2

Ferry Buildings

Gurley & Nicholls

Correspondence from
Nov 1979

C & N. Concrete Construction. April. Issue. 25/2/82

FERRY
BUILDINGS
GURLEY etc.

550 Box File

Auckland HarbourBoard

To: Engineer File

Please place in "Box File" for Ferry Building

RW-G
2.3.82

NICHOLLS

TURAL ENGINEERS

47-215, Ponsonby Ph 767-152 767-432 760-772

RECEIVED
1 MAR 1982

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This has been cleared for publication by our client the Auckland Harbour Board on the understanding that the Ferry Building is not referred to as an 'investigating' committee is still 'to sit' and deliberate on our reports as issued.

Please call us if you need further information.

Yours faithfully
GURLEY & NICHOLLS

J S NICHOLLS

bcc

Mr Wells-Green Please keep with other Ferry Bldg reports.

RW-G
2.3.82.

550 Box File

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40 Co… …w Zealand PO Box 47-215, Ponsonby Ph 767-152 767-432 760-772

RECEIVED
1 MAR 1982

25th February 1982

The Director
New Zealand Concrete Research Assn
P O Box 50156
PORIRUA

Dear Sir

re APRIL ISSUE - CONCRETE CONSTRUCTION

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GURLEY & NICHOLLS

CIVIL AND STRUCTURAL ENGINEERS

40 College Hill, Auckland, New Zealand PO Box 47-215, Ponsonby Ph 767-152 767-432 760-772

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2.3.82.

Colin R. Gurley M.Eng SC. BE(Hons) MNZIE, FIEAust, MASCE, Registered Engineer
J. Spencer F. Nicholls BE, MICE, MNZIE, Registered Engineer

GURLEY & NICHOLLS

Consulting Engineers

OLD MASONRY & EARTHQUAKES

- BRICK TESTING AND A STRENGTHENING METHOD

EARTHQUAKE ENGINEERING BACKGROUND:

In New Zealand it is a familiar idea that the construction and continued use of any building in an earthquake prone country necessarily involves an acceptance of a degree of risk. Current seismological thinking suggests that the 'maximum credible' event would induce horizontal accelerations of the order of three times gravity in many old masonry buildings, which are typically low rise and hence structurally stiff. Few if any, even of the best modern buildings, would survive such an attack. The risk of this maximum event is certainly very small - probably much less than one chance in a million in any one year and as one comes down the scale and considers much smaller earthquakes the risk of occurrences becomes much higher. Current practice for the design of new buildings seems to imply consideration of events with a return period rather more than 100 years.

Furthermore the costs of reducing the risk to any specific building can be markedly sensitive to the degree of reduction. Clearly this involves major issues of public policy, particularly when the public interest in earthquake hazard reduction appears to be in conflict with the public interest in the preservation of historic buildings.

If a building is considered under the District Planning Scheme as having both historical architectural and community significance it can also be expected that the Historic Places Trust would consider it under a high category e.g. "Those buildings which merit permanent preservation 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 earthquake byelaws to be improved to meet specified requirements.

The structural 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 1960'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 C.C. requirements can be as low as 1/16 of the requirements of NZS 4203.76.

As in L.A. the underlying philosophy of the local requirements is to reduce danger to life without attempting necessarily to protect the buildings concerned from damage. However, when considerable sums of money is considered for the use of restoring and preserving an old building a logical attitude would include the concern that a high degree of protection must be given to any 'historic features' on the building i.e. it is not just the removal of the risk of collapse but that any work done should prevent physical damage and "loss of that visible characteristic which is the reason for its preservation."

Discussion/dialogue between the public and private engineering design sectors on this subject is still in its early stages but indications are that if an old building is going to need an injection of public funding the MOWD as the checking consultants of the Local Authority Loan Board would view the work in this way and hence must apply more restrictive standards than those of the old NZS 1900's.

STATE OF THE ART

In this situation a reasonable man might ask whether the relationship between the risk of failure and the cost of strengthening some specific building has some critical point such as illustrated by Fig 1.1 (a). If such a point exists and if both the corresponding costs and the corresponding risks seem to be of a reasonable order of magnitude then the point concerned would seem to provide a sensible comprise. This seems to us a better approach than that of starting with some preconceived ideas of the precise level of "acceptable risk".

The state-of-the-art in relation to new buildings has expanded dramatically in the last decade as a direct result of a series of major and quite damaging earthquakes striking at the technically advanced areas of North America, Japan and eastern Europe. The rate of change has been such as to render obsolete much pre-existing knowledge. Nevertheless construction of new buildings has had to go on through the period of change. The cost of risk reduction at the time of initial construction is obviously much less than at some later time. For this reason it is natural that the Codes of Practice for the design of new buildings would move to a fairly conservative stance until the state-of-the-art has settled down again. For this reason they are not necessarily directly relevant to the strengthening of old buildings.

The new knowledge does, at a basic and fundamental level, provide much better understanding of the performance of buildings in earthquakes. At the level of technical data and detail, however, it has been principally concerned with the modern structural materials (structural steel and reinforced concrete) and with the structural form that has been predominant in the 1970's. Most multi-storey buildings constructed in the 1970's are "ductile frame" buildings and are characterised by low lateral strength, low damping, only moderate stiffness and high ductility.

Application of the new basic knowledge to the materials and structural forms existing in old buildings has barely begun. There has been some recent efforts here and in California to develop ad-hoc expedient solutions to pressing problems but these have mostly been concerned with buildings of a modest scale and of little historic significance and the discussion as has taken place tends to assume that the new concrete (or structural steel) would be designed as a primary 'stand-alone' structure while the existing fabric is regarded as 'ride along' architectural decoration.

STRUCTURAL DUCTILITY

The property of ductility is the characteristic which determines the performance of a building after the onset of general structural damage. A ductility factor of 1 corresponds to a brittle building. If such a building is attacked by an earthquake exceeding its strength then the damage caused by each successive major seismic jolt will drastically weaken the ability to resist later jolts. The damage and weakening effect will accumulate and collapse may follow after just 2 or 3 major jolts. Ductile buildings have some ability to accept and limit or, at least, slow the accumulation of damage from successive jolts. So a ductile building will survive substantially longer than a brittle building in any given damaging earthquake and there is a much better chance that it will still be standing albeit damaged, afterwards. Thus, for any given level of earthquake risk there is a trade-off available between strength and ductility. Ductile buildings can be relatively weaker for any given degree of safety. Modern ductile-frame buildings are intended to have ductility factors in the range of 4 to 6. We do not think that is possible to transform every old building into a highly ductile structure but it is desirable to try and aim at some moderate degree of ductility, say in the range of $1\frac{1}{2}$ to 2.

SEISMOLOGY

Figure 1.4 (a) shows curves which relate earthquake return period to base shear coefficient. The return period is so defined that for example, the 100 year earthquake has:

* 1 chance in 100 of being exceeded in the next year
* 1 chance in 10 of being exceeded in the next 10 years
* 2 chances in 3 of being exceeded in the next 100 years

These curves are based on recent studeis by a technical committee of the N.Z. National Society for Earthquake Engineering on bridges (NZNSEE Bulletin Sept '80). Nevertheless they do seem to provide the best current information and they do imply that the seismic risk exposure of Auckland as compared say, to that of Wellington and Christchurch is significantly lower than had been thought. We have noted that a later paper by Priestley et al (NZNSEE March 1981) has again modified the results.

USE OF EXISTING MASONRY

Several recent earthquake strengthening projects have used the expedient solution of adding a 'stand alone' structure in modern steel or concrete however the introduction of large quantities of new concrete into an old building would have two disadvantages:

* To the extent that it increased overall weight particularly at higher levels it would increase the forces developed by an earthquake
* The costs of such an approach could well become too high

It seems then better to look first at the possibilities of using the existing masonry as a major component of any new earthquake resisting system. This requires a sampling and coring program in two stages:-

* First seek to establish that the existing brickwork is of a certain basic minimum standard or can be brought to that standard by feasible methods of repair, and then, if that is successful:
* To gather essential data on the physical characteristics of the materials actually used.

The design of the testing program needs to be done with some care but INITIAL TESTING to enable design to commence would be briefer and less complex. The recently available "Tentative Los Angeles Ordinance and Testing Programme" (Schmid Kariotis & Schwartz) is an excellent example of a programme assembled to give results suitable for use on a design process however we found the programme very particular and only partly relevant to our situations. The tests listed are a combination of laboratory and insitu tests and generally the isitu tests have advantages of the lab.

The laboratory tests suffer badly from:

a) sample breakage and sample loss
b) sample cracking and material disturbance during removal leading to unuseable results.

(see appendix 1 (a) - Brickwork Testing Programme)

SOME RESULTS - ACTUAL BRICKWORK TESTING

Coring and testing of a substantial (turn of the century) Auckland building gave initial compression results in the range of 19 to 24 MPa i.e. roughly equivalent to weak concrete. However, during the testing an impression formed that the brickwork was 'much less stiff' than concrete of the same strength. There does not seem to be any easily available data on the stiffness properties of old masonry either in the general international literature or in the 'earthquake hazard' literature and test-report series carried on recently in New Zealand and California. Further testing therefore sought to measure stiffness and to check behaviour at high strains. To provide a basis for comparison we summarise the Youngs Modulus (E) of the various modern building materials i.e.

Steel, E :	200 GPa
Concrete, E :	20 to 30 GPa
Modern Brickwork: E	10 to 20 GPa
Pinus Radiata, along grain, E:	5 to 10 GPa

The test results are quite remarkable for example:

* The tangent stiffness is of the order of 2 (not 20) GPa at low stresses (1 to 2 MPa) reducing to 1 GPa at higher stresses.
* The creep behaviour is less significant than for concrete - the (nett) creep strain is less than half of the elastic strain and creep is virtually complete within a month
* The strain capacity is much larger than concrete. Unconfined concrete typically spalls at strains in the range 0.3 to 0.4%. Strain capacity REDUCES as strength increases and, in this sense, high strength concrete is more brittle than moderate strength concrete.
  For this building brickwork strains were measured in excess of 1%. Measurement of such high strains with conventional equipment is difficult and measurement at higher strains was not possible. However, there is, at least, a suspicion that ultimate strains exceeded 1½%.

In seeking explanations for this behaviour one notes, first of all, that the lime mortars used in historic buildings are much less stiff than modern mortars. Indeed current practice in, say, Britain and Australia for new brickwork seems to have changed over the last 10 to 20 years. In the 1950's to 1960's most brickwork seems to have been constructed with 'all-cement' mortars which provide maximum strength, stiffness and weathering resistance. However, such brickwork seems often to have been unduly brittle and to fracture too readily in response, for example, to minor foundation movements. More recent practice seems to use 50/50 cement/lime mortars as providing reasonable strength combined with much less brittleness. Clearly, an 'all-lime' mortar would again reduce strength but also improve brittleness and improve strain capacity.

However, the mortar character is likely not the sole explanation. There is, at least, a suspicion that this building bricks were under-fired as compared to modern bricks and that this effect is significant.

We noted also that the second test series produced strength results approximating 12Mpa. This is a good deal lower than those of the first test series (19 - 24 MPa) but nevertheless still higher than the presumptive standard (8MPa) for modern Grade A masonry. This may reflect actual variability or differences in sampling technique but we are inclined to suspect that it reflect the rate of load application. Because of the strain measurements, the tests of the second series were carried out at a much slower rate than those of the first series.

DESIGN RELEVANCE

The comparative strength/stiffness properties of this brickwork and modern (30MPa) concrete are sketched in Figure 6.2
Conventional design for 'normal' concrete limits the steel content so as to preclude strains in excess of 0.3%. Even in non-seismic structures, critical elements such as gravity-loaded columns are provided with some minimal degree (0.1% to 0.2% of area in each direction) of confinement steel but the strain limit is nevertheless maintained at 0.3%.

In new concrete structures specifically designed for earthquake the confinement steel in critical areas is drastically increased with the objective of increasing the strain capacity to, say, some value in excess of 1%. There may also be some increase in strength but this is not the primary objective.

Quite apart from the effect of cost, very high contents of confining steel can lead to severe construction problems in placing dense, compact concrete. The present aim is to keep the confinement steel content down to the minimal content (0.1% to 0.2% in each major direction) generally used in the design of new buildings for seismically non-critical components. While this falls well short of the content used in new seismically critical areas, it will certainly constitute a marked improvement on the present situation of zero confinement.

Acceptability of this approach will depend on the overall strength thus imparted to any particular building.
In this case it is necessary to limit maximum strain under design earthquake load to 0.3% and to evaluate brick stresses accordingly. Because of the variable nature of the material and the other highly subjective estimates entering into any analysis, great refinement does not seem warranted. The brickwork might thereforebe treated as

* EITHER An elastic material with modulus E = 2 GPa in the strain range 0 to 0.3%. Peak stress 6 MPa
* OR A plastic material with strength 4 MPa right across the strain range 0.05 to 0.3%.

All of the testing has been of fairly dry (but not dried) samples. It may well be that the strength of old brickwork will reduce substantially when it is wet. Of course continuing dampness in the fabric of an old building causes many problems and also becomes critical when one proposes to introduce reinforcing steel. For this reason no strengthening proposals should be seriously entertained unless they are to be combined with proposals for upgrading weather resistance to acceptable levels.

A PROPOSED STRENGTHENING METHOD (FONDEDILE TECHNIQUES)

The Fondedile S.p.a of Naples has been involved in the repair of ancient buildings and monuments in Europe for many years. It has a British subsidiary, "Fondedile Foundations Ltd" which is associated with the British contractors, Sir Robert McAlpine Ltd., of London. 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 hundred (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. In the first instance the detailed geometry of this array will have to meet the criteria of practical construction. However, it is evident that this is not the sole basis for design. The array must also be effective in conferring adequate strength and robustness on the structure considered as a total reinforced entity.

The technical director of Fondedile is Dott. Ing. F. Lizzi. He has written on this matter (to the best of our knowledge) only at a very general non-quantitative level although even that is sufficient to 'stir the blood' of any experienced practising engineer.

It is evident that what he is doing is, at this stage anyway, more of an art than a science but he seems to be doing it successfully.

There are subtle issues involved. For example, the complex non-perpendicular Lizzi reinforcement arrays are a good deal more difficult to appreciate in quantitive design terms than the simple perpendicular arrays used in new buildings. This does not necessarily mean that they are less effective. Indeed, we consider that they may often be more effective. On suspects that we do not yet understand the subtle interplay of flexure, shear, bond and anchorage and, in particular, that we continue to underestimate the role of bond, anchorage and confinement which does depend markedly on detailing.

PRESENT PURPOSE

The present purpose is to evolve some coherent, systematic strategy for the detailed design of strengthening work for any old masonry building as some combination of:-

* The Fondedile style and technique

and

* New Zealand Codes for the design of new buildings,

recognising that the standards implied by the latter may not always be attainable at reasonable cost.

In particular, the aim will be to utilise the existing fabric as fully as possible supplementing it with new reinforcement and some new concrete. Where new concrete is added the aim will be to make it work compositely with the fabric. This aim is quite radical in the context of current thinking in New Zealand and in California. However, it seems particularly significant for structures likely to be subject to intense ground shaking (say MMVII or greater). An earthquake does not impose some definite force. Instead, as a first approximation, it injects some fairly definite amount of vibrational energy. That energy will seek out <u>any</u> weak chink in a building, exploit it and dramatise it.

Hence although this reinforced assemblage may not meet exactly the criteria laid down in the modern codes the steel bars convert the fabric into a material with predictable structural properties and this coupled with appropriate earthquake design loadings shows a potential for markedly improving a masonry building's original condition.

GURLEY & NICHOLLS

25th February 1982

Appendix 1 (a)

BRICKWORK TESTING PROGRAMME

A. **BRICKS** : Remove sample bricks from existing structure. Take samples from inner and outer wythes and any bricks of varying colour.

1. Laboratory test to assess compressive and shear strengths.
2. Laboratory test to assess range of strengths in the structure.
3. Analyse results to assess mean brick strengths, floor by floor.
4. Laboratory test for any particular properties e.g. excessive moisture penetration, excessive moisture movement, any thermal movement.

Investigate the use of 'SCHMIDT' hammer to aid in above.

B. **MORTARS**: Remove samples of both cement and lime mortars using diamond coring equipment. Cut, trim and 'cap' as necessary and laboratory test to give:

1. Compressive strengths.
2. Mean compressive strengths

Investigate the use of chemical analysis per an Industrial Chemist.
Investigate the expected strengths deducible from any original contract specification clauses.

C. **BRICK WORK** :

1. Assess rupture strength of mortar 'in shear'
   - a) using diamond drum coring tools & lab. tests
     - in outer wythes
     - in inner wythes
   - b) using insitu tests by selective mortar removal Refer L.A. programme
     - in outer wythes
     - in inner wythes (where feasible)
2. Assess rupture strength of the brick & mortar composite in compression
   - a) cut & remove & lab. test sample say 600 x 600 x 325
     - in outer wythes
     - in inner wythes
   - b) using an 'INSITU' face test Refer L.A. programme
     - in outer wythes

Test C. 2 b measure strain and stress during the test. This test can only assess a maximum likely face compression resistable by the outer wythes of the sampled wall.
i.e. A useful check on the walls capacity to resist 'face' loads.

Test C. 1 a Check local test machine sample size limiatations.

Tests C 1 & 2 Ensure a sampling from both the main building cross walls and the tower walls.
Ensure the sampling identifies both the lime and cement mortars.

Space precludes a full list of likely tests however, if other materials exist in the building fabric e.g. sandstone, sampling and testing will be necessary and also a range of investigations would follow into the buildings alignment and any metal fixings or anchorage, envisaged for the remedial schemes.

