two shallow test pits were dug to a depth of about 2 m. The clay fills were damp but not wet and easily excavated by hand. No tidal effect was noted. The precast piles seemed in excellent condition with no signs of rusting or spalling.

6. The facades are presently supported on the North and East sides by the sea-wall. The remaining facades plus the interior are pile supported. Clearly then, the existing foundation has a major seismic torsion problem with the shear centre of the system located close to the North East corner.

7. The North sea-wall retains sands and clay fills to a depth of 9 metres over a length of about 70 metres between returns. The factor of safety of overturning of this wall is marginal even under ordinary static (non earthquake) conditions.

($K_0 = 0.6$. High tide. 1 metre lag)

The geoseismic pressures to be developed during earthquake cannot be determined with precision. Some liquefaction is likely but the effect and extent of this is sensitive to the soil properties in-situ and to the degree of variability over the site. Current expert opinion seems to tend to the view that the overall effect cannot exceed the hydrostatic overturning produced by a heavy (19 kN/m^3) fluid acting over the full height of the wall.

8. It is also considered that liquefaction may lead to settlement of the fills to a depth approaching half a metre. The present ground floor is not suspended so post-earthquake egress/evacuation may be difficult.

9. The new strengthening works proposed have two major components:

- (a) Provision of a new reinforced concrete ground floor slab.
- (b) Provision of vertical prestress to the sea wall.

There are other alternatives and variations mentioned later.

10. The new ground floor slab would be designed:

- (a) To extend beyond the building to the West so as to engage the return seawall there.
- (b) As a suspended slab capable of carrying appropriate gravity loads if the underlying fills slump away.
- (c) To grip and enclose building shear walls at ground level. To define a level below which new wall vertical steel can be anchored.
- (d) To provide a reliable substrate for any new quality finishes.

Clearly the new slab will substantially mitigate the foundation torsion effect.

However, as a diaphragm the slab does have to span about 70 metres East-West between sea wall returns. The North-South 'depth' as a beam is defined by the building width and (allowing some extension beyond facades) will be about 17 metres. The aspect ratio is suspect and stress calculations suggest that the slab cannot be economically designed to carry more than about half the building base shears. The balance of the base shears will have to be thrown into the top of the North seawall as a transverse top load and they will add to the overturning effects on that wall for South-to-North earthquake.