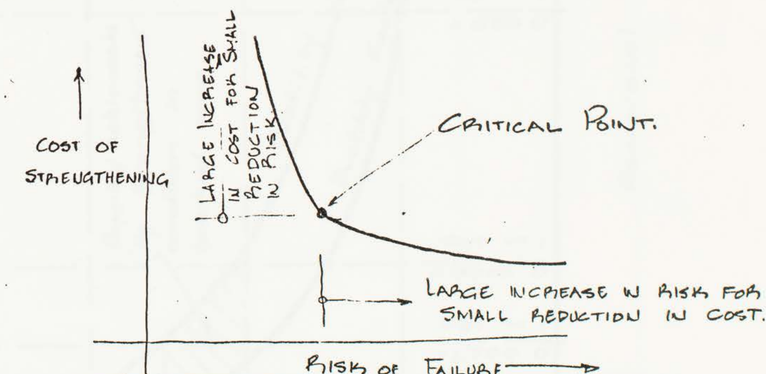
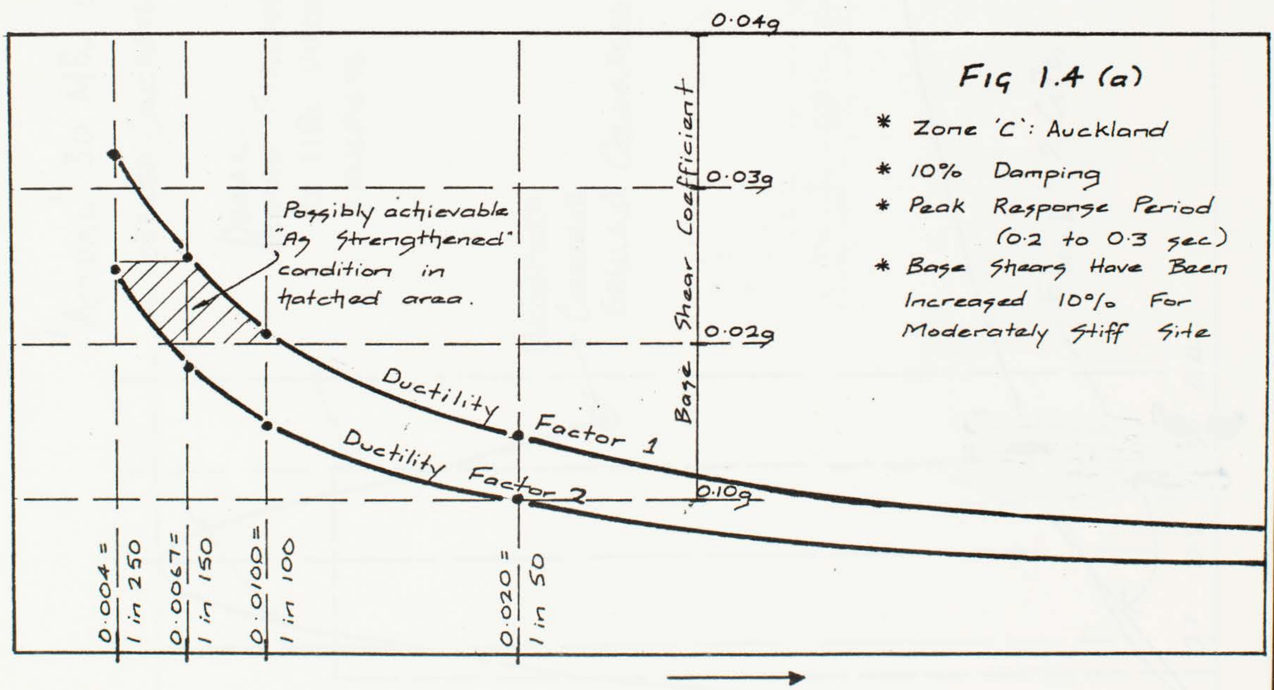


Fig 1.1(a)



Reciprocal Return Period = Risk of Occurrence Within the Next Years

CG APRIL 1980

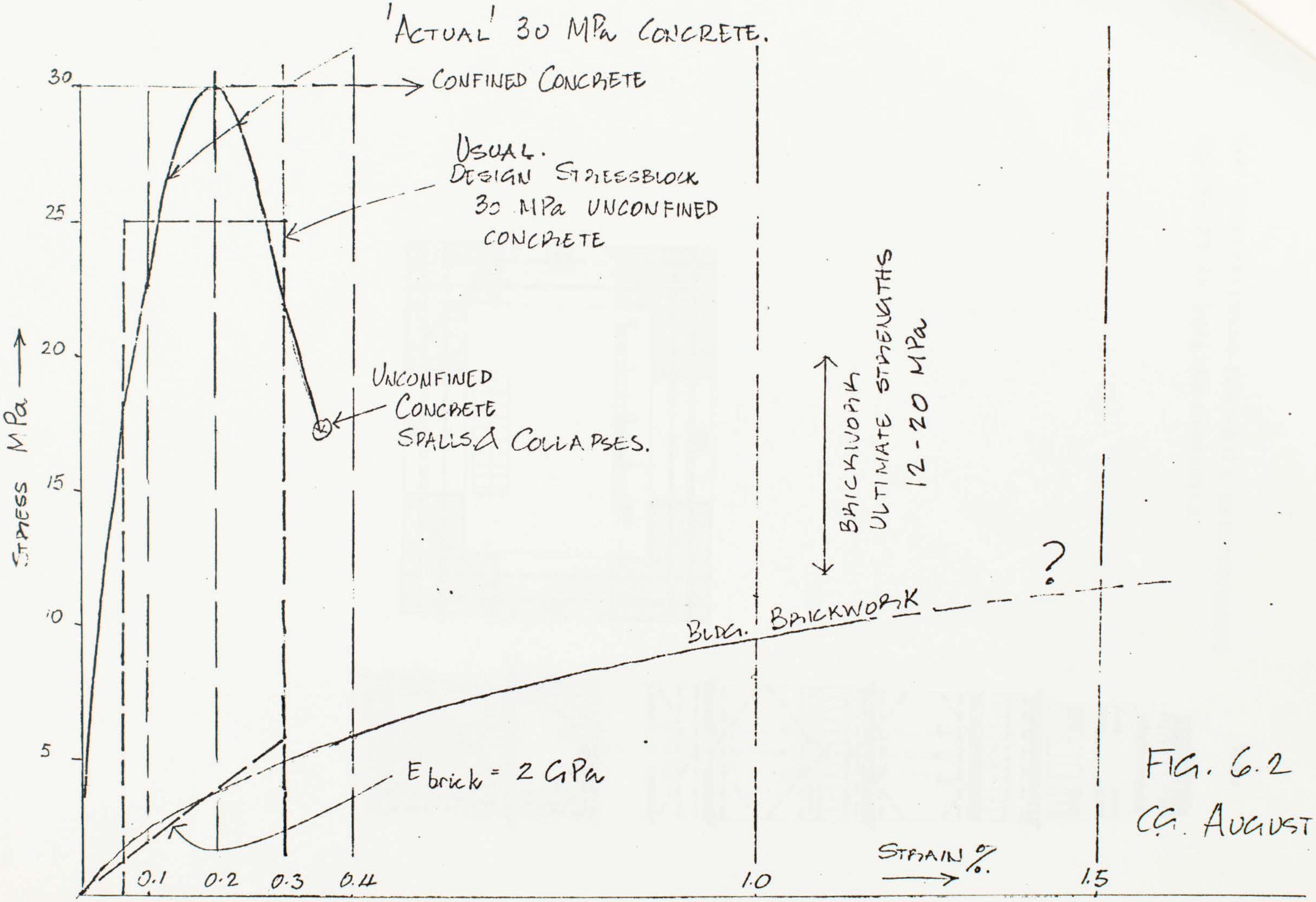
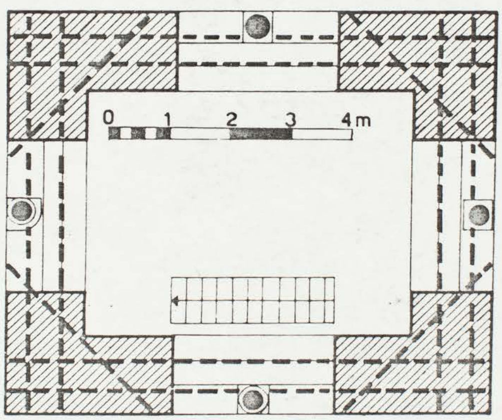
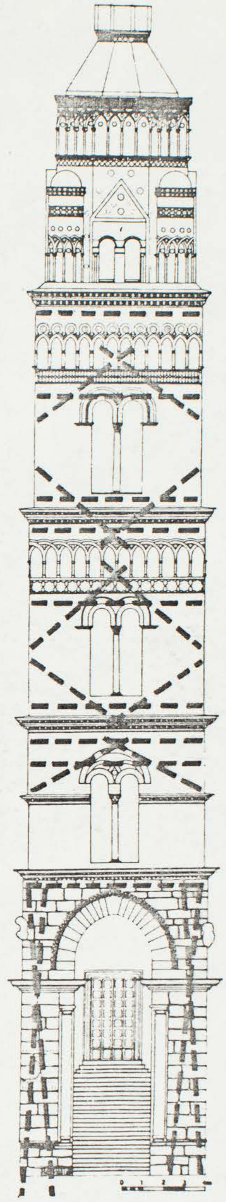


FIG. 6.2
CG. AUGUST 81

NON PERPENDICULAR FONDEDILE REINFORCEMENT ARRAYS IN AN ITALIAN CAMPANILE





87/2

Auckland Harbour Board

RECEIVED 12 FEB 1982 ACKD. AUCKLAND HARBOUR BOARD

Memorandum

To: THE GENERAL MANAGER Date: 11 February 1982

From: THE CHIEF ENGINEER

RECEIVED 17 FEB 1982

FERRY BUILDINGS - REQUEST BY CONSULTANTS FOR PERMISSION TO PRESENT A PAPER

Gurley & Nicholls have, by letter dated 10 February 1982, requested permission to present a technical paper related to their work on the structural upgrading of the Ferry Buildings to the N.Z. Concrete Research Association Magazine "N.Z. Concrete Construction" - April issue.

I would expect that by the time that the article appeared in public that the Sub-Committee on the Ferry Buildings would have met and reported to Board at the March meeting so that any technical information published in the magazine would not come as news to Members, consequently I wish to approve the request. Before approving I seek your agreement.

[signature]

CHIEF ENGINEER TO THE BOARD

BRLeC:JMH

Enc. Copy of letter from Gurley & Nicholls dated 10 February 1982

The C/E,

Agreed

[signature]
General Manager. 17/2/82

Staff Engr to note & inform Nicholls

Nicholls advised of approval 22.2.82
Draft paper returned to him PW-G.

BLeC

GURLEY & NICHOLLS
CIVIL AND STRUCTURAL ENGINEERS
40 College Hill, Auckland, New Zealand PO Box 47-215, Ponsonby Ph 767-152 767-432 760-772

22 December, 1981

The Chief Engineer
Auckland Harbour Board
PO Box 1259
AUCKLAND

ATTENTION: Mr. B. Le Clerc

Dear Sir,

re: FERRY BUILDING

We here confirm an issue by hand last week of one copy of the Sandstone Report by Dr. G.S. Gibbons. Copies of this document, with the drawing attachments have been made available to Architect, Mr. T. Dixon and Quantity Surveyor, Mr. A. Dickenson.

We here enclose for your information :

One copy of the A.R.O. (University of Auckland) report on Seismic Wall Pressures (November, 1981) by Dr. T.M. Larkin.

Overall costing information by Hallam-Eames & Partners, incorporating both the above report, and

Notes from ourselves on suggested construction recommendations.

We consider the report by Dr. Gibbons to be of excellent value and he has identified and tried to quantify many problem areas. As he mentions, it is not unusual for a restored stone building to continue to show deterioration and even deteriorate at a greater rate and hence our reliance on Dr. Gibbons to correctly analyse the existing deterioration causes is of paramount importance. Dr. Gibbons has listed a formidable range of required works but individually the majority are not so daunting.

Ensuring the building has good drainage and properly functioning windows, flashings and fabric (e.g. structural steel beams in the tower) is work well within normally available skills. Also, the works involved with removing opportunity for efflorescence, pigeon access and water penetration. Even though the extent of stonework repair can still not be accurately assessed, it is encouraging that at this stage, Dr. Gibbons considers the bulk of the work will be redressing, sweetening and only the odd stone requiring replacement.

.... 2/

Colin R. Gurley M.Eng SC. BE(Hons) MNZIE, FIEAust, MASCE, Registered Engineer
J. Spencer F. Nicholls BE, MICE, MNZIE, Registered Engineer

Auckland Harbour Board 22 December, 1981

The Larkin report on the likely liquefaction problems is encouraging and we feel is as far as we need to go at this stage. As the author summarises on page 7, the cost of further analysis would be high and the feedback doubtful. From this work, we have the confidence to design the wall as a dam to resist the pressures from the liquefied soils and this loading was covered by the presentation in our Section 11 (Foundations) and our previous costing estimates.

We have enclosed some suggested construction recommendations, but we consider them preliminary and in outline only. When the direction of the project is more specific, we would like the opportunity to rethink and extend these comments.

Yours faithfully,
GURLEY & NICHOLLS

J S Nicholls

Enc.

1.0 PRELIMINARY WORK:

With New Zealand fixed into an inflationary spiral, the earliest project start is the best, and delay in spending any planned monies can only mean an increase in the funds expended. However, it will be difficult to avoid financial and physical restraints that must effect this project.

We do not wish to comment on the problems of loan monies, nor at this stage, the effects likely of the Ministry of Works and Development being involved in the project, however we see at this stage, several areas of research and/or feasibility studies that will need to be woven into any job programme. For example :

1. Kenitex removal from stonework and stone cleaning - use of water and/or solvents.
2. Replacement stone - accelerated weathering tests and/or durability tests.
3. Brickwork drilling and brickwork grouting.
4. Internal plastering and finishing. Examine types and cost of alternatives.
5. Masonry, external jointing and pointing. All sandstone, basalt and brickwork.
6. Final finishing - brickwork and sandstone - all to reduce the visual and long term effects of the plugging of reinforcing steel and patching of damaged fabric.

Dr. Gibbons gives several options with regard to Items 1 and 2 and both these activities are major problem areas that will, no doubt, never give a total answer. If an approach can be found that will reduce the immediate problems that may be the best solution achievable, consequently, the unsolvable parts of these problems will then need to be built into the total programme.

Item 3 could commence any time when a suitable building or brick assemblage is discovered to be available. Longyear Drilling Co., have indicated a willingness to set up some trial drilling and on 5 January, 1982, Mr. Malcolm Brain, ex Fondedile (UK) Works Manager (now resident in Fiji), will be briefly in Auckland. A trial drilling operation, as above, with Harbour Board personnel and Mr. Brain and ourselves present would give valuable feedback to our schemes and plans. Whether this proceeds in 1982 or 1983, it looks to be essential research which may need to be arranged rapidly when a suitable venue comes available. We can arrange to meet Mr. Brain in the new year and mention to him the possibility of such an operation occurring.

Items 5 and 6 presumably will not be finalised until Items 1 and 2 are complete, but at that stage, further recommendations will need to be sought from Dr. Gibbons and it could be opportune to set up sample panels on the building and see the weathering and inter-seasonal effects at least over the reconstruction period. While the building is scaffolded during this period, access will be readily available to all parts of the building fabric. Never again will there be such an opportunity. Maybe also, at this stage, any varying or alternative recommendations planned for the maintenance schedule could be compared and examined.

Many sections of work should ideally only go ahead when everyone involved has been able to develop a complete familiarity with the fabric as existing (and as desired at completion) and this feedback can only come from this physical research programme and a continuing literature search.

2.0 THE CONTRACT:

Procedures normally used for commercial building projects are not the type that would encourage the 'special' project atmosphere that we feel is warranted here. We consider any construction work on the Ferry Building must be considered as restoration of an 'antique' and using the word antique with all its emotional connotations. Working on antique objects of any type requires enthusiasm, dedication and patience and a determination to make the finished product as perfect as possible. Whether the inevitable commercial pressures will allow the Harbour Board to see the project in this light, we do not know.

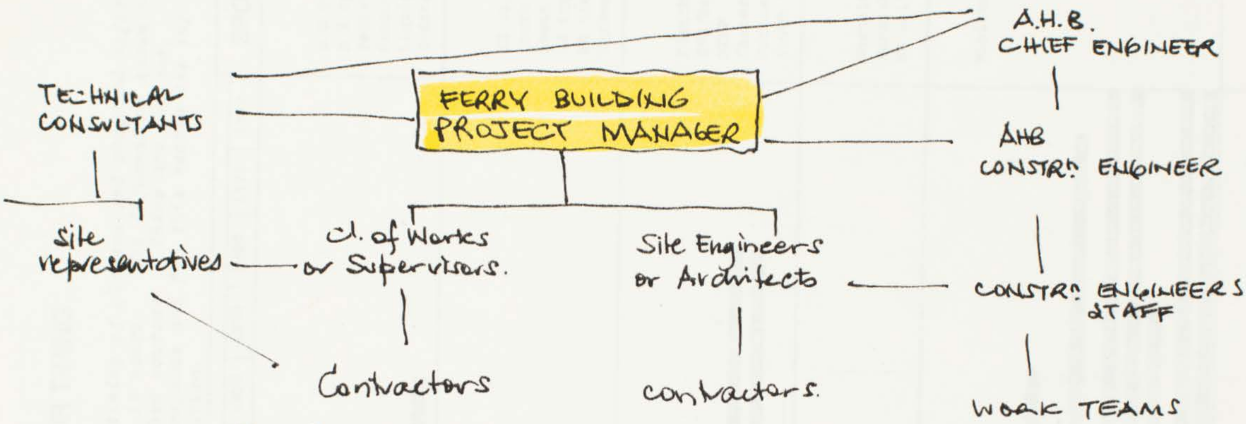
We certainly anticipate difficulties in assembling a meaningful set of documents that would allow normal tendering and letting of lump sum or even charge-up contracts. Considering especially the range and implications of the research and feasibility studies discussed in Section 1.0, we feel provision must be made to keep individual work packages modest in size so that advantage can be taken of feedback from any ongoing studies.

We suggest a project manager should be appointed to have full control over the project. He would be on the Board's staff, but independant of other Board projects and dedicated only to the Ferry Building. He would report to the Chief Engineer and his staff and would have direct links to the Construction Engineer and his staff. We see it as useful and appropriate that many of the smaller jobs could be carried out by the Construction Division and that whole department would have an involvement in the reconstruction.

The Project Manager would cross relate and maintain discourse with the technical Consultants and he would have a staff of supervisors - Architects, Engineers and Clerk of Works who would have direct control over the onsite work teams or contractors.

Using this method :

* Contractural pressures are minimised,
* No work need proceed until all the correct information is available and the timing is right,
* All supervision is by the one team and this group is under the direct control of the Board.



We also attach here the latest estimate by HERA of the major projects planned in New Zealand over the next ten years. There has been suggestions made that these works would effect the availability of a work force for the Ferry Building. Discussing this point with a local contractor and a group that does much work in the Queen Street valley, they do not anticipate these projects affecting their own established teams. We feel however, it must affect the choice of supervisory personnel and the speed with which some specialist groups will perform on the job.

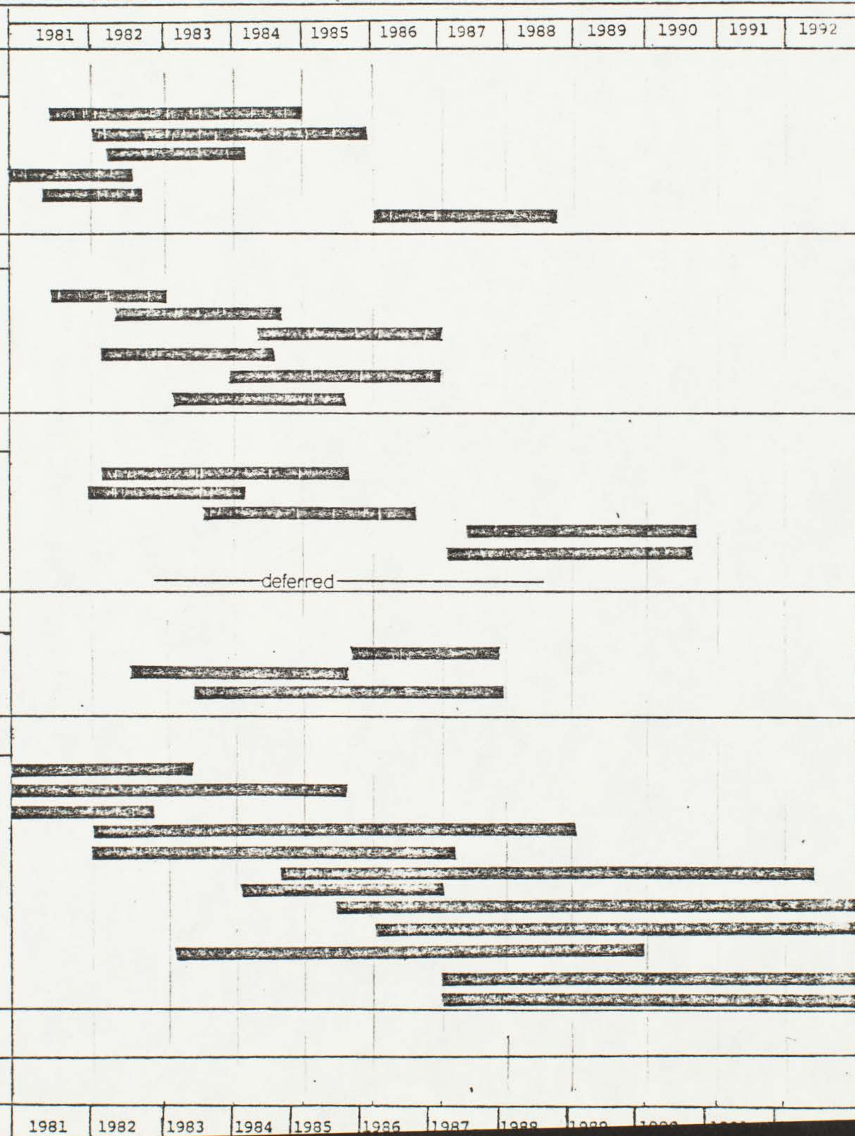
GURLEY & NICHOLLS

MAJOR PROJECTS (over $50m) AND THEIR TIMING

HERA NO. 5 OCT. 81.

The table below sets out the broad timing of planned and proposed major construction projects over the next 10 years. The information shown has generally been obtained from published sources. There are obvious uncertainties associated with all projects and for this reason the information shown should be interpreted with caution.

Planned Start/Finish



PROJECT	SPONSOR
PETROCHEMICAL	
Marsden Refinery Expansion	N Z Refining Company
Synthetic Petrol	N Z Synthetic Fuels Corp
Chemical Methanol	Petralgas Chem. N Z
Ammonia Urea	Petrochem Corp. of N Z
LPG Distribution	Liquigas
Ethane Extraction	Undetermined
BASE METAL	
N Z A S 3rd Potline	NZ Aluminium Smelters
2nd Aluminium Smelter I	South Pacific Aluminium
II	South Pacific Aluminium
N Z Steel Expansion I	NZ Steel Development
II	NZ Steel Development
Silicon Carbide	Ceramco
PULP & PAPER	
Kawerau 4	Fletcher/Challenge
Nelson Pulp Mill	CSR/Baigent
Marsden Pulp/Paper	NZFP
Whirinaki Newsprint	Carter/Oji Kokusaku
Northern Pulp	Northern Pulp
Otago Pulp	South Wood
IMPROVEMENTS TO TRANSPORT	
Northland Forestry Port	Northland Harbour Board
Extension to Port Taranaki	Taranaki Harbour Board
North Island Rail Electrification	NZ Railways
POWER PROJECTS	
Ohau B	Ministry of Energy
C	"
Rangipo	"
Clyde	"
Ohaki	"
Ngawha Springs	"
Marsden B Conversion	"
Queensbury	"
NI Thermal	"
Luggate	"
Kawerau Gorge	"
Lower Waitaki 1 & 2	"
CEMENT WORKS	
Oamaru	NZ Cement Holdings

COLIN GURLEY

PRESERVATION AND RESTORATION TECHNOLOGY: STONE

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INTRODUCTION

When rock in the in-situ state is exposed to natural weathering processes it undergoes chemical and physical changes that can lead to changes in its appearance and a reduction in its durability. The original minerals in the rock can be altered chemically to form less durable secondary minerals. (Chemical weathering is described geologically as decomposition.) Mechanical weathering processes can bring about a physical breakdown of the fabric of the rock with little or no significant change in the composition of the constituent minerals. (This process is described geologically as disintegration.) Water and air are the principal agents in these weathering processes but temperature changes, wind, carbonation, and plant and animal action can also contribute to the breakdown of rock. When the rock has a strong resistance to weathering processes, deterioration may take place extremely slowly and little change in durability occurs over very long periods of time, but when rock is quarried for building stone and is placed in an aggressive environment or in locations in a structure where accelerated weathering can take place, deterioration may occur much more rapidly than in the original environment.

Although no masonry buildings in Australia are yet two hundred years old, there are now many instances of serious deterioration in some of the older structures and there is an increasing realisation that if the national heritage is to be preserved there is an urgent need to restore and protect many of the buildings that have been classified as being of historical importance.

CLASSIFICATION OF STONE

The building stone used in Australia is obtained mainly from sedimentary and igneous rocks. The main sedimentary rocks used for this purpose are sandstones and limestones. Sandstones consist essentially of fragments of quartz and subsidiary amounts of other minerals. The weathering properties of sandstone are influenced significantly by the nature of the matrix and the cementiteous material that binds the quartz grains together and which in descending order of durability can be siliceous, ferruginous, calcareous or argillaceous according to whether silica, iron oxide, calcium carbonate or clay is the predominant constituent. Some of the more common igneous rocks used in building construction include granite and basalt. With the exception of slates and marbles, metamorphic rocks are not commonly used for building purposes. As stone buildings in Sydney are built mainly of sandstone, the information given in this paper applies mainly to this stone.

CAUSES OF DETERIORATION

Disruptive forces resulting from the presence of harmful salts in porous building stone are a major cause of decay. These salts can come from the atmosphere, the foundations, the mortar joints, unsuitable cleaning agents or they may have been present in the rock before it was quarried.

Most of this kind of damage is caused by soluble sulphates and chlorides. In the larger cities increasing atmospheric pollution caused mainly by fuel combustion has led to the formation of large quantities of sulphur dioxide which react with carbonates present in limestone, calcareous sandstone and mortars to form calcium sulphate.

The sulphur dioxide first changes to sulphuric acid which then reacts with the calcite in calcareous sandstone to produce gypsum. This last reaction is accompanied by a 115 per cent increase in volume which can disrupt the stone.

$$SO_2 + H_2O \rightarrow H_2SO_3$$

$$2\,H_2SO_3 + O_2 \rightarrow 2\,H_2SO_4$$

$$CaCO_3 + H_2SO_4 + 2\,H_2O \rightarrow CaSO_4.2\,H_2O + CO_2 + H_2O.$$

Atmospheric aerosols of marine origin are another important source of chloride and sulphate contamination of building stone in coastal areas. A study of the chemical composition of rain-water in the Sydney area has shown that rain coming from the east or south-east is saline because winds bring large amounts of sea salt aerosols across the coast. The amount of salt deposition from this source diminishes rapidly as the distance inland increases, but as much as 70 tonnes per km^2 of soluble material is deposited per annum on Sydney's coastal suburbs and most of this material consists of sea salts[1].

Another significant source of salt attack is associated with rising damp problems which are often encountered in old masonry buildings where there are no damp-proof courses or the damp-proof courses have broken down. The moisture that is transferred by upward capillary movement from the ground into the masonry usually contain soluble salts (mainly chlorides and sulphates) and over a period of time the increasing salt concentration that accumulates in walls is sufficient to cause decay of the stone.

The mechanism of salt attack in stone is still not fully understood but it is believed to be caused partly by expansive forces resulting from an increase in volume associated with crystallisation and partly with volume changes that result when the salts go through cycles of hydration and denydration induced by changes in temperature and relative humidity. For example, anhydrous sodium sulphate ($Na_2 SO_4$) takes up a large amount of water when it hydrates ($Na_2 SO_4.10 H_2O$) and the hydration is accompanied by appreciable volumetric expansion. The hydration of magnesium and calcium sulphate also results in expansion although to a lesser degree than the sodium salt. Hygroscopic salts like impure sodium chloride or magnesium chloride can take up moisture directly from the atmosphere and changes in ambient temperature and relative humidity can result in moisture changes and the development of expansive forces in stone containing these salts. The magnitude of salt disruption will also be influenced by the pore size distribution in the stone. The resistance of some stone to disruption decreases as the proportion of fine pores increases.

Scaling of stone surfaces can be caused partly by thermal movements and partly by deposition of salts near the surface. Differential stresses can be set up by a thermal gradient within the stone and result in a shear failure between the weak surface layer and the underlying stone. This phenomenon is common in calcareous sandstone where calcium sulphate is deposited as a surface crust that subsequently flakes off.

Salt attack can also result from salts migrating from backing material or from the original contaminated stone to newly replaced blocks nearby. Calcium sulphate can migrate from limestone to adjacent sandstone and cause decay.

Another form of deterioration can occur as the result of an ion exchange between contaminating salts and certain clay minerals present in argillaceous sandstone and other rocks.

Wind is not normally a significant cause of deterioration in masonry buildings. However, the shape or location of a building may result in localised air movements being created which cause increased evaporation of moisture and which result in an accumulation of salts in parts of the building. Increased stone deterioration may follow. Signs of this type of attack are often noticeable near openings and in other locations where there is increased air movement.

Less common causes of stone deterioration are the leaching of carbonates from limestone and calcareous sandstone caused by atmospheric carbon dioxide dissolved in rain-water penetrating into the stone, expansion caused by frost action and damage caused by plants such as ivy and by mosses, lichens and algae. Disruptive expansion caused by the rusting of iron or steel inserts placed in stone can cause deterioration.

Deterioration can take a number of forms such as crumbling and powdering of the stone associated with a breakdown or leaching of the binding material that holds the grains together, by splitting along planes of weakness, by rounding of edges and corners, by cracking and by exfoliation in which thin sheets of stone flake off. The last type of failure can also occur as contour scaling in which the delaminating sheets follow surface contours of the blocks.

THE INFLUENCE OF THE PROPERTIES OF THE STONE ON DETERIORATION

Where deterioration has occurred in stone buildings it will often be noticed that some blocks of stone are badly decayed yet adjacent blocks of what appear to be the same type of stone show no signs of deterioration. Visual indications of uniformity can be misleading even when the stone was taken from the same quarry. The amount of weathering, the porosity, the mechanical strength and the nature of the bedding (in the case of sedimentary stone) can vary greatly depending on the location of the stone in the quarry. There can be differences in the durability of stratified material, soft seams can be intercalated with harder and more durable ones. Natural fissures or cracks may have been present in the rock before it was quarried or they may develop during quarrying and subsequently result in water penetration and decay. (Cracks may also develop as a result of foundation movements that lead subsequently to stone deterioration.)

REPAIRS

It is essential to determine the nature of the deterioration and the underlying causes before an attempt is made to repair the damage. As water is one of the principal causes of deterioration it will be necessary to do everything possible to prevent continued penetration of excessive amounts of moisture into the stone.

Stone deterioration is frequently the result of poor architectural design and detailing or of faulty construction and lack of regular maintenance rather than a choice of unsuitable stone. The likelihood of a damaging build up of salts in the stone initiated from surface deposition will be reduced if the masonry surfaces are flushed down regularly by rain-water. Unprotected horizontal surfaces and projections provided by hoods, cornices, copings, sills, parapets and ornamental features allow water to collect and soak into the stone and encourage the internal concentration of salts in localised parts of a building. (Covering these surfaces with lead flashing will often prevent continued deterioration.)

Stone of sedimentary origin should be laid with the natural bedding planes horizontal. If it is laid with the bedding planes vertical and parallel to the face of the wall, splitting and exfoliation of sheets in this plane can occur.

Stone may be incorrectly bedded because of an error on the part of the mason or because the orientation of the bedding planes could not be readily identified on the site. If this last problem also exists with new stone the bedding planes should be marked before the stone leaves the quarry.

Lack of maintenance is often associated with leaking roofs, gutters and downpipes which cause water to flow over parts of the stonework. Staining and deterioration of the stone results. Open joints present another means for the ready penetration of water into the stone. Rising damp problems may be associated with a failure of damp-proof courses.

Deterioration can be caused by incompatability of stone. If limestone and sandstone are placed close together, calcium sulphate formed by the reaction between calcite in the limestone and sulphur dioxide from the atmosphere can be transferred into the sandstone and cause it to decay.

It will often be necessary to repair the joints in the stonework as part of the restoration work. They should be more permeable than the adjacent stone so that moisture can move from the stone into the joints. If a dense impermeable mortar is used it will act as a moisture barrier and deterioration of the adjacent stone caused by salt deposition may result. A 1 : 2 : 9 mix (by volume) is a suitable mortar for use with a porous sandstone. Too much cement in the mix can also lead to excessive shrinkage and cracking of the joints. Mason's putty made from whiting, lime and linseed oil is a good jointing material and was used extensively in the past. It is now being manufactured commercially for masonry work. Flexible sealants such as butyl mastic and silicone rubber are also being used for jointing but their long-term durability has not been proven and they will act as barriers to moisture movement from the stone into the joints. Some of these materials become brittle after prolonged weathering and crack.

Redressing, recutting, turning of blocks, veneering and the use of synthetic stone have been used as alternatives to the replacement of defective stone particularly where it is difficult to obtain new stone to match the original material. The results of these treatments have often been unsatisfactory. Following the quarrying of stone, pore water (quarry sap) will move towards the freshly cut surfaces and can deposit salts there to form a hard skin at the faces of the blocks. The removal of the outer durable layer by redressing or recutting may result in the accelerated deterioration of the softer material underneath. (Sometimes a case-hardened skin is formed by the deposition of gypsum at the surface of the stone which is underlain by weaker material.)

Portland cement, oxychloride cement and polymers such as epoxy resins, pvas and polyurethanes have been used as the binding agent in synthetic stone. Pigments and crushed stone or specially selected sands are used to match the existing stone. The results have often been disappointing because of colour mismatching, cracking and loss of adhesion, cement staining and discolouration of resin binders. The successful use of this method of repair requires considerable expertise both in the choice of suitable materials and in the method of application.

DURABILITY

Often when serious deterioration necessitates the replacement of badly decayed stone and replacement stone is chosen, the main emphasis is on appearance and the need to match the existing stone, and too little attention is given to the long-term durability of the new material. Renewed deterioration has frequently taken place not long after the repair work was completed. The appearance requirement may not be met either. Sometimes a light-coloured stone was chosen with the expectation that it would darken with time until it had a similar appearance to the original stone, but after a number of years there were still unsightly variations in the general appearance of the building.

It is possible to avoid some of these mistakes by carrying out tests to assess the likely durability of different kinds of stone although there are still no definitive short-term tests available that enable the long-term performance of stone to be predicted with certainty. It is impossible to simulate in the laboratory the weather conditions that will occur at the building site. Not only do exposure conditions differ from one site to another but they can also differ in different parts of a building. One part may be frequently washed by rain and be subjected to wetting and drying or to marked diurnal changes in temperature, while another part may be sheltered from the rain and be in continuous shade. The durability of a particular stone may differ markedly in these different environments. Furthermore, although we have information on the influences of certain properties of stone on durability we still cannot rank accurately their order of importance. Nevertheless in spite of these shortcomings, the tests commonly used for assessing durability can still provide a useful guide when a suitable stone is being selected for renovation work.

A petrological examination will enable the main constituents of the stone to be identified. An examination under the microscope will provide an indication of the degree of soundness of the stone. It may show an abundance of durable minerals such as quartz and certain felspars or the presence of altered secondary minerals and clays which are likely to reduce the resistance of the stone to attack by aggressive substances. If clay minerals are present, X-ray diffraction tests can be used to determine the type of clays present. Some sandstones contain the clay mineral illite which expands in the presence of moisture and may contribute to the decay of the stone. Chemical tests can be used to give an indication of a stone's likely resistance to attack by atmospheric pollutants such as sulphur dioxide and sodium chloride.

As has been previously mentioned, damage by salt crystallisation and hydration is one of the most common causes of stone deterioration and the sodium sulphate test is frequently used to assess a stone's resistance to this type of attack.

Cubes of stone are subjected to a number of cycles of soaking in a 14 per cent (by weight) sodium sulphate solution followed by oven drying. The resistance of the stone to the disruptive effect of the salt attack is gauged by the amount of the weight loss and by visible signs of deterioration. The forms of deterioration resemble those observed in buildings where sandstone is damaged by salt crystallisation, although the test is highly artificial and severe. The total immersion of the cubes in the salt solution does not correspond to a likely practical situation in buildings and the time scale of the attack is grossly distorted. It is difficult to obtain an accurate determination of the net weight loss caused by deterioration because the cubes also gain weight because of salt absorption. Nevertheless the test is still useful in that it enables the durability of different stones to be compared under the same test conditions.

A stone that performs poorly in the test is unlikely to show good resistance to salt attack in a building. At the same time a good result in the sodium sulphate test does not necessarily ensure that a particular stone will perform well in all aggressive environments. Wallace[2] gives an example of a stone which stood up well to the sodium sulphate test but gave poor results when immersed in sulphuric acid because it contained calcite which readily dissolved in the acid. This stone would be unsuitable in an environment that contained a large amount of sulphur dioxide.

There are other accelerated weathering tests that are used to assess the long-term performance of stone in aggressive environments. The Weatherometer is used to expose small specimens to cycles of heating, ultraviolet radiation and wetting and drying. In other tests the specimens are exposed to salt sprays, or to sulphur dioxide or steam. All these tests are used mainly for comparison purposes and are not as reliable as long-term field exposure tests, but where it is not possible to wait years for the results of the field tests they can be of help in choosing suitable stone or in assessing the likely effectiveness of preservative treatments.

Tests to measure porosity, permeability and water absorption are useful in that they will indicate whether aggressive solutions will be able to penetrate readily into the stone, but it should be mentioned that pore size distribution has a greater influence on resistance to salt attack than total porosity. Stone with small pores is more likely to suffer damage from the expansive forces associated with salt attack than stone with larger pores which provide more relief to these expansive forces.

The relationship between mechanical strength and durability has not yet been clearly established. Wallace stated that no high strength stone tested by him gave poor results in the sodium sulphate test although some low strength stones gave good results in the sulphate test. Transverse strength tests have more relevance to stone deterioration than compressive strength tests because disruptive forces associated with internal expansion are more likely to generate damaging tensile or splitting stresses than compressive ones. The transverse strengths of sandstones are often reduced appreciably when they become saturated, and tests should be carried out on test prisms in the dry and wet states to determine the magnitude of this decrease in strength.

Although during the period in the nineteenth and early twentieth century when many important sandstone buildings were constructed in Sydney there were no widespread attempts to sort out good quality stone from bad, and often good and bad material were placed side by side, there were a number of quarries operating from which durable stone could be obtained. Most of these quarries have since closed down and today it is often difficult to match existing stone and to choose suitable, good quality material from the few quarries that are still open. Of the sandstones that are available, Wondabyne from the Gosford area and Yellow Block from Bondi are considered to be among the most durable. The choice of suitable material is made even more difficult because it is possible not only for stone to show marked variations in its properties from one quarry to another, but also to vary within one quarry. This is illustrated in the test results obtained by Golding[3] on samples taken from different levels of two Gosford quarries. There were differences in density, porosity, absorption and in quartz, clay and carbonate contents, Fig. 1. The graphs show that generally as the sand content decreases and the clay content increases, the porosity and absorption decrease. Increasing clay content and decreasing sand grain size are often associated with a reduction in the total porosity (and probably also in the pore size) so that this stone does not absorb water as readily as sandy, coarse-grained material. This is also illustrated in Fig. 2 where Guinea Gold stone from Gosford, which consists mainly of sand, absorbed water much more rapidly than either the Bondi or Wondabyne stones which contain more clay, and are less porous than the Guinea Gold stone, Table 1. (The graphs in Fig. 2 were obtained from a test to determine the initial rate of water absorption by 50-mm sandstone cubes. The test provides a useful guide on the pore structure of a stone in that an absorption curve which rises sharply and then flattens out is usually associated with an absorbent quartz sandstone with high macroporosity. Whereas dense argillaceous stone with pronounced microporosity usually absorbs water at a slower rate after immersion. The absorption curve increases slowly but steadily during the two-hour test period.)

The results of EBS tests on the three kinds of stone show that the Guinea Gold stone had the highest absorption and effective porosity and also the lowest modulus of rupture when tested in a saturated condition which suggests that this stone would probably not be very durable if it was placed in an aggressive environment.

Guinea Gold has been used extensively for repair work mainly because of its pleasing, buff colour but has frequently begun to deteriorate not long after it was placed in a building.

Successful repairs of stone deterioration are unlikely to be achieved unless the cause of the deterioration is diagnosed correctly and remedied. In addition to a petrological examination of the stone, it will often be helpful to analyse drillings taken from the in-situ stone to determine the nature and distribution of the salts present, where salt attack has occurred.

Table 1. PROPERTIES OF SANDSTONE

Type	Max. Absorption per cent (by wt)	Effective Porosity per cent (by vol.)	Bulk dry Density	2 hour Saturation Coefficient	Modulus of Rupture (Air Dry) (MPa)	Modulus of Rupture (Saturated) (MPa)
Bondi	6.4	14.8	2.3	0.4	4.7	3.1
Wondabyne	6.7	15.5	2.3	0.4	7.5	3.0
Guinea Gold	7.8	17.5	2.2	0.5	4.7	2.8

Spry[4] carried out a survey in 1976 of a number of stone buildings which had been repaired and found many instances where the work had been unsuccessful and renewed deterioration had taken place. It appears that large amounts of money and effort have been wasted because of a continued repetition of old mistakes and a lack of knowledge of sound methods of restoration. There is an urgent need to document restoration work and to monitor long-term performances so that people working in the field can profit from information on past successes and failures and use this knowledge to advantage in future work.

CLEANING

The cleaning, repair and protection of stone work in buildings is often carried out together. Indeed many protective treatments cannot be carried out unless the building is first cleaned. The cleaning may therefore have a dual function to improve the appearance of a building and also to help to preserve it by removing harmful surface deposits that promote the deterioration of the stone. In other situations however, the use of an aggressive cleaning treatment may not only remove the dirt but also a durable surface layer which previously protected the less durable stone underneath from deterioration. Accelerated attack may take place after this protective layer is removed. Some cleaning treatments are effective in removing deeply ingrained dirt and can restore a pristine appearance to a building but at the expense of causing irreparable damage to the stone. Sometimes this damage may not become apparent until a long time after the cleaning has been completed.