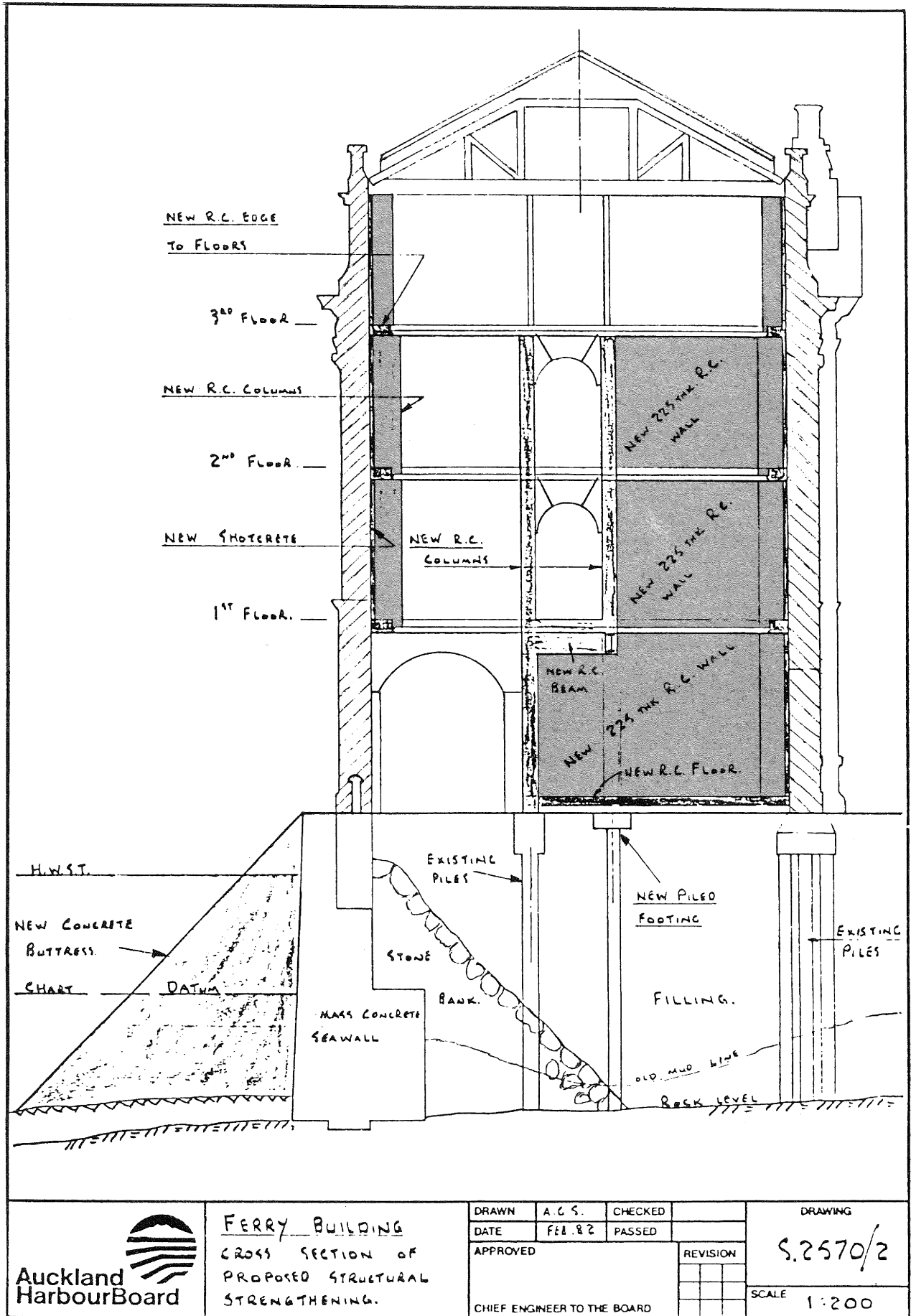


FIGURE 9: PROPOSED STRENGTHENING.

11. Two curtains of prestress are proposed.

- (a) A partial height seaward face curtain of the order of 50 tonnes/metre.
- (b) A full-height rock anchored soil face curtain of the order of 100-150 tonnes/metre.

Intermediate critical sections for the seawall occur at each of two levels because of the stepped profile. At these levels the wall stresses depend on dynamic differences between superstructure base shears and geoseismic pressures for each earthquake direction. Each of these effects is known only with 'order of magnitude' precision and predictions of nett effect cannot be reliable. The use of prestress curtains in both faces will at least provide for a range of reversible nett loads. The seaward curtain strength at 50 tonnes/metres corresponds to an 'uncracked section' flexural strength at the upper step of 2 MPa and to a transverse load at the top of the wall somewhat in excess of the superstructure base shear.

12. The heel curtain is able to deal with the liquefaction pressures plus about half of the building base shears. This proposal needs refinement both in terms of the techniques to be used in anchoring of such large forces to rock and also in terms of the risk of damage to the building during stressing operations.

13. If it later proves necessary to reduce the heel prestress then two avenues are open to reduce the loads on the wall:

- (a) Remove fills immediately behind the walls down to low tide level or to some higher level.
- (b) Provide external buttressing to the seawall from the North (Ferry Basin) side.

These steps are, in any case, worth looking at in any more refined investigation of overall economics. The buttress proposal may very well be orchestrated so as to drastically reduce diaphragm stresses in the new ground floor slab. Clearly, however, external buttressing needs to be considered in the content of future use and any extension or rebuilding of the Ferry Building, Ferry Basin, Tees (the Ferry Wharf) and Queens Wharf.

VIII CONCLUSION

An investigation like this necessarily breaks down into simultaneous and parallel investigations into a number of highly specialised areas. Most of these, in turn, open up a myriad of questions which are beyond the state-of-the-art and often even beyond the best available research findings.

There is no mandatory code of practice but only the judgement of the designers and their specialist consultants, together with the discretionary authority held by authorities such as Auckland City Council

and the Ministry of Works and Development as consultants to the Local Authority Loans Board.

After all of these investigations the community must make a decision through its elected representatives and there is a major problem in distilling all of these advanced and diverse investigations into conclusions, which are both simple and as accurate as current know-how permits.

For our part we are aware that we have had to make or accept the best-available subjective assessments on many difficult questions. In the end one can only sit back and contemplate the overall global effect of these, both technical and financial.

We think that it is possible to provide the building with a base shear strength of the order of 0.20g and to strengthen the foundations to some related but somewhat lower figure. We think that it is possible to restore the facade and weathering envelope along lines already established for similar buildings in Sydney. The budgets also provide for upgrading to modern standards of the interiors, building services and means of access. The skills required need to be drawn from local sources, from Sydney and from Europe.

It would not be cheap. Nobody would expect to find cheap transport by restoring an antique Rolls Royce.

One wonders too, whether the community could reasonably expect the Harbour Board to bear the entire cost.

IX ACKNOWLEDGEMENTS:

We thank the Auckland Harbour Board for giving us permission to print this paper and wish to make quite clear that we have no intention of prejudicing any decisions they may have to take in the future.