It would be preferable to use a treatment that does not damage the stone even if it does not remove all the dirt. Unfortunately a survey of the commonly used methods of cleaning indicates that some of the treatments that are less likely to damage the stone are often of limited effectiveness on badly soiled buildings.

Some cleaning methods rely on abrasion or scarification to remove grime. They include the use of sand or grit blasting (both of which can be done wet or dry), high pressure water jets, revolving carborundum wheels and wire or fibre brushes. Scarification can be done manually with chisel and scraper or mechanically with pneumatic chisels and needle guns.

All these methods depend largely for their effectiveness on the skill and experience of the operator.

Their over-enthusiastic use may lead to a loss of architectural detail or damage to carved surfaces and mortar joints, and they may also have a harmful effect on the durability of the stone. There are usually dust problems associated with the dry processes. Some form of abrasion is often used to remove paint or render from masonry.

Damage caused by abrasion is less likely to occur with washing processes that rely on prolonged soaking with low pressure sprinklers or hosing together with manual brushing to loosen and remove dirt, but these methods may not be effective in cleaning badly soiled buildings satisfactorily. Steam cleaning has been used widely in the past but it is slow and not considered to be as effective as some of the other methods being used at the present time.

Chemical methods are frequently used together with water or steam cleaning to remove grime and are often effective where other treatments are unsuccessful. Many of them however have a detrimental effect on stone, particularly sandstone, and should not be used on some of the local stone. They include hydrofluoric, hydrochloric and phosphoric acids. Hydrofluoric acid will attack all the minerals in sandstone (as well as adjacent glass and metals). It is an extremely dangerous substance and can create serious health hazards for the operator. Although it is one of the most effective chemicals available for removing dirt it may leave a whitish deposit of colloidal silica on the stone or may bleach it and leave the building with a drab and lifeless appearance. The cleaning of the Customs House is an example of this defect. Phosphoric acid can also have a bleaching effect on sandstone. (It is used to remove iron stains.) The use of hydrochloric acid cannot be recommended either, because even if the stone is flushed with water afterwards to remove the acid it may have already reacted with some of the minerals in sandstone and result in the deposition of detrimental chloride salts in the stone.

Acid cleaners may also dissolve the calcareous or ferruginous cement in sandstone and react harmfully with the clay minerals. Even though these aggressive cleaning agents are allowed to act for a short time and are then removed by washing the stone with plenty of water, they can still cause damage.

Ammonium bifluoride has been used as an alternative to hydrofluoric acid. It is less corrosive and toxic than the acid but is still an aggressive material and can react harmfully on sandstone. Caustic alkalis such as sodium and potassium hydroxides are not recommended for similar reasons to those mentioned concerning acid treatments. They can form harmful salts and may stain or bleach sandstone.

Detergents are frequently used with steam or water cleaning to help remove grime. Some of them contain additives which can have a harmful effect on stone. As it is undesirable to allow ionic salts, or acids or alkalies to enter the stone, non-ionic detergents which do not contain these additives should be used for cleaning purposes. Some detergents attack the clay minerals in sandstone. Hence there is a need to carry out preliminary tests on the suitability of these materials before their use is permitted. The wetting action of detergents may result in the deeper penetration of harmful substances into the stone after the cleaning treatment has been completed. Each section of the building should be thoroughly wetted with clean water before the detergent is applied and thoroughly washed down immediately after the stone has been cleaned.

It can be seen that the choice of a safe and effective method of cleaning stone is often difficult. Before any treatment is used, trials should be carried out to determine its suitability. Laboratory tests will help to identify possible harmful effects on the stone and they should be supplemented by field tests on a small area of stonework in an inconspicuous part of the building.

RISING DAMP

At a previous seminar a paper was presented on rising damp and although many of the problems dealt with in that paper apply to stone preservation it is not possible to cover the subject again today. Deterioration caused by salt contamination is frequently caused by rising damp and restoration work in a masonry building may also require the treatment of a rising damp problem. Where the problem has been present for a long time, future preservation of the stone may require the removal of the damaging salts either by repeated flushing with water, by poulticing with absorbent clays or by the use of a sacrificial render coat.

PROTECTIVE TREATMENTS

Overseas investigations have been carried out for many years into methods of preserving and protecting stone work in buildings. Generally the results have been disappointing.

There are three main classes of materials that have been used for this purpose; waterproofers, water repellents and stone consolidants.

1. Waterproof coatings

The purpose of these materials is to make the stone impermeable to water penetration in the liquid or vapour form. They include oils, waxes, bitumen, varnishes, paint, metallic stearates, lime wash and cement and plaster renders.

Some of these materials will be unsuitable on aesthetic grounds or where the stonework cannot be concealed. Linseed oil and some waxes and varnishes are unsuitable because they darken or collect dirt. Paint coatings have often broken down within a short time after application. Their long-term durability will be influenced by the effectiveness of the preparation of the substrate and failures have often occurred because of unsatisfactory preparation of this surface. If the stone is contaminated with salts and there is moisture behind the painted surface a premature failure is likely.

Field and Weatherometer tests carried out by EBS and the Institute of Technology on an aluminium stearate showed that it lost its water repellency property within a short time.

Limewash has been used for centuries on masonry buildings but overseas investigators have expressed doubts about its value as a protective coating.

Where deterioration of the stone has taken place, the use of a cement or plaster render may provide a more effective method of protection than some of the methods just described. A 1 : 1 : 6 or a 1 : 2 : 9 mix (by volume) is suitable for this purpose as it will allow moisture to evaporate from the wall. If there is an excessive amount of cement in the mix it will act as a barrier to evaporation and may force internal moisture to move into other parts of the masonry. The use of a porous render will encourage the movement of salts from contaminated stone into the render as moisture evaporates from the wall. The build up of salt deposits in a sacrificial render can lead to its eventual deterioration but at least the stone will be protected against decay if salt disposition takes place in the render rather than in the stone.

2. Water repellents

These materials have been used widely to prevent rain penetration. They can be applied easily and as they are usually clear solutions they do not hide the stone. Silicones and siliconates are the most commonly used materials in this class, although tests carried out by the BRE, which are described below, throw some doubts on their long-term effectiveness.

In contrast to the waterproof coatings, they line but do not block the pores of the stone and hence allow it to breathe. Water often penetrates behind the treated surface layer either by passing through it in the vapour phase or by absorption of rain or ground water at some other unprotected location. Subsequently this moisture may evaporate from behind the repellent layer and any salts in solution crystallise there and cause spalling and exfoliation of the treated stone. Differential thermal and moisture movements within this thin surface layer may also contribute to accelerated deterioration. Silicones are applied with a brush or spray on clean, dry surfaces. They have a limited life and in polluted atmospheres may not keep the buildings clean.

Siliconates contain alkaline salts which can react harmfully with the stone.

3. Consolidants

This group of materials is used to consolidate friable stone, to restore or increase its tensile strength and to prevent additional crystallisation damage either by making the stone more resistant to crystallisation forces through modification of properties such as its pore size distribution or by making the salts inaccessible to water. Some of these materials have both water repellent and consolidating properties. It is also important that they should penetrate the stone readily. The BRE recommends a minimum depth of penetration of 25 mm[5].

The most commonly used materials in this group are silicon-based and include inorganic and organic silicates, fluosilicates, alkoxysilanes and alkalkoxysilanes. Chemical reactions within the stone lead to the formation of silica deposits in the pores of the stone. The success of the treatment will depend partly on the effectiveness of the penetration and in some stone it is difficult to achieve this goal. Effective penetrations is sometimes achieved by mixing the consolidant with a solvent of low viscosity. This technique is used with some of the silanes. However the loss of an alcohol solvent by evaporation results in incomplete filling of the pores.

Polymers have also been used extensively as stone consolidants. Polymerisation is usually carried out in-situ and may be achieved by heat, radiation or by the use of chemical catalysts. The first two methods have limited application for field use or for the treatment of large masses of stone. Acrylic resins such as methyl and butyl methacrylate have been used as consolidants and moisture barriers, but there are conflicting reports about their effectiveness, Although epoxy resins have good strengthening properties they cannot be recommended because they can discolour and deteriorate when exposed to sunlight. There may also be problems in obtaining good penetration with epoxies. Acrylics are generally more resistant to sunlight than epoxies but they also may deteriorate with time.

In general the use of synthetic consolidants on Australian stones should be considered with caution, partly because there is still insufficient information available on their long-term performance and also if they are found to be unsuitable it will usually be difficult if not impossible to remove them and to carry out other kinds of protective treatment on the stone. Reports of overseas successes may not be relevant to local conditions and it will be unwise to use a consolidant in a building before carrying out laboratory and field trials even if this involves a lengthy delay in making a decision of its usefulness.

Overseas tests suggest that among the silicon-based consolidants the alkalkoxysilanes which have both strengthening and water repelling properties show the most promise.

An alkoxysilane was used extensively in restoration work in the Cologne Cathedral and was applied by brushing. However when this material was used by EBS on Sydney sandstone, it was found that the solution only penetrated a few millimetres into the stone even when it was used with an ethyl alcohol solvent. (The presence of clay minerals in some of the local stone results in much lower water absorptions than are commonly associated with European building stones.) Satisfactory penetration could only be achieved by using vacuum impregnation which would place serious limitations on the use of this type of consolidant in sandstone buildings.

The costs of many of these consolidants are high and their application on large areas of a building could be ruled out on economic grounds. However they may be useful for the conservation of valuable ornamentation and architectural detail.

A series of long-term field tests on stone preservation was carried out by BRE and the Department of the Environment in the U.K.[6] in which the preservatives were applied by brushing on large wall panels in 24 different sites. Comparisons were made between adjacent treated and untreated panels at regular intervals of time. The periods of observation were generally between 4 and 6 years and most of the work was done between 1964 and 1970.

The materials tested were either consolidants, water repellents or combined consolidants and water repellents. They consisted mainly of silicones, siliconates, silicon esters, and lime washes.

The results of these tests were largely negative and there was no noticeable reduction in the rate of decay. In some instances the treatments resulted in accelerated decay. The use of silicones frequently resulted in darkening of the stone or noticeable dirt streaking. On some sites there was a loss of water repellency within three years of the application of the silicone. Often there was a re-occurrence of algal growth during the test period. A silicone, a silicone ester, a siliconate and a lime wash were applied to salt contaminated walls in the Salt Tower and none of these treatments prevented a continuation of the deterioration. The investigators were of the opinion that desalination would probably be necessary to overcome the problem.

The U.S. National Bureau of Standards[7] published a report in 1977 on the results of a comprehensive series of laboratory tests carried out on 54 preservatives including silicones, acrylic polymers, fluorocarbons and silicates. None of the materials tested met all the performance criteria, although the tests indicated that some preservatives might be helpful in protecting stone against certain types of attack. The authors pointed out that because of a lack of an acceptable accelerated laboratory test that related closely to field conditions it was difficult to obtain reliable information on the likely long-term performance of the preservative on a building. Also a preservative might be effective on one type of stone in a particular environment and be ineffective on another stone in a different environment. In the evaluation of a new product it is essential to supplement laboratory tests with long-term field trials carried out in a similar environment to that experienced by the building to be repaired. One of the main accelerated tests used in this investigation involved exposure to infra-red and ultra-violet lamps, water sprays, sulphurous acid, soaking in sodium chloride solution and to freezing and thawing. It was noticeable that only 9 of 52 preservatives subjected to this test did not undergo a significant colour change after exposure to 20 cycles of this weathering test.

CONCLUSIONS

A large part of the information given in this paper has been of a cautionary nature and a number of examples have been given of failures or partial successes in the repair and preservation of old masonry buildings all of which suggest that there are still many gaps in our knowledge of the subject.

There is a need for the collection and dissemination of information on the preservation of masonry buildings so that the past mistakes are not repeated.

Some attempts are being made to remedy this deficiency following the affiliation of an advisory group to the Heritage Commission to provide technical advice and information on the restoration of old buildings and to keep abreast of current developments in this field. The group will welcome information on the long-term performance of restoration treatments.

At present practitioners should adopt a cautious approach to restoration problems and they should carry out thorough preliminary tests where there are any doubts about the effectiveness of the treatments or procedures under consideration.

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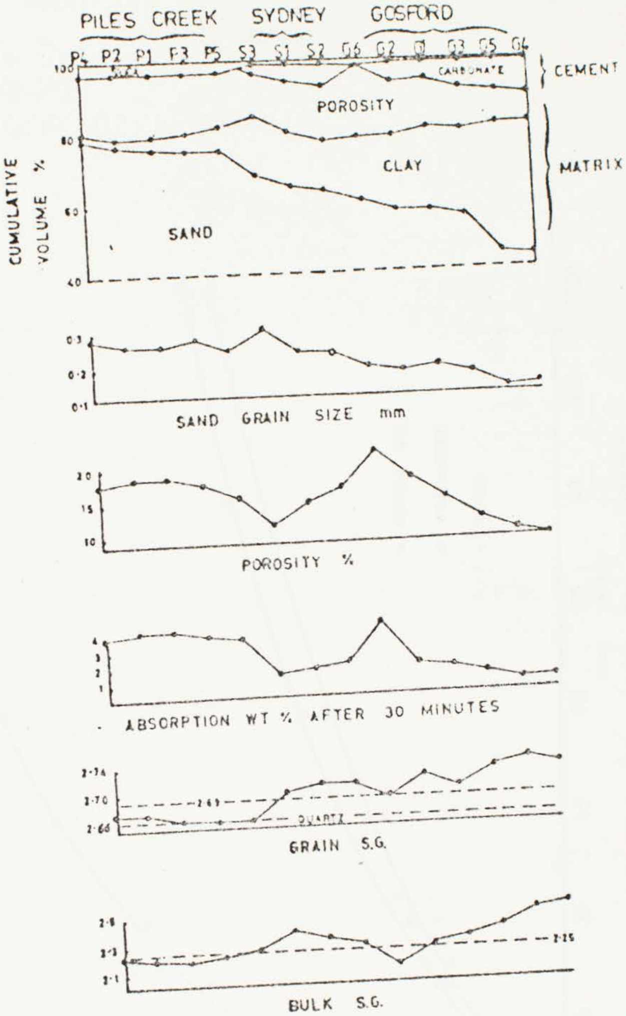

FIG. 1 EFFECTS OF CHANGES IN SAND CLAY CONTENT ON SANDSTONE PROPERTIES

FIG. 2

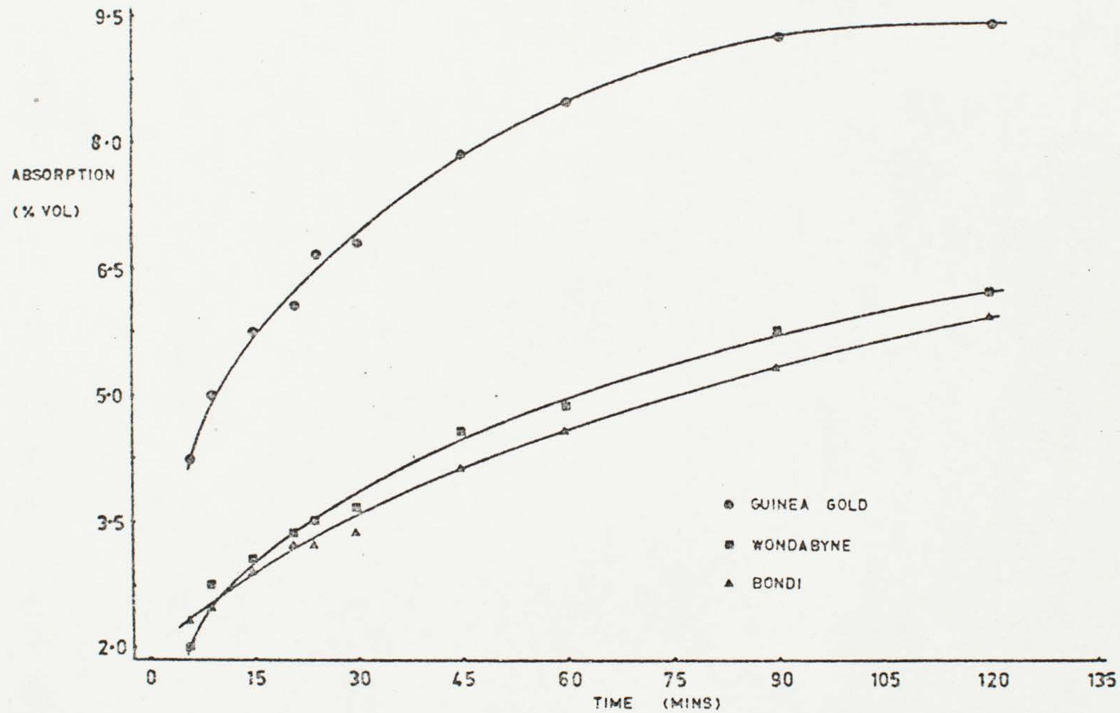

FIG. 2 RATE OF ABSORPTION OF WATER BY SANDSTONE

17.25

COLIN GURLEY
NSP

AN EVALUATION OF METHODS OF TREATING RISING DAMP

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INTRODUCTION

Rising damp (also known in some areas as salt damp) is a common cause of deterioration in stone and brick buildings. If groundwater comes in contact with the base of a masonry wall, because of the porous and absorbent nature of the brick or stone, moisture will tend to rise some distance up the wall by capillary action unless there is an effective barrier to stop this movement. Consequently rising damp problems are frequently encountered in old masonry buildings where either there is no damp-proof course or the damp-proof course has broken down. Slate damp-proof courses are often troublesome in this regard.

A typical sign of rising damp is a roughly horizontal tide mark on the wall above which there is little or no damage, but below which the paint or plaster has been damaged or the wallpaper is stained or has lifted. The moisture that moves up the wall often contains dissolved salts, usually chlorides or sulphates, and they are deposited near or at the wall surfaces where evaporation takes place. Efflorescence may appear and surface coatings and the masonry may deteriorate as a result of salt crystallisation.

Mould growths may appear on persistently damp surfaces. On external surfaces, efflorescence and mould growth may also be accompanied by fretting of the stone or brick and crumbling of the mortar. There may be a musty smell in the affected rooms and prolonged dampness may lead to the rotting of the skirting boards. The damage is usually restricted to a zone which does not extend more than 1 m to 1.5 m above the floor, although in very damp situations the damage may occur at higher levels, particularly where there is poor ventilation.

The height of the capillary rise will be influenced by the pore size of the masonry. The smaller the pores the higher the capillary rise. It will also be influenced by the rate of evaporation from the wall. An increase or decrease in the rate of evaporation will be accompanied by a fall or rise in the height of the zone of dampness.

The height of the capillary rise of moisture in a wall will also be influenced by the height of the water table. Hence the size of the damp zone in a wall may fluctuate with seasonal changes in climate. During a dry time the soil around the base of the wall may draw moisture out of the masonry. A wet season accompanied by a rise in the water table can result in an increase in the height of the capillary rise of moisture.

Mason[1] has pointed out that in addition to these relatively short-term moisture movements, long-term changes can occur. Calcium hydroxide is sparingly soluble in water and may be slowly leached out of lime mortar resulting in an appreciable increase in the pore size and hence in permeability. The resulting increase in the height of the capillary rise may continue for many years. He also mentions that if the groundwater is hard and contains salts that react with the hydroxide to form calcium carbonate, the precipitate may slowly block the pores and the height of the capillary rise will gradually diminish. It appears therefore that a number of factors can influence the movement of moisture in masonry and that they can interact in a complex manner.

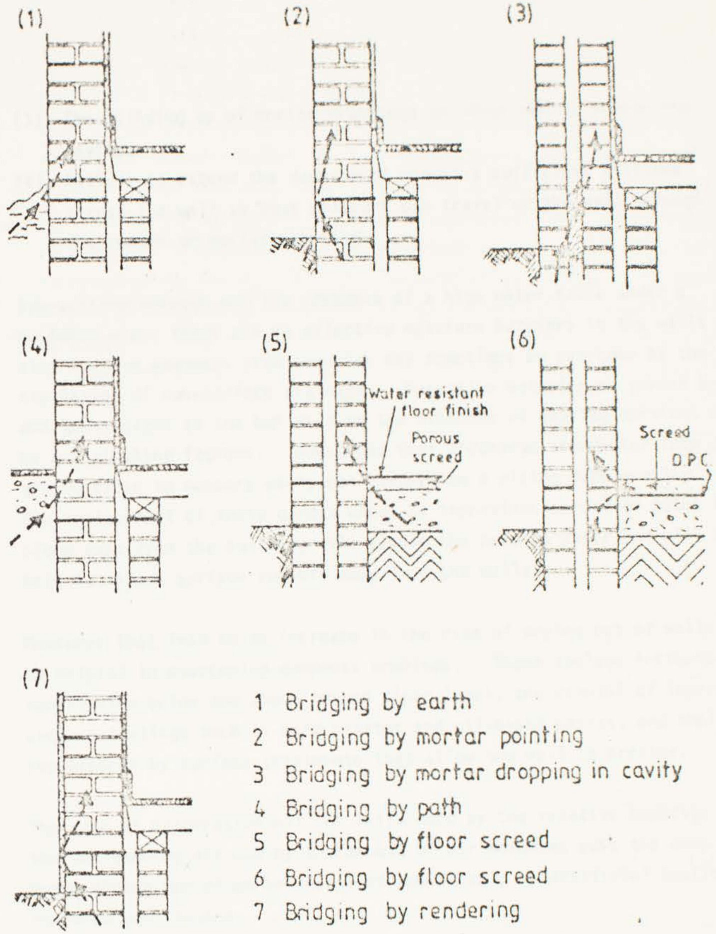


FIG 1. BRIDGING OF DAMP-PROOF COURSE

The EBS is presently carrying out an investigation into moisture movements through brick walls made with sandstock bricks obtained from old demolished cottages and from lime mortar. The walls were placed in water to a depth of 50 mm and subsequent moisture movements through the brickwork have been monitored with microwave equipment and by measuring the volume of water required to replace evaporation losses from the walls. There has been a gradual reduction in the moisture transmission suggesting that the pores in the lower mortar joints are becoming blocked. The microwave measurements have shown that only the bottom four or five courses above the water line are moist. Above this level, both the exposed and rendered brickwork are comparatively dry.

Ineffective treatment of a dampness problem can result from a faulty diagnosis. Damage caused by rising damp is sometimes confused with that caused by falling damp where the moisture comes from leaking roofs or downpipes, or through defective flashings or from blocked gutters. Sometimes the dampness is caused by the lateral penetration of rain-water through solid masonry walls where the penetration may take place through porous bricks and through porous or defective mortar joints. Sometimes it is primarily a condensation problem and there are occasions when the dampness is the result of a combination of causes. A treatment suitable for one type of dampness problem will be inappropriate for another and a thorough examination of the building should be carried out before any remedial measures are undertaken.

Rising damp problems can be caused by the bridging of the damp-proof course which allows water to rise up the wall above the level of the membrane and which can sometimes be rectified without much difficulty or expense (Fig. 1.).

Some common sources of this bridging action are:

(1) The heaping of soil or debris against the wall;
(2) The location of a concrete path or floor slab directly against the wall;

(3) The building up of mortar droppings or other debris inside the cavity;
(4) Failure to extend the damp-proof course a sufficient distance across the wall so that moisture can travel up the wall through the render or mortar pointing.

Sub-surface seepage and the presence of a high water table under a building where there are no effective moisture barriers in the walls can also cause a dampness problem which may sometimes be overcome by the installation of sub-surface drainage. Excessive watering of garden beds and lawns close to the building or the presence of leaking services can be contributing factors. Downpipes that discharge stormwater into the ground close to masonry walls can exacerbate a rising damp problem. The replacement of these garden areas by impervious aprons or paths that slope away from the building will remove the need to water and will also help to divert surface run-off away from the walls.

Measures that lead to an increase in the rate of drying out of walls may be helpful in overcoming dampness problems. These include increased ventilation below and above ground floor level, the removal of impervious surface coatings such as hard plaster and oil-based paints, and their replacement by surface treatments that allow the wall to breathe.

The rate of evaporation will be influenced by the relative humidity of the surrounding air and by the amount of air movement past the damp wall. Hence drying can often be increased appreciably by artificial heating and improved ventilation.

At the same time it should be borne in mind that elimination of the dampness by these methods may not necessarily solve the problem completely. An increase in the rate of drying may also result in an increase in the amount of salts deposited at the wall surface and deterioration may increase rather than diminish at least for a while after the drying out process has been accelerated. Where hygroscopic salts have accumulated in old render or plaster, these coatings should be removed after a treatment to prevent rising damp has been carried out. It will be beneficial not to replace the coating until some months later in order to assist the drying out of the wall. It may be possible to remove by brushing the salts which are brought to the surface by the increased drying. However where walls are heavily contaminated it may not be possible to remove a sufficient amount of salt by this method and poulticing or flushing techniques described later will be required to reduce the salt content to a satisfactory level. Very thick stone walls may take years to dry out completely.

OTHER METHODS OF TREATING RISING DAMP

1. Insertion of a new damp-proof course

The insertion of a new damp-proof course often provides an effective method of overcoming a rising damp problem provided the installation is carried out satisfactorily, but high costs or practical difficulties may rule out its use. It can be accomplished by removing a few bricks at a time from a course of brickwork near the base of the wall, inserting a section of the damp-proof course along the horizontal bed joint, replacing the bricks and mortar and repeating the process until a continuous membrane has been placed in the brickwork affected by the dampness. A less tedious method is to cut a narrow slot for a short distance along a bed joint with a power driven reciprocating saw or with an abrasive disc. Special chain saws have been used for this purpose. Thick walls over 600 mm thick have been slotted successfully with a high pressure water jet.

With this equipment it is often possible to carry out the repairs working on one side of the wall. Cavity walls are usually slotted from both sides. It may also be necessary to re-locate any services placed in the walls. The membrane is inserted immediately after the slot is formed and is laid in strips about 0.5 m long with lap distances of at least 100 mm. In heavily-loaded sections of the wall, for example, near joints and intersections it may be necessary to cut the slot in shorter lengths. Copper, lead, bitumen-coated aluminium or black polyethylene sheet have been used to form the moisture barrier. Care has to be taken in the insertion of the polyethylene sheet to ensure that it is not torn. Uncoated metallic membranes may corrode if they are placed in masonry containing appreciable amounts of aggressive salts and in this situation it would be preferable to use bitumen-coated aluminium or polyethylene. There is a widespread belief that lead is one of the most durable damp-proof course materials. This is not always correct. Lead is attacked by the lime present in mortars while the lime is undergoing carbonation and should preferably be protected by a bituminous coating. Furthermore the lead produced today has less impurities than previously and as a result is more likely to become brittle and crack. The manufacturers of a new type of lead damp-proof course that contains traces of copper claim that this material is more flexible and durable than sheet membranes made from pure lead.

Pieces of slate, asbestos-cement sheet or other suitable packing are wedged in the gap above a flat membrane material to prevent damaging settlement of the masonry. The remaining space is then packed with mortar.

Restricted access often rules out the insertion of a new damp-proof course. It may also be impracticable to use this method in random rubble wall construction. If as has already been mentioned, the walls were heavily charged with salts, the insertion of a new damp-proof course may result in an increase instead of a decrease in the rate of deterioration unless the salt content is reduced.

2. Chemical injection treatments

The aim of a chemical injection treatment is to form a barrier to moisture movement either by blocking the continuous voids in the masonry or by forming a water repellent film on the walls of pores and capillaries which alters the surface tension forces at the solid-liquid interfaces and thereby impedes the upward movement of moisture. Mixtures of rubber latex and siliconates have been used as pore fillers. Solvent-based silicones and aluminium stearates, and water-based siliconates have been used to form repellent films. Solven-based silicones polymerise in the presence of moisture to form silicone resins, whereas the curing of water-based siliconates requires the presence of carbon dioxide in addition to moisture to form resins which are deposited within the masonry. Aluminium stearate adheres to the treated surfaces to form a water repellent film after the solvent has evaporated. As these water repellents line but do not block pores and capillaries they allow the passage of water vapour through the masonry and they are unsuitable for damp-proofing basements or walls subjected to hydrostatic pressure.

In this treatment the chemical solutions are injected under pressure or percolate by gravity into the masonry through holes drilled at intervals near the base of the wall. The success of the treatment will depend largely on the effectiveness of the penetration. The formation of a continuous barrier will be influenced partly by the choice of a suitable spacing of holes. Ideally the saturated zone of masonry formed around each hole by the injected solution should overlap those formed around the adjacent holes. In practice it is not always possible to meet this objective and this is one reason why chemical injection treatments are not always successful. For example a mixture of rubber latex and a siliconate had been injected successfully overseas under pressure into different types of masonry and was claimed to form an effective moisture barrier, but in tests carried out by the EBS and the Division of Building Research of the CSIRO it was not possible to obtain satisfactory penetration because the pore sizes in the types of masonry treated were smaller than the rubber particles.

In injection tests carried out by EBS in sandstock brickwork, good penetration was achieved with a mixture of silicone and white spirit, injected under a pressure of 0.4 MPa. However it may be more difficult to obtain satisfactory penetration in brick or stonework with a low permeability.

Injection tests on Sydney sandstone using the same mixture were unsuccessful even though the pressure was increased to 0.8 MPa. Too small a quantity of the silicone solution penetrated through the stone to form an effective moisture barrier.

Sometimes effective penetration by pressure injection cannot be achieved because an excessive amount of fluid is lost in cavities and fissures in the masonry. The presence of weak mortar unable to withstand the pressure of the injected solution may also lead to excessive losses and incomplete penetration.

It is usually much easier to obtain effective penetration in dry masonry than in saturated material. In order to form a complete barrier across the treated wall it would be necessary for the injected fluid to displace the moisture in the pores and capillaries. The Division of Building Research of the CSIRO have carried out tests in walls with high moisture contents and they recommend that white spirit should be injected into the masonry to force out excess water that may be present prior to the injection of the silicone solution.

The presence of large residual islands of moisture could prevent the formation of an effective barrier to the rising damp. On the other hand although a complete moisture barrier may not be formed across the wall there may be a sufficient reduction in the upward moisture movement to achieve a worthwhile improvement in the dampness problem.

As these materials have only been in use in Australia for a few years, reliable information is not yet available on their long-term effectiveness. Chemical injection systems have been used successfully in the U.K. for a longer period and many installations are still working successfully more than ten years after installation. The British Agrement Board is of the opinion that the silicone injection system should be effective for at least twenty years. It does not necessarily follow however that this information is relevant in Australia. Work done at

Other overseas investigators have also expressed doubts about the effectiveness of the system.

The EBS and DBR investigated two buildings, some years ago in which the system had been installed and could not find any evidence to indicate that the treatments had contributed to the drying out of the walls(3) Potential differences were measured both in the walls of one of the houses and also in brick and sandstone laboratory specimens, but these measurements did not agree with the theory put forward by the promoters of the system. The measured potentials fluctuated erratically and if they were caused only by the movement of water then the moisture in both the walls and the test prisms must have been moving up, down and sideways in a very strange manner. It is more likely that these changes in potential were caused by variations in salt concentration than by the causes described in the electro-osmotic theory.

In a more recent field survey, the EBS found a few successes and some failures. Electro-osmotic damp-proofing installations carried out in South Australia to combat salt damp have been markedly unsuccessful.

4. Cementitious grouts

There is another type of proprietary damp-proofing treatment which relies on the introduction of a cementitious grout into the masonry to form a moisture barrier. This grout consists of a mixture of portland cement, sand and chemicals which when they come in contact with moisture in the masonry migrate through the capillaries as a result of osmosis and then crystallise. The crystals reduce the size of the voids and create a barrier to the passage of moisture. The grout can be introduced into walls through holes drilled in the brick or stone or it may be applied in the mortar joints. In one method, a stiff grout is packed into a continuous shallow groove formed in one of the bed joints near the base of the wall. The manufacturers of these products claim that the treatment has been used successfully overseas for a number of years. It has only been introduced into Australia a few years ago and as yet there is no independent information available on its long-term performance in this country. The EBS has carried out laboratory tests on

two proprietary treatments of this type in which the grout was introduced into prisms made with sandstock bricks and lime mortar. In both series of tests an extensive crystal growth developed over the external surfaces of the specimens but there was no significant reduction in the upward moisture movement through the brickwork.

5. Damp-proof mortars

Damp-proof mortars in which admixtures such as stearates or bituminous compounds are added to reduce the permeability of the mix have been used for many years in parts of Australia, particularly in Victoria and South Australia. They are usually cheaper than sheet damp-proof membranes and in many instances they have prevented rising damp problems. But there have also been reports of failures and it is generally recognised that damp-proof mortars are not as reliable as sheet membranes. Where failures occurred they were often attributed to faulty workmanship or unsuitable mortar mixes. Mix proportions were specified incorrectly or the mortar was not batched according to the specification. Sometimes unsuitable sands with high clay contents were used, and excessive shrinkage and cracking of the mortar resulted. Damaging foundation movements may also result in cracking of the joints and dampness problems. The use of admixtures will not prevent dampness in these situations.

6. Knapen tubes

It was mentioned previously that rising damp problems can sometimes be reduced or even overcome by increasing the rate of evaporation of moisture from walls.

A method that has been used to increase the evaporation is to insert Knapen tubes in inclined, blind holes drilled at intervals close to the base of the walls. Knapen tubes are made from lightly-fired clay or from perforated metal or plastic and it is claimed that the air currents set up inside them increase the rate at which moisture evaporates from the walls. They are grouted into the holes and the exposed ends are covered with perforated caps. The manufacturers claim that moist air flows out through the lower part of the cap and is replaced by drier air flowing into the tube through the upper part of the cap.

Tests carried out overseas on plastic, metal and ceramic tubes showed however that the best results were obtained when holes were used without any tubes inserted in them. The performance of the Knapen tubes was also found to relate to the type of exposure; the walls of some unheated rooms that did not receive any direct sunlight were reported to become damper instead of drier after the insertion of the tubes. Laboratory tests carried out at the EBS on similar brick wallettes with and without ceramic Knapen tubes showed that the installation of the tubes resulted in a slightly lower rise of moisture up the brickwork than in the untreated wallettes but did not solve the dampness problem.

Knapen tubes have been used extensively in South Australia to combat salt damp but the results have been disappointing. Many of the tubes disintegrated within a few years as a result of disruption caused by salt crystallisation within the tube walls.

7. Surface treatments

The application of impervious surface coatings on walls where there is rising damp may not provide a permanent and effective solution to the problem. Often it will result in an increase in the height of the capillary rise of the moisture or may result in the dampness appearing on parts of the walls which were previously unaffected. There is also a possibility of a failure either because of a loss of adhesion or a breakdown of the coating caused by disruptive forces associated with salt crystallisation close to the interface between the coating and the masonry. Nevertheless there are situations where because of excessive costs or practical difficulties, none of the treatments previously described can be used and concealment of the dampness is considered.

Cement renders have been used to prevent dampness from reaching the surface of the wall. The mixes often include a water-proofing admixture or an air-entraining agent to reduce their permeability. The old plaster is removed to a height of about 0.5 m above the line of rising damp and is replaced with the render. Water-based epoxy resins have also been used for this purpose, although they had not always been successful.

Two different surface treatments were tested as moisture barriers on the walls of a brick cottage in the Glebe project where there was a rising damp problem. A 1 : 1 : 6 (by volume) render containing a silicone solution was applied on one wall and a water-based epoxy resin was painted over other re-rendered walls. Neither treatment was successful. Salt deposits and other signs of dampness re-appeared on the walls given the silicone treatment a few months after its application. About the same time, dark patches appeared on one of the walls coated with the water-based epoxy paint and subsequently there was an adhesion failure and the paint began to lift. It seemed likely that neither treatment formed a completely effective barrier to the escape of moisture from the walls and that the deposition and crystallisation of salts in the new render underneath the epoxy coating had a disruptive effect and caused the adhesion failure. The failure of this epoxy coating was discussed with the manufacturer who claimed that better results would have been achieved if a thicker coating has been applied.

If the surface coating develops any cracks as a result of shrinkage of the render or movement of the foundations, the cracks will provide a ready passage for the escape of moisture. Salt deposition may then occur and may be followed by additional damage to the surface coatings.

In another method a thin lead foil is attached to the wall with a special adhesive that is not affected detrimentally by moisture or salts. Wallpaper is then placed over the foil. In another treatment a suitable lining material such as hardboard or asbestos-cement sheet is nailed to timber battens fastened to the wall. It is necessary to impregnate the battens and plugs in a preservative to protect them against decay and to use rust-resistant fastenings. It may also be helpful to place strips of bituminous felt between the battens and the wall. Laboratory tests are at present being carried out at the EBS on a modification of the treatment in which the evaporation of moisture from a damp wall is being prevented by covering it with a polyethylene sheet. (If this method was used in the building, the plastic sheet could be concealed behind a lining.) The wall was covered in three stages: to half height, three quarters height and finally it was completely covered. The reduction in the additions of water required to replace the evaporative losses showed that most of the evaporation took place in the lower half of the wall (Fig. 2). After the wall had been covered to this level little evaporation took place. At the same time however the microwave readings began to increase in the lower part of the wall indicating that the damp zone was slowly rising up the wall. The readings increased most sharply after the wall was completely covered. Six months after the commencement of this treatment, the rise in height of the dampness was still continuing. This phenomenon can also occur in buildings affected by rising damp where the lower parts of damp walls are covered with an impervious surface coating or a lining in the hope that the damage caused by dampness will be either be corrected or concealed. The moisture barrier may eventually force the dampness higher up the walls than before, and after a time it may re-appear above the level of the surface coating or lining. This possibility may sometimes be prevented by extending the surface coating or lining at least 500 mm above the upper level of the original damp area or even extending it up to ceiling level.

There can be other problems associated with the use of the lining method. Condensation may occur on the treated walls unless some form of insulation is placed behind the lining and its projection beyond the wall surfaces may create difficulties especially around openings. In spite of these shortcomings this method has been used successfully in many buildings.

MOISTURE MEASUREMENTS

The accurate measurement of the moisture content of damp walls in the field by non-destructive methods can present problems and a reliable and prompt assessment of the effectiveness of a proprietary damp-proofing system is often precluded because of the choice of an unsatis-

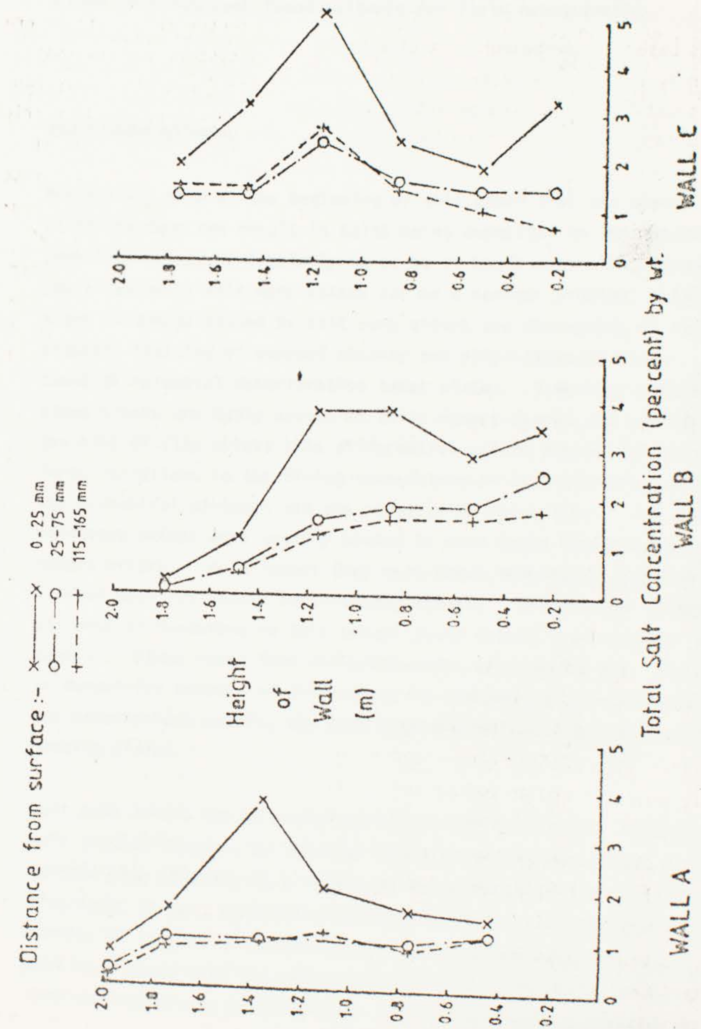


Fig 3 Salt Distribution in Sandstone Walls

factory or inaccurate method of measurement.

Methods based on the measurement of changes in electrical resistance are frequently used by the installers of proprietary systems to show that their treatment is working effectively. The drying-out of masonry can be associated with an increase in electrical resistance but tests carried out by EBS on one of the commonly-used moisture meters showed that it could give grossly inaccurate values of moisture content. The presence of salts in masonry will have a significant influence on resistance measurements and can cause large variations in readings independently of variations in the amount of moisture present in the masonry. Furthermore, resistance readings are usually taken at or close to the surfaces of the wall where most of the drying takes place and they do not give an accurate indication of the moisture content of the interior of the wall.

Another commonly-used field method involves the drilling of holes in the masonry and the determination of the moisture content of the cuttings by placing them in a calcium carbide meter. The moisture in the cuttings reacts with the carbide to generate acetylene gas. A reading on a suitably calibrated pressure gauge attached to the meter enables the moisture content to be measured. Tests carried out by BRE have shown that this method will only give reliable results provided a special measurement technique is used and the instrument is properly calibrated using undisturbed samples of the brick or mortar in the wall under examination. Moisture will be lost from the drillings as a result of heat generated from the bit. On the other hand in very wet walls the drillings may absorb moisture and an excessively high reading will result.

Other factors such as the sharpness of the bit, the speed and pressure on the drill, the initial temperature of the drill and the properties of the masonry will also influence the moisture determination. Reliable results can only be obtained by carrying out preliminary calibration tests in which the results obtained from the meter are compared with those obtained from tests on pieces of similar brick or mortar whose moisture contents are determined by drying them to constant weight in an oven.

Other non-destructive techniques involving the measurement of changes in capacitance or the use of microwaves have been used in the laboratory but as yet have not been found suitable for field measurements.

SALT DAMP ATTACK

Mention was made at the beginning of this paper that the upward movement of rising damp can result in salts being deposited in the masonry. In some localities, particularly in parts of South Australia, damage to masonry caused by salt damp attack can be a serious problem. Typical signs of damage caused by salt damp attack are disruption of render or plaster, fretting of exposed masonry and exfoliation of stone. Sometimes differential deterioration takes place. Some clay bricks or stone blocks are badly weathered while others nearby are unaffected. In the case of clay bricks this differential attack may be the result of large variations in the firing temperature of the kiln which affect both the mechanical strength and the porosity of the bricks. For example sandstock bricks were usually heated to much lower temperatures than modern bricks. As a result they were often underburnt and some of them offered poor resistance to salt damp attack. Differences in the resistance of sandstone to this attack could result from a number of causes. Stone taken from different parts of a quarry can vary markedly in durability because of differences in the composition and structure of the cementitious matrix, the pore size and in the orientation of the bedding planes.

Salt damp attack can be caused partly by expansive forces associated with salt crystallisation, by chemical changes and expansion that occur in certain clay minerals as a result of their reaction with the contaminating salts or by a combination of both actions. Chemical analyses of samples of render, brick and stone taken from buildings in the Sydney area have shown that the salts introduced into the masonry by rising damp consist mainly of chlorides. Sometimes smaller amounts of sulphates are also present

The distribution of salts in damp sandstone walls in a building subjected to severe salt damp attack is shown in Fig. 3. Analyses of drillings taken from the walls showed that the maximum salt concentration occurred in a zone between 1 m and 1.5 m above the ground and this is where most of the deterioration of the stone took place. The graphs also show that in the horizontal direction at distances in excess of 25 mm from the wall surfaces there was a substantial reduction in the salt content. The interaction between upward capillary movement of moisture and surface evaporation resulted in a large proportion of the salts being deposited close to the wall surfaces where crystallisation and reaction with the clay minerals led to disruption of the stone.

In this building the rate of deterioration was greatly accelerated following the insertion of a new metallic damp-proof course in the walls to overcome a rising damp problem. The drying out of the walls lead to increased salt attack and accelerated fretting of the stone. Below the level of the new damp-proof course the stone still remained damp and only limited fretting took place. Even if the insertion of a new damp-proof course is effective in preventing upward movement of moisture, the presence of hygroscopic salts close to the wall surfaces can result in moisture being absorbed directly from the atmosphere. Changes in temperature and humidity can result in the salts going through cycles of solution and crystallisation. The dampness may then persist and also fretting of the masonry may continue.

If this situation occurs in internal walls, sometimes the insertion of a new damp-proof course and the removal and replacement of salt charged render or plaster may overcome the problem. But where severe salt contamination is present it may be necessary to reduce the salt content by soaking the walls with fine water sprays for a number of days and then drying the walls with fans and heaters to draw the salts to the surface. The wetting and drying process will have to be repeated a number of times until successive analyses of drillings taken from walls show that the

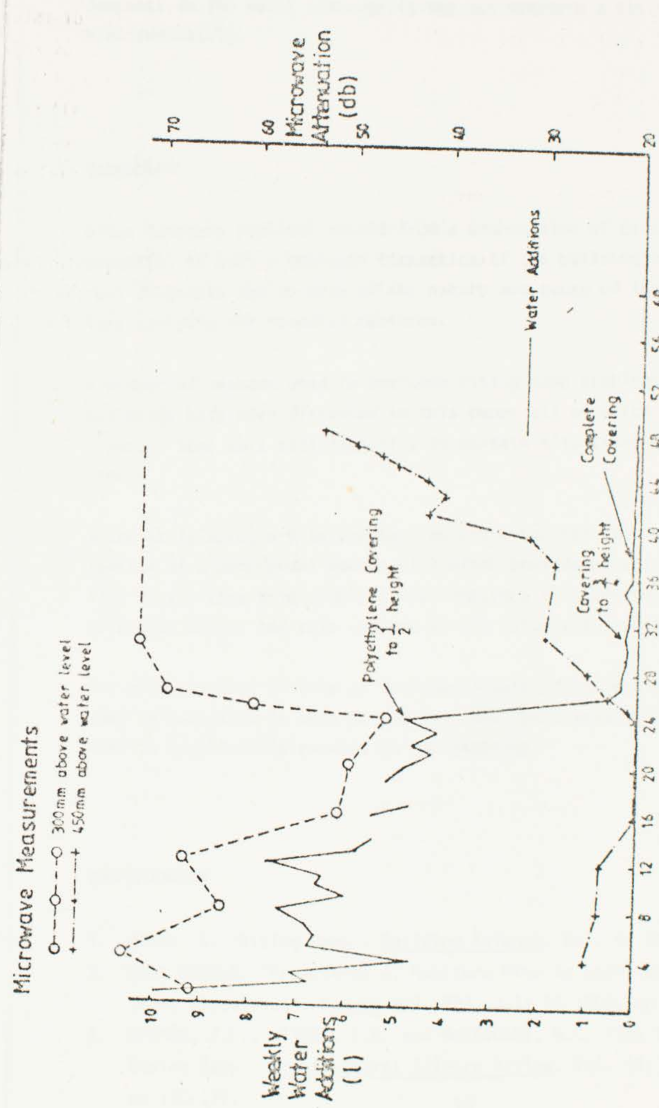


Fig 2 Influence of Polyethylene Covering on Moisture State of Wall

salt content has been reduced sufficiently. In a variation of this treatment the walls are first wetted thoroughly and are then covered with a poultice of absorbent clay or diatomoceous earth for at least a month(4). As the walls dry out it is hoped that the salts will be transferred into the poultice. It is likely that this treatment will also have to be repeated a number of times to obtain a beneficial reduction in the salt content.

In another method that has been used overseas for removing chlorides from masonry, an electric current is passed through the wall and the chlorides migrate to a metallic plate where they are removed by means of an ion-exchange resin.

Where prolonged and severe salt damp attack has taken place it may be necessary to replace badly fretted brick or stonework. Often the choice of new masonry is made solely on visual considerations and the need to match the existing masonry as closely as possible. There have been instances where new stone was even less durable than the original material. Within a few years sufficient aggressive salts migrated from the adjacent old stone into the new material to initiate renewed deterioration and fretting took place at a faster rate than before. In situations of this kind it is desirable to seek the advice of a geologist or brick technologist when selecting suitable stone or bricks.

Continued fretting of exposed brick or stonework caused by salt damp attack has sometimes been prevented by covering the damaged walls with a sacrificial render coat. The render must be sufficiently porous to allow the walls to breathe and evaporation of moisture results in salts being deposited in the render rather than in the masonry. A 1 : 1 : 6 or 1 : 2 : 9 mix (by volume) is used for this purpose. The render will slowly deteriorate and will eventually have to be replaced but the masonry will be protected from attack.

Where the walls were previously coated with relatively impervious paint or hard plaster, this treatment may also help to reduce the level of

dampness in the walls although it may not overcome a rising damp problem completely.

SUMMARY

Often dampness problems result from a combination of causes and it is essential to make a thorough inspection of the building so that a correct diagnosis can be made of the nature and cause of the trouble, before carrying out remedial measures.

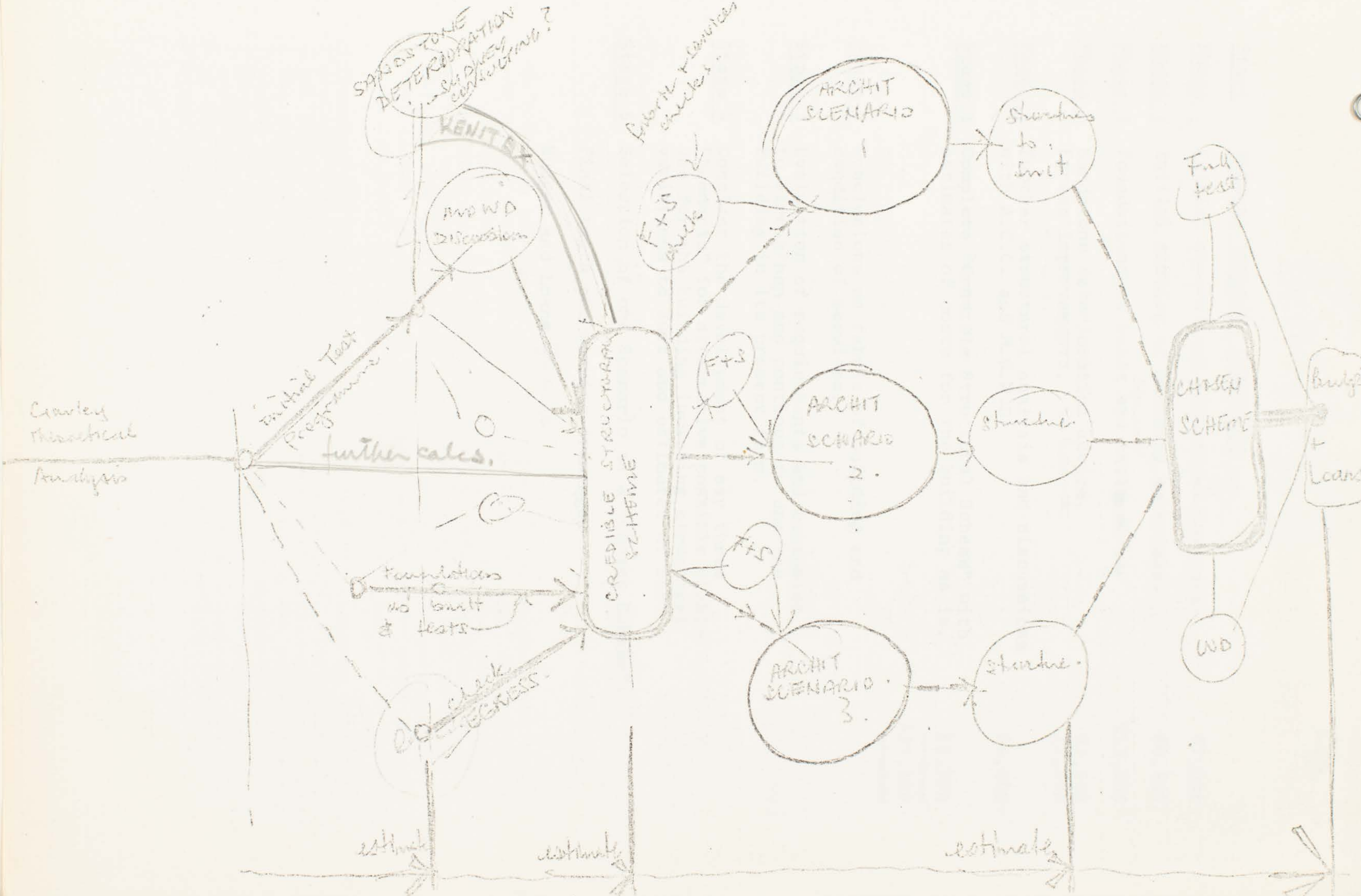
A number of methods used to overcome rising damp problems in masonry buildings have been discussed in this paper all of which have limitations. Some work satisfactorily in certain situations but not in others.

Where it is necessary to create a moisture barrier in the wall, the insertion of a damp-proof course will often provide a satisfactory solution to a rising damp problem, but sometimes this treatment will be ineffective unless the salt content of the masonry is reduced.

The effectiveness of some of the proprietary treatments at present being used in Australia is open to question and could involve the prospective user in considerable expense to no advantage.

REFERENCES

1. MASON, G. 'Rising Damp'. Building Science, Vol. 9. 1974. pp 227-231.
2. GRAY DONALD. 'Prevention of Moisture Rise in Capillary Systems by Short Circuiting'. Nature Vol. 223. July 26 1969. pp 371-374.
3. HEIMAN, J.L., WATERS, E.H. and McTAGGART, R.C. 'The Treatment of Rising Damp'. Architectural Science Review. Vol. 16. December 1973. pp 170-177.



SANDSTONE DETERIORATION
SYDNEY CONVINCING ?
KENITEX
Fabric & services checks
ARCHIT SCENARIO 1
Structure to suit
Full test
AND WD DISCUSSION
F+S check
Initial Test Programme
Crawley Theoretical Analysis
further calcs.
CREDIBLE STRUCTURAL SCHEME
F+S
ARCHIT SCENARIO 2.
Structure
CHOSEN SCHEME
Budget + Loans
Foundations
WD built & tests
F+S
Check PROGRESS
ARCHIT SCENARIO 3.
Structure
WD
estimate
estimate
estimate

C/c

Start	PRESENT BUILDING CONDITION	
Stage 1 Phase 1	Initial theoretical structural analysis.	$7,000
Phase 2	Initial testing of building materials.	$6,500
Phase 3	Foundations drawings - as built and tests drilling	$13,000
Phase 4	Sandstone deterioration advice.	$2,500
	Egress Improvements. scheme	$2,000
Phase 5	Further structural analysis and discussions with A.C.C. and M.O.W.D.	$5,000
Stage 1	Complete "Credible Structural Scheme" with estimates of costs for the building as is.	$2,500
	BUDGET No 1	$37,500

Phase 6 Conclusions on fabric of building and condition of services.

Stage 3 Evaluation of requirements and economics to strengthen and continue to use the building in its present form. BUDGET No 2

Stage 4 Consider the development of say three "Scenario" for alternative possible viable use for the building including structural variations to suit and estimates of cost.

Stage 5 Selection of one Scenario as a "Chosen Scheme". BUDGET No 3

Final tests and working drawings.

Budget and Loans. Finance.

GURLEY & NICHOLLS
CIVIL AND STRUCTURAL ENGINEERS
40 College Hill, Auckland, New Zealand PO Box 47-216, Ponsonby Ph 767-152 767-432 760-772

22 December, 1981.

The Chief Engineer
Auckland Harbour Board
PO Box 1259
AUCKLAND

ATTENTION: Mr. B. Le Clerc

Dear Sir,

re: FERRY BUILDING

We here confirm an issue by hand last week of one copy of the Sandstone Report by Dr. G.S. Gibbons. Copies of this document, with the drawing attachments have been made available to Architect, Mr. T. Dixon and Quantity Surveyor, Mr. A. Dickenson.

We here enclose for your information :

One copy of the A.R.O. (University of Auckland) report on Seismic Wall Pressures (November, 1981) by Dr. T.M. Larkin.

Overall costing information by Hallam-Eames & Partners, incorporating both the above report, and

Notes from ourselves on suggested construction recommendations.

We consider the report by Dr. Gibbons to be of excellent value and he has identified and tried to quantify many problem areas. As he mentions, it is not unusual for a restored stone building to continue to show deterioration and even deteriorate at a greater rate and hence our reliance on Dr. Gibbons to correctly analyse the existing deterioration causes is of paramount importance. Dr. Gibbons has listed a formidable range of required works but individually the majority are not so daunting.

Ensuring the building has good drainage and properly functioning windows, flashings and fabric (e.g. structural steel beams in the tower) is work well within normally available skills. Also, the works involved with removing opportunity for efflorescence, pigeon access and water penetration. Even though the extent of stonework repair can still not be accurately assessed, it is encouraging that at this stage, Dr. Gibbons considers the bulk of the work will be redressing, sweetening and only the odd stone requiring replacement.

BLC.

.... 2/

Auckland Harbour Board 22 December, 1981

The Larkin report on the likely liquefaction problems is encouraging and we feel is as far as we need to go at this stage. As the author summarises on page 7, the cost of further analysis would be high and the feedback doubtful. From this work, we have the confidence to design the wall as a dam to resist the pressures from the liquefied soils and this loading was covered by the presentation in our Section 11 (Foundations) and our previous costing estimates.

We have enclosed some suggested construction recommendations, but we consider them preliminary and in outline only. When the direction of the project is more specific, we would like the opportunity to rethink and extend these comments.

Yours faithfully,
GURLEY & NICHOLLS

J S Nicholls

J S Nicholls

Enc.

1.0 PRELIMINARY WORK:

With New Zealand fixed into an inflationary spiral, the earliest project start is the best, and delay in spending any planned monies can only mean an increase in the funds expended. However, it will be difficult to avoid financial and physical restraints that must effect this project.

We do not wish to comment on the problems of loan monies, nor at this stage, the effects likely of the Ministry of Works and Development being involved in the project, however we see at this stage, several areas of research and/or feasibility studies that will need to be woven into any job programme. For example :

1. Kenitex removal from stonework and stone cleaning - use of water and/or solvents.
2. Replacement stone - accelerated weathering tests and/or durability tests.
3. Brickwork drilling and brickwork grouting.
4. Internal plastering and finishing. Examine types and cost of alternatives.
5. Masonry, external jointing and pointing. All sandstone, basalt and brickwork.
6. Final finishing - brickwork and sandstone - all to reduce the visual and long term effects of the plugging of reinforcing steel and patching of damaged fabric.

Dr. Gibbons gives several options with regard to Items 1 and 2 and both these activities are major problem areas that will, no doubt, never give a total answer. If an approach can be found that will reduce the immediate problems that may be the best solution achievable, consequently, the unsolvable parts of these problems will then need to be built into the total programme.

Item 3 could commence any time when a suitable building or brick assemblage is discovered to be available. Longyear Drilling Co., have indicated a willingness to set up some trial drilling and on 5 January, 1982, Mr. Malcolm Brain, ex Fondedile (UK) Works Manager (now resident in Fiji), will be briefly in Auckland. A trial drilling operation, as above, with Harbour Board personnel and Mr. Brain and ourselves present would give valuable feedback to our schemes and plans. Whether this proceeds in 1982 or 1983, it looks to be essential research which may need to be arranged rapidly when a suitable venue comes available. We can arrange to meet Mr. Brain in the new year and mention to him the possibility of such an operation occurring.

Items 5 and 6 presumably will not be finalised until Items 1 and 2 are complete, but at that stage, further recommendations will need to be sought from Dr. Gibbons and it could be opportune to set up sample panels on the building and see the weathering and inter-seasonal effects at least over the reconstruction period. While the building is scaffolded during this period, access will be readily available to all parts of the building fabric. Never again will there be such an opportunity. Maybe also, at this stage, any varying or alternative recommendations planned for the maintenance schedule could be compared and examined.

Many sections of work should ideally only go ahead when everyone involved has been able to develop a complete familiarity with the fabric as existing (and as desired at completion) and this feedback can only come from this physical research programme and a continuing literature search.

2.0 THE CONTRACT:

Procedures normally used for commercial building projects are not the type that would encourage the 'special' project atmosphere that we feel is warranted here. We consider any construction work on the Ferry Building must be considered as restoration of an 'antique' and using the word antique with all its emotional connotations. Working on antique objects of any type requires enthusiasm, dedication and patience and a determination to make the finished product as perfect as possible. Whether the inevitable commercial pressures will allow the Harbour Board to see the project in this light, we do not know.

We certainly anticipate difficulties in assembling a meaningful set of documents that would allow normal tendering and letting of lump sum or even charge-up contracts. Considering especially the range and implications of the research and feasibility studies discussed in Section 1.0, we feel provision must be made to keep individual work packages modest in size so that advantage can be taken of feedback from any ongoing studies.

We suggest a project manager should be appointed to have full control over the project. He would be on the Board's staff, but independant of other Board projects and dedicated only to the Ferry Building. He would report to the Chief Engineer and his staff and would have direct links to the Construction Engineer and his staff. We see it as useful and appropriate that many of the smaller jobs could be carried out by the Construction Division and that whole department would have an involvement in the reconstruction.

The Project Manager would cross relate and maintain discourse with the technical Consultants and he would have a staff

Project Manager - where from?
- 3yr contract?

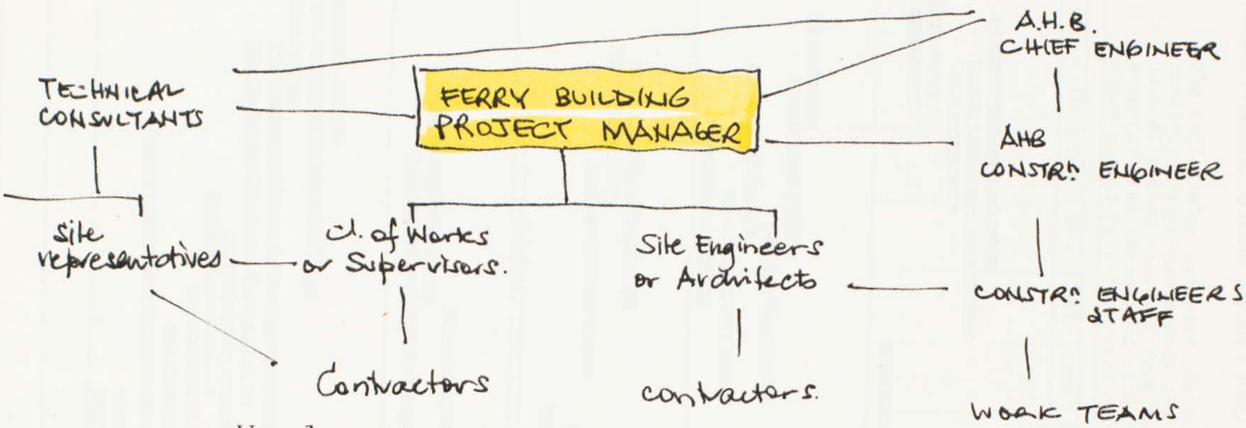
Staff for P.M. - extent.

Preparation of Contract Documents?

of supervisors - Architects, Engineers and Clerk of Works who would have direct control over the onsite work teams or contractors.

Using this method :

* Contractural pressures are minimised,
* No work need proceed until all the correct information is available and the timing is right,
* All supervision is by the one team and this group is under the direct control of the Board.



We also attach here the latest estimate by HERA of the major projects planned in New Zealand over the next ten years. There has been suggestions made that these works would effect the availability of a work force for the Ferry Building. Discussing this point with a local contractor and a group that does much work in the Queen Street valley, they do not anticipate these projects affecting their own established teams. We feel however, it must affect the choice of supervisory personnel and the speed with which some specialist groups will perform on the job.

GURLEY & NICHOLLS

HERA

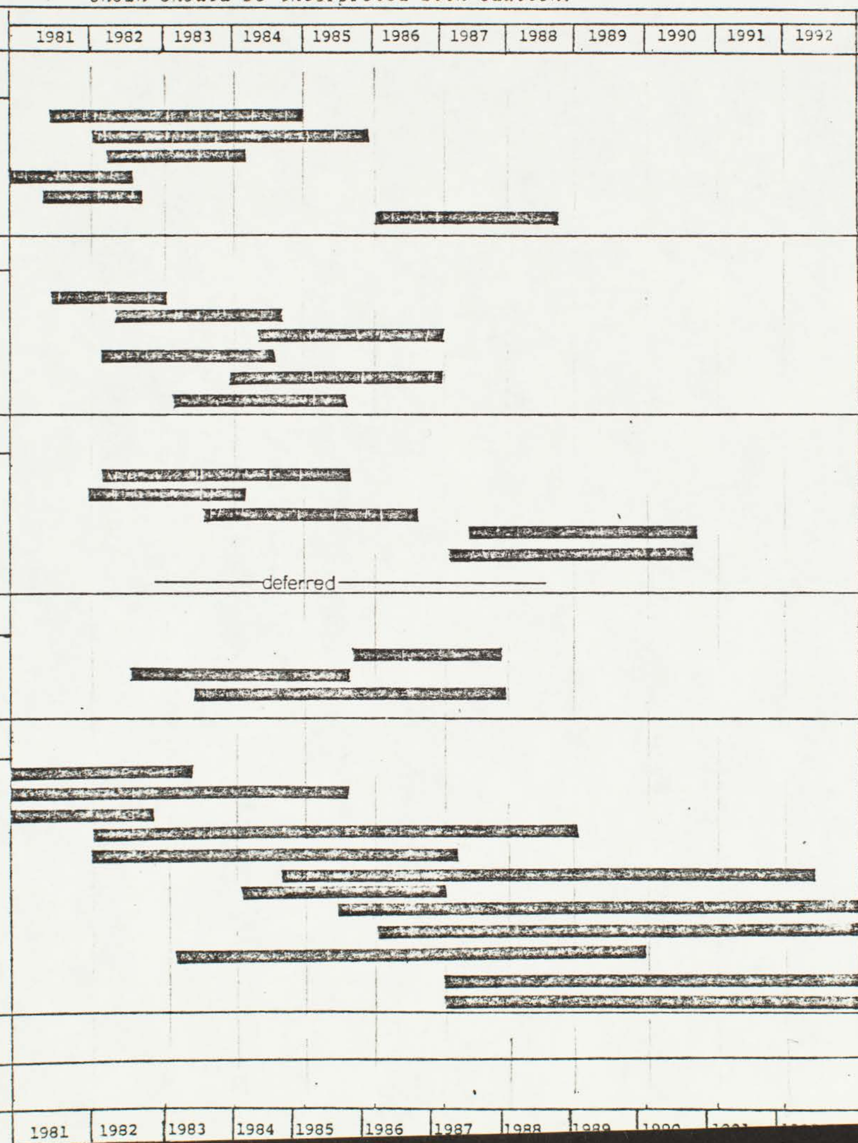
NO. 5
OCT. 81.

MAJOR PROJECTS (over $50m) AND THEIR TIMING

The table below sets out the broad timing of planned and proposed major construction projects over the next 10 years. The information shown has generally been obtained from published sources. There are obvious uncertainties associated with all projects and for this reason the information shown should be interpreted with caution.

Planned Start/ Finish

PROJECT	SPONSOR
PETROCHEMICAL	
Marsden Refinery Expansion	N Z Refining Company
Synthetic Petrol	N Z Synthetic Fuels Corp
Chemical Methanol	Petralgas Chem. N Z
Ammonia Urea	Petrochem Corp. of N Z
LPG Distribution	Liquigas
Ethane Extraction	Undetermined
BASE METAL	
N Z A S 3rd Potline	NZ Aluminium Smelters
2nd Aluminium Smelter I	South Pacific Aluminium
II	South Pacific Aluminium
N Z Steel Expansion I	NZ Steel Development
II	NZ Steel Development
Silicon Carbide	Ceramco
PULP & PAPER	
Kawerau 4	Fletcher/Challenge
Nelson Pulp Mill	CSR/Baigent
Marsden Pulp/Paper	NZFP
Whirinaki Newsprint	Carter/Oji Kokusaku
Northern Pulp	Northern Pulp
Otago Pulp	South Wood
IMPROVEMENTS TO TRANSPORT	
Northland Forestry Port-	Northland Harbour Board
Extension to Port Taranaki	Taranaki Harbour Board
North Island Rail Electrification	NZ Railways
POWER PROJECTS	
Ohau B	Ministry of Energy
C	"
Rangipo	"
Clyde	"
Ohaki	"
Ngawha Springs	"
Marsden B Conversion	"
Queensbury	"
NI Thermal	"
Luggate	"
Kawerau Gorge	"
Lower Waitaki 1 & 2	"
CEMENT WORKS	
Oamaru	NZ Cement Holdings



Auckland HarbourBoard

COLIN GURLEY
14 CLIFTON ST.,
BALMAIN, 2041
AUSTRALIA

PH 818-3824

J.G. 14

→ Correspondence.

→ Structural calculations
Sect 1 – 12

→ Miscellaneous.

1. Bricks.
2. Fire + Egress.
3. Foundations.
4. Sandstone – Gibbons.

1

Ferry Buildings – upgrading for earthquake requirements
Visit of Spencer Nichols to English branch of Fondedile Ltd. Oct 1981.

Approach taken in talks in England.

1. Assess drilling information of Gurley and Nicholls scheme.

2. Show Fondedile comments on technicalities of Gurley and Nicholls scheme.

3. Show Fondedile method or approach.

4. Comments re Fondedile and Earthquakes.

5. How will contract go.

Copy to Staff Engr
for info

RBC
5/11/81

WHAT FONDEDILE HAS:

1. Substantial historical and technical back-up from Italy.

2. Much experience in providing a complete scheme or service - i.e. design and construction.

   They see themselves as specialists, and specialists with an extensive or well developed philosophy.

3. Much experience in detailing their steel for maximum effectiveness.

4. They consider the geotechnical side and the superstructure work go hand in hand. They are essential bedfellows. One side is rarely needed without an injection from the other.

5. They have much experience in the techniques of drilling in buildings.

   They appear to have great confidence in what they can 'get away with' in a building.

   They know exactly what their rigs can do and modify units continiously to improve performances. All their rigs do considerably more than their manufacturers originally intended.

6. An ex-works manager in Fiji.

7. A willingness to act as specialist consultants for a fee - e.g. provide some manhour/productivity information.

8. Are willing (for us) to price (in UK figures) an outline scheme.

AUCKLAND FERRY BUILDING 6 November, 1981

Report by : GEORGE GIBBONS

I have been having a look around the First Floor cornice and the North-West annex of the building. As far as the North-West annex is concerned there is nothing remarkable about the weathering either on the coping or cornice around that section; probably the whole area will need rubbing back similar to that on the main building. There is some damage visible on the main cornice of the main building which is just above the roof of that annex. The annex roof itself drains to a central downpipe which looks to be in good condition. On that annex and also on the main building as far as the First Floor cornice is concerned, the stone is bedded horizontally, which is different from what was aksed for in the specifications, but it is consistent with the practise in Sydney.

On the West side it is notable that where there is a stone sill under one of the windows - that is the central window, there seems to have been less water going in and as a result less damage in the archway underneath. On the other hand, above the centre door there is more damage which seems to be related to the detail on top of the cornice. It interacts with the rustication there just underneath the arch above the door and as a result, when you get down in the doorway itself, there seems to be more damage.

It's not clear what the cause is of the bad deterioration around the toilets. It is probably related to water getting in through windows and also underneath the timber sills. This seems general that where there is that stone sill things are a lot better than where there is the timber sills because the water is running under that, there being no flashing there. There is also an overflow pipe from a storage tank inside the toilets. It may be that that overflow pipe has let in a lot of water into that section at some time in the past, although it is not showing any sign of recent flow. It is also possible that some of the damage underneath the ledges may have been caused in the past by pigeons having been nesting on these ledges, although there is no sign of them now.

It appears there used to be pigeons nesting around the building before around 1970 and that could have contributed to damage in the past.

As far as the South side of the building is concerned, there seems to be general contour spalling on the outer edge of the First Floor cornice and it is likely that this will have to be chipped back for 15mm or so all the way along. There is likely to be a lot of similar spalling in the moulding underneath even though the Kenitex seems to be holding it well in place, but it is likely that there will have to be very extensive cutting back along there and that will probably result in a contour quite different from what there is at present and to retain that contour will need a lot of infill pieces, possibly continously all the way along, which will be fairly expensive.

As far as the hood is concerned above the main entrance, that is leaded on the top, but it is not flashed into the building and it drains back towards the main wall so that the water is, in fact, taken back up against the main wall and into the main wall. There is a gutter near the centre of the hood and that should protect the area above the door itself, but the drain hole from that was about nine-tenths blocked when I was there with material fallen from above, so that was not operational and there was probably water getting back into the main structure there as well which is probably contributing to the rusting of the RSJ set back in the wall which you can see from the staircase. There is also cracking near the centre of the lead.

On the West face, the top cornice, although it is lead capped is showing quite bad deterioration compared with the other cornices on that facade. It is possible that some of the lead has lifted with the wind in that area, or alternately, it may be that the flashing is inadequate particularly up against the windows in the facade. The fixing of the ironwork at that level might also be checked because it does appear that water is getting in at that level in some way.

On the South facade there are three pediments. The ones to the East and West don't seem to have any more problems than on the North side generally, except right at the extreme East end of the Eastern pediment, but that's probably just a local statistical variation. However, the centre pediment, which is associated with the tar, does show damage on the main cornice, particularly where the joints come through. This may possibly be related to the tower wall or to a steel bar used as a support there but it is also possible that water is getting in behind the sill to the arch in that centre window. The lead seems to have operated properly all along the rest of this facade, so it is probable that it is in the un-leaded area near the window that the problem exists at that level.

Along the lower cornice, which is the one that shows the general contour spalling, there is a good deal of evidence that the joints are leading water through, mainly underneath the windows and it is probable that this is because driving rain is getting in underneath the timber sills and then coming down in that area. It is particularly the joints under the windows where the deterioration is most visible. That may not be particularly relevant in terms of the problem there which is the contour spalling, but it certainly is something that needs attention when the contour spalling is fixed up. These sills need to be properly flashed and sealed.

Above the main hood mould there is an air conditioner. I think that that is not currently contributing to the problem, I think it is related to the lack of flashing behind the main hood there, but certainly the air conditioner is a potential problem and when it is fixed up, the position of that and the possibility of that leaking water into the structure needs to be checked.

Comments on the East facade are generally identical to those relative for the West facade, particularly there seems to be some deterioration of the drip line, particularly underneath where the iron ornamentation is that need to be checked as far as the flashing there is concerned and it also needs a double check on the state of the medallions and the other parts of that main cornice on that level.

Direct tape transcript.

Applied Research Office
The University of Auckland

aro

The University of Auckland, Private Bag, Auckland. Telephone 792-300 Ext. 328 or 330.

Proposal for the Assessment of Seismic Loadings

on the Sea Wall of the Auckland Ferry Building

1. Aim: To carry out an evaluation of the magnitude of the earthquake induced lateral loading on the sea wall from the foundation soils.

2. Scope of Work: The amplitude of the design earthquake used will be approximately that suggested in the preliminary geotechnical investigation carried out by Brickell Moss and Partners.

   Very little data is available on recorded pressures on walls during earthquakes of engineering significance. The work will use an approach based on a simple yet reasonable model to compute the seismic response of soils. This information will be used to estimate the loads created by the confining effect of the wall. A limited amount of computer work will be carried out.

   The question of seismic liquefaction of the soft silts and sands beneath the building will be of some concern in this work. Engineering jugdement will be exercised in regard to this question. An attempt will be made to gauge the effect on the loading on the wall of significant strength loss in the subsoils. The very heterogenous nature of the materials makes it very difficult to assess the liquefaction potential in the fill materials. This work will not directly evaluate the liquefaction potential but will investigate the seismic loading on the wall using a range of soil properties to model varying degrees of strength loss caused by liquefaction of the fill material.

3. Estimate of costs:

Professional time	- 14 hrs @ $35 per hour	$490
Computer costs	-	$60
		$550

TO: CHIEF ENGINEER AHB: 02/10/81
As discussed by phone
I have instructed
Larkin to proceed.
Draft report by end
this month!
Colin Gurley

T.J. Larkin,
2nd October 1981

Hallam-Eames & Partners
Quantity Surveyors – Building Economists

ALAN Q. DICKINSON ANZIQS AAIQS

Mr Le Clerc

AUCKLAND PO Box 5497 Phone 31-639 796-042 | WELLINGTON PO Box 2496 Phone 728-632 | DUNEDIN PO Box 1449 Phone 776-369 | INVERCARGILL PO Box 1449 Phone 86-544

[H]AMES & PARTNERS

[QUA]NTITY SURVEYORS

[BU]ILDINGS [AU]CKLAND NZ | 29 SHORTLAND STREET TELEPHONE 31-639, 796-042

2 October 1981.

Recd. 12/10/81
3pm.
by hand. Bec

Messrs. Gurley and Nicholls,
Consulting Engineers,
P.O. Box 47-215,
Ponsonby,
AUCKLAND.

Dear Sirs,

PROPOSED UPGRADING AUCKLAND FERRY BUILDING - BUDGETS

CONSTRUCTION BUDGET

As requested we have prepared global budget estimates for the items excluded from our report of 4 September 1981.

We set out in that report estimates of cost for Superstructure Structural Upgrading and as outlined, those costs will depend upon actual drilling rates, and further research and trial drilling will clarify this area.

This report also contained a list of exclusions and we have now prepared global budget estimates for these items.

Our budget estimates contain allowances for carrying out basic interior and exterior restoration and upgrading work which, in our opinion, will be necessary. We have allowed merely to restore the building, in its present configuration of offices and amenities, but to a modern standard of amenity. No allowance has been made for providing services for uses other than offices on the first, second or third floors.

As the likely use of the interior of the building is unknown at this stage, and as reports on the extent of remedial work required to the exterior facade have yet to be made, the majority of our estimates are in fact budgets rather than estimates of anticipated work. These budgets are however, based on allowances for similar projects.

The stone restoration work to the exterior facade is an area, where the cost is greatly depended on the advice of the overseas consultant. If sections of the stone have to be cut out and replaced, the cost could well rise above our budget allowance. We have at present allowed to remove the existing sprayed application and then allowed a rate per square metre to cover cutting out and patching the stone in large, but isolated areas.

Our estimates are based on all of the Work being under the control of one single contractor who would charge a profit margin on the total value of the Work. However, thought could be given to letting separate contracts for

.../2

HALLAM-EAMES & PARTNERS

QUANTITY SURVEYORS

8TH FLOOR GENERAL BUILDINGS 29 SHORTLAND STREET
P O BOX 5497 AUCKLAND NZ TELEPHONE 31-639, 796-042

2 October 1981.

Recd. 12/10/81
3pm.
by hand. BGC

Messrs. Gurley and Nicholls,
Consulting Engineers,
P.O. Box 47-215,
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Our budget estimates contain allowances for carrying out basic interior and exterior restoration and upgrading work which, in our opinion, will be necessary. We have allowed merely to restore the building, in its present configuration of offices and amenities, but to a modern standard of amenity. No allowance has been made for providing services for uses other than offices on the first, second or third floors.

As the likely use of the interior of the building is unknown at this stage, and as reports on the extent of remedial work required to the exterior facade have yet to be made, the majority of our estimates are in fact budgets rather than estimates of anticipated work. These budgets are however, based on allowances for similar projects.

The stone restoration work to the exterior facade is an area, where the cost is greatly depended on the advice of the overseas consultant. If sections of the stone have to be cut out and replaced, the cost could well rise above our budget allowance. We have at present allowed to remove the existing sprayed application and then allowed a rate per square metre to cover cutting out and patching the stone in large, but isolated areas.

Our estimates are based on all of the Work being under the control of one single contractor who would charge a profit margin on the total value of the Work. However, thought could be given to letting separate contracts for

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sub-structure strengthening, superstructure strengthening, and non-structural upgrading. If this method was adopted, the total project would need to be very tightly co-ordinated.

SUMMARY OF ESTIMATES

1. Superstructure Structural Upgrading Estimate

Refer to our report of 4 September 1981. We would recommend that a conservative estimate be used based on drilling say two 2 metre long holes per day — $3,624,093.00

2. Sub-structure Structural Upgrading Budget. (Other than new ground floor). — $ 400,000.00

3. Non-structural Upgrading Budgets

(a)	Internal Demolition	$ 30,000.00	
(b)	Carpentry	150,000.00	
(c)	Stairs - upgrade existing	10,000.00	
(c)	External Windows - restore, reglaze as necessary and repaint	51,000.00	
(d)	Lift single car and shaft	95,000.00	
(e)	Floor coverings - tiles, carpets and vinyl	125,000.00	
(f)	Electrical	90,000.00	
(g)	Plumbing	45,000.00	
(h)	Drainage	8,000.00	
(i)	Internal plastering and making good, internal painting	190,000.00	
(j)	External Walls, restoration and decoration	140,000.00	
(k)	Roof- repairs to existing and gutters	20,000.00	
(l)	Fire hose reels	5,000.00	
(m)	External Work - paving and planters	37,000.00	
(n)	Estimating Contingency	200,000.00	
(o)	Preliminaries - permits, supervision, plant, scaffolding, on-site administration	150,000.00	
(p)	Profit Margin and Overheads	154,000.00	
	BUDGET ESTIMATE	$1,500,000.00	$1,500,000.00

SUMMARY OF ESTIMATES CONTD.

4. Consultants Charges

While the engagement of Consultants may well be on an actual time and expense basis, it is, in our opinion, more realistic to establish a budget on a percentage fee basis at this stage. We have therefore allowed 12½% overall to cover preliminary investigation (including specialist investigation), pre-contract design and documentation and contract administration

Budget Allowance	$ 691,000.00
Construction Budget (excluding escalation) as at 30 September 1981	$6,215.093.00

ESCALATIONS

(a) Pre-contract

Our budget estimates are based on current costs. Provision would therefore need to be made for escalation both over the pre-construction and construction periods.

From our own research, escalations on this type of work have been running at approximately 18 to 20% per annum. It will depend therefore on the length of lead in time prior to construction as to what provision should be made for escalations prior to construction starting. We would consequently suggest that Mr Le Clerc allow for escalation in his budgets accordingly.

(b) Contract

In our opinion a two year construction period should be budgetted for. On current trends we could expect therefore, that over the construction period alone, costs will escalate some 40%. However, as construction proceeds, progress payments will be made and, as a result, the actual rate of escalation will decrease. Our research shows that the actual amount of escalation will usually equate to :-

Escalation = Contract Value x 20% p.a. x 2 years x 60% rate of expenditure x 80% rate of recovery

= 20% for consts.

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CONCLUSION

We would like to be given the opportunity of analysing specialist consultants' reports as they become available in order to advise you of the effect of such reports on the Budgets.

Our experience on these types of projects has been that the Budget needs to be continually reviewed to keep up with the evolving design criteria and changing ideas for possible building use.

Yours faithfully,

HALLAM-EAMES & PARTNERS.

2025/AQD/ads

WILLIAMSON PARTNERSHIP
DON BUNTING SPENCER NICHOLLS
ARCHITECTS & CONSULTING ENGINEERS
40 COLLEGE HILL, AUCKLAND, NEW ZEALAND
BOX 47-215 PONSONBY . TEL 767152, 767432

I S F NICHOLLS . BE, MICE, MNZIE, REG ENG

WARGON CHAPMAN AND GURLEY

REGISTERED ENGINEERS
CONSULTING STRUCTURAL & CIVIL ENGINEERS

P.O. BOX 47-215 AUCKLAND, N.Z. PHONE 760-772

CONFIDENTIAL

Our Ref : AK185

AUCKLAND

Dear Mr. Le Clerc,

re: FERRY BUILDING

We write to summarise the present situation.

SUPERSTRUCTURE STRENGTHENING:

Our current appreciation is contained in our main report up to and including Section 10 and in (A1 size) Drgs. Nos. GA1 to 5, D1 to 3, D7 to 10, handed to you at our meeting of Friday, 4th September.

In our view, it has now been established that it is technically possible to upgrade the Ferry Building to a base shear strength of the order of 0.2g with detailing standards which,at least, pay some attention to the understanding underlying modern standards.

There is considerable scope for refining these proposals in the light of architectural acceptability, construction practicability and cost.

However, now that the engineering design priorities and implications are fairly well appreciated, it does seem to be time for the wider historical/social/use implications to be weighed so as to provide a specific brief for more refined engineering work.

So far as construction technique is concerned, it is clear that cost is sensitive to site productivity in drilling masonry. We have discussed this aspect with Mr. Goord and with several local drilling companies so as to achieve some appreciation of local expertise. As agreed, Mr. Nicholls will be looking at European experience during October and returning to Auckland about 1 November.

MINISTRY OF WORKS & DEVELOPMENT POLICY

I have had a toll discussion with Mr. Trevor Mitchell (newly appointed) Assistant Chief Structural Engineer, Ministry of Works Head Office. MOWD are currently trying to formulate policy for major historic buildings (as distinct from the broad run of old buildings) and they are also interested in moves by the NZ National Society for Earthquake Engineering (NZNSEE) to initiate deliberations into the criteria for strengthening such buildings. Any NZNSEE Committee would include private sector representation, but the time-scale for a 'strengthening code' may be such that the Ferry Building and a few others provide the 'test-cases' which determine code policy.

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22/9/81

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Principal:
COLIN R. GURLEY, MEng. Sc., BE (Hons.), MNZIE, FIE Aust.
Consultants:
ALEXANDER WARGON, MSc, CE, MNZIE, FIE Aust, FASCE.
ROBERT F. CHAPMAN, BE, ASTC, MNZIE, FIE Aust.
JSF NICHOLLS, BE, MICE, MNZIE
Consultant

WARGON CHAPMAN AND GURLEY
REGISTERED ENGINEERS
CONSULTING STRUCTURAL & CIVIL ENGINEERS
P.O. BOX 47-215 AUCKLAND, N.Z. PHONE 760-772

18 September, 1981

The Chief Engineer
Auckland Harbour Board
PO Box 1259
AUCKLAND

CONFIDENTIAL

Our Ref : AK185

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re: FERRY BUILDING

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Mitchell initially thought that, where strengthening involves substantial public funds, then the scope of the work ought to aim at 'indefinite life', i.e. full and detailed compliance with current codes and philosophies. However, he does concede that :-

* There will be situations where (regardless of cost considerations) modern standards can not be achieved without so seriously detracting from the historic asset that the work does not make sense and that

* There will be situations in which the community is not prepared to go to the level of expenditure implied by the application of modern standards.

He seems to agree that the effectiveness of detailed strengthening work in tying the building together is a more significant and more important issue than the precise overall base shear strength achieved. He agrees that for very thick facades (500 to 1000mm) the provision of even a minimal percentage of basketting steel somewhere in the outer half of the facade would, at least, provide a major improvement even if the modern maximum spacing limits were eased, say, to values of the order of $1\frac{1}{2}$ to 2 metres. He was already aware that we had been considering prestressing and he seemed interested in following up that possibility. He points out that the effectiveness of prestressing anchor zone detailing may become the crucial issue that will determine spacing limitations. This has, of course, substantial cost implications.

In summary then, it seems that Mitchell accepts that the standards to be applied are, at least, open to some negotiation at a detailed technical level. It seems desirable to press for some documented confirmation on these policy attitudes as soon as your Board is able to declare its interest in funding the work by loans.

FOUNDATIONS:

Brickell Moss Report No. 56259 of 9th September, 1981 is enclosed and Section 11 of our own report will follow shortly. At a later time there will be some need to refine the geoseismic risk estimates to ensure that the level of risk accepted for foundation failure is consistent with the level of risk accepted for superstructure failure. The expertise required for such studies is sparse and the most likely sources are the Universities and/or MOWD.

The Brickell Moss Report concludes that '... liquefaction must be considered a real possibility under severe earthquake conditions'. There does not seem to be any way to improve and densify these materials in-situ. Our present report (Section 11) only contemplates strengthening (mostly) within the present confines of the Ferry Building and the preferred approach is to prestress the sea-wall down to rock to resist overturning. In this context the seismic forces generated below ground level are more significant than those generated in the superstructure. If liquefaction occurs over the full height of the sea-wall then the geoseismic pressures on the sea-wall will be those on a dam retaining a fluid which is (roughly) twice as dense as water. These forces are manageable within reasonable financial limits.

There is some suggestion however that a more serious situation might arise if liquefaction occurred only below, say, low tide level. If the upper soil mass does not liquefy but rides along on a lubricated plane over the lower (liquefied) mass then a more difficult situation arises. The only readily available estimates are those based on the passive resistance of the upper mass and these come close to the limits technically achievable by stressing the wall.

We suspect that these estimates may be too pessimistic and we are making some enquiries as to whether specialist dynamic studies may be able to provide reliable and usable results in this area.

If the strength achievable with the current approach does not seem acceptable, then other options or variations open to investigation are :-

* Remove the existing fills to form a battered bank behind the sea-wall down to low tide level.

* Supplement the sea-wall with some external buttress structures, for example, under the Ferry Tees near the North-West corner of the building and in the North-East corner under Queen's Wharf.

ARCHITECTURAL OVERVIEW:

We note that you have appointed Mr. Tom Dixon of Pepper and Dixon, Architects, to review our proposals in terms of their significance in an architectural/historical sense. We have had several meetings and some useful feedback from Mr. Dixon.

FACADE WEATHERING DAMAGE:

We note your instructions to arrange for specialist consultation with Dr. G.S. Gibbons of the New South Wales Institute of Technology. We are certainly impressed with the scope and expertise implied in his initial proposal. We are confirming arrangements for him to arrive in Auckland, Wednesday, 4th November and depart Monday, 9th November. He may need to work through the weekend and we will check on his requirements for safe access to the facade exterior.

FIRE AND EGRESS:

Report No. 2989 of 3 September, 1981 by McDonald Barnett Partners was handed to you on 4 September. It seems that this will only become a significant issue in overall budgetting if a substantial part of the building is to be used as a major restuarant, night club or cabaret. In the latter case, there will be both a statutory and a functional need to provide new facilities as well as upgrading existing. Presumably, the cost of these should be regarded as a charge against the particular use to be accounted for in the relevant commercial negotiations.

AUCKLAND CITY COUNCIL:

We will continue to keep the City Design Engineer (Mr. Leadbeater) informed of our structural proposals. We do not anticipate any major difference of opinion on structural matters. Some proposals (e.g. extension of ground floor slab, say, one metre onto the Quay Street footpath) will have town planning and/or legal significance to ACC. Please advise whether we are also to pursue these aspects.

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CONSTRUCTION BUDGET:

ALAN DICKINSON
31-639

Our Quantity Surveyors, Hallam-Eames and Partners, have reported estimates for structural strengthening of superstructure and these were handed to you on 4 September. These estimates exclude :-

* structural work below ground other than new ground floor slab
* restoration of weathering damage to facade
* restoration of interior fittings and architectural decor
* upgrading and/or renewal of building services, including lifts
* upgrading fire and egress provisions.

As instructed, Hallem-Eames and Partners are looking into these matters and we expect further information from them before the end of this month.

You will realise that :-

* The scope of facade restoration work will not be known until Dr. Gibbons reports in November.

* Several of these areas involve some consideration of the future use of the Ferry Building. See, for example, Paragraph 6 above.

TESTING OF BRICKWORK:

The second phase of testing at Auckland Engineering School is complete and a copy of A.R.O. report dated July, 1981 was handed to you on 4th September. Our main report, Section 6 refers. We consider that this has gone far enough for the moment although there may be a need for further testing (e.g. of prestress anchor zones in brickwork) at a later stage.

Section 6

Yours faithfully,
WARGON CHAPMAN & GURLEY

Colin Gurley

Colin Gurley

Principal:
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CONSULTANT

WARGON CHAPMAN AND GURLEY
REGISTERED ENGINEERS
CONSULTING STRUCTURAL & CIVIL ENGINEERS

P.O. BOX 47-215 AUCKLAND, N.Z. PHONE 760-772

18 September, 1981

The Chief Engineer
Auckland Harbour Board
PO Box 1259
AUCKLAND

CONFIDENTIAL

Our Ref : AK185

Dear Mr. Le Clerc,

re: FERRY BUILDINGS

You are aware that Mr. J.S. (Spencer) Nicholls has been assisting me with this work since late last year. Mr. Nicholls has been in private practice on his own account for ten years. He is a member of ACENZ and immediate past chairman of NZIE, Auckland Branch.

Mr. Nicholls will be joining this firm as a partner as from Thursday, 1 October and it will then be named 'Gurley & Nicholls'. Address and phone number will not be changed.

I am personally committed to spending a major proportion of my time in Sydney over, at least, the next six months. It is therefore appropriate that Mr. Nicholls take over the day-to-day running of this practice. I will, however, be available in Auckland for matters, including the Ferry Building, as and when you and he consider that appropriate. I have in mind that this might be for a period of two weeks, say, every few months but, in any case, it would be in response to the needs of the job. Such work would be carried on in our Auckland office in the usual way and charged out in accordance with the NZIE Scale relevant at the time.

Mr. Nicholls will be in Europe in October and, for part of that time, he will be looking at restoration of historic buildings there. I will be spending some time in Sydney in October and I will take the opportunity to look at some of the projects on which Dr. Gibbons has been involved. Sydney telex and telephone contacts will be supplied but you may find it convenient to use Mr. Alan Dickinson (Auckland phone 31-639) as an initial contact in this period.

Mr. Nicholls returns to Auckland on November 1st and I will be here for Dr. Gibbons arrival on November 4th and through to November 14th for the NZNSEE earthquake seminar at Auckland University. Thereafter I will return to Sydney and, inter alia, be available to review Dr. Gibbon's report at draft stage.

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NZIE, in an amendment dated 5 August, 1981, has increased the hourly charge out rate for private practitioners to the range \$40 - \$65. However, we intend to stand by the present agreed financial basis for Mr. Nicholl's investigations in Europe and we therefore propose to adopt a rate of \$46 as from November 1st.

We trust that these arrangements are satisfactory to you.

Yours faithfully,
WARGON CHAPMAN & GURLEY

Colin Gurley

Colin Gurley

HALLAM-EAMES & PARTNERS
QUANTITY SURVEYORS
Guardian Assurance Building
229 Queen Street, Auckland
P.O. Box ~~6683~~ 5491
PHONE 31-639

Attention: Mr Le Clerc

WITH COMPLIMENTS

Rcd 4/9/81
36C

PROPOSED STRUCTURAL UPGRADING - AUCKLAND FERRY BUILDING

As requested we have prepared a preliminary assessment of the cost of carrying out the proposed superstructure structural upgrading of the Auckland Ferry Building, as shown on your drawings D1, 2, 3, 9 and 10 and GA1, 2 and 3 and Section 7 of your Brief. Our estimates do however allow for strengthening the internal load bearing cross walls and corridor walls by using the 'Shotcrete' method rather than the drilling and concrete nib method.

The following items are not included in our estimates:

1. Foundation work. We have allowed for the new 300 mm thick Ground Floor slab but no allowance has been made for any work below this level.

2. Internal or external architectural work. We have not allowed for any architectural demolition, making good or upgrading nor for any new services. Plumbing and electrical services will be damaged during the structural upgrading and no allowance has been made to reinstate such services.

   Our estimates do include for flushing up and making good rebates, holes and chases and the like which are formed to facilitate the structural work but that is the extent of our allowances.

3. External waterproof sealing of masonary.

4. Upgrading of Fire Stairs, new sprinklers or thermal alarm systems.

5. Repairing or renewing roofing, gutters, downpipes and the like.

6. Timber floors. Where concrete floor beams are poured against the internal and external cross walls and corridor walls, an allowance has been made to make good the timber flooring. However in practice it may prove to be impossible to support

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HALLAM-EAMES & PARTNERS

QUANTITY SURVEYORS

8TH FLOOR GENERAL BUILDINGS 29 SHORTLAND STREET
P O BOX 5497 AUCKLAND NZ TELEPHONE 31-639, 796-042

4 September 1981

Messrs. Gurley and Nicholls,
Consulting Engineers,
P.O. Box 47-215,
Ponsonby,
AUCKLAND.

Dear Sirs,

PROPOSED STRUCTURAL UPGRADING - AUCKLAND FERRY BUILDING

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or retain the floor between the internal walls whilst this operation is proceeding. We have not allowed to replace the floors but merely to make good around the perimeter of each room.

7. New Lift

8. Consultants' charges

9. Cost fluctuations from 31 August 1981.

The likely rate of masonry drilling has a major influence on the total cost of the project. Research has indicated various likely drilling rates and in our opinion it would aid our costing work if trial drilling could be carried out on an existing Harbour Board building which is similar to the Ferry Building.

We have set out an estimate summary which shows the effects of various drilling rates per day.

As discussed, we are now proceeding to prepare global estimates for the items excluded above and we will report when these additional estimates are completed.

Summary of Superstructure Structural Upgrading

			TOTAL
1.	Total Estimate without any cost allowance for drilling masonry	-	$2,075,022
2.	Allowance for drilling masonry at the rate of five two metre long holes per day. ($44.00 per m)	$811,418	$2,886,440
3.	Allowance for drilling masonry at the rate of four two metre long holes per day ($50.00 per m)	$922,066	$2,997,088
4.	Allowance for drilling masonry at the rate of two two metre long holes per day ($84.00 per m)	$1,549,071	$3,624,093

5. Allowance for drilling masonry at the rate of one two metre long hole per day ($140.00 per m)	$2,581,788	$4,656,810

Note:

Allowances for drilling include for Preliminaries and Contractors Margins.

Yours faithfully,

HALLAM-EAMES & PARTNERS

MACDONALD BARNETT PARTNERS

REGISTERED PROFESSIONAL ENGINEERS
CONSULTING CIVIL & STRUCTURAL ENGINEERS
A. J. MACDONALD, B.E., F.N.Z.I.E., M.I.C.E.
C. R. BARNETT, B.E., F.N.Z.I.E., M.I.C.E.
M. B. SPICER, B.E., M.N.Z.I.E.

P.O. BOX 37-077
429 PARNELL ROAD,
PARNELL, 1,
AUCKLAND, N.Z.
TELEPHONE 795-670
TELEGRAMS: "MACBAR"

Wargon Chapman & Gurley,
Registered Engineers,
P.O. Box 47-215,
AUCKLAND

YOUR REFERENCE: OUR REFERENCE: 2989 MBS

3 Sep. 81

Dear Sirs,

RE: THE AUCKLAND HARBOUR BOARD FERRY BUILDING
FIRE RESISTANT CONSTRUCTION AND EGRESS ASPECTS

1. On 21st August you visited our office to discuss the redevelopment work that is being considered for the Auckland Harbour Board Ferry Building. Subsequently you requested that we advise you on "Fire Resistant Construction and Egress" aspects of the existing building, also to report on these aspects in relation to the possible redevelopment alternatives that you have been considering. Our Mr. Barnett and Spicer visited the site in company with the chief technical officer of the Auckland Fire Brigade. The purpose of this meeting was to appraise the current situation and to consult with the fire brigade controlling authority on various redevelopment proposals that were being considered. A verbal undertaking was obtained from them as to what would be required in terms of fire and egress upgrading to ensure compliance with NZS 1900 Chapter 5 and the fire brigade authority.

2. The building is located in the central fire risk area. This implies that it would need to meet the conditions of a "Type 2 construction" to fully comply with the requirements of Chapter 5. The floor plan area is approximately 935 m^2 (10,065 ft^2) and the base building comprises four storeys, above which rises the clock tower superstructure.

3. Commercial and industrial buildings such as office buildings, showrooms, shops for non-combustible and non-explosive materials are classified in the low risk division Group D1. A maximum floor plan area 1859 m^2 (20,000 ft^2) with unlimited storeys is allowed for this classification. General shops, restaurants, sale rooms, department stores, market buildings and workshops and workrooms manufacturing or storing materials for semi-hazardous processes are classified in the moderate risk division Group D2. In this classification a maximum floor area of 1394 m^2 (15,000 ft^2) with unlimited numbers of storeys is allowed.

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4. We understand that all of the main floors of the building consist of a 150 mm (6") reinforced concrete slab, which is overlaid in most instances by timber joists and decking to form the walking surfaces. A 150 mm thick concrete floor has a 3 hour fire resistance rating, assuming a minimum cover of 25 mm to the reinforcing steel. The maximum fire rating of a floor in a building classified as low risk Group D1 is 1½ hours and in a moderate risk Group D2 building is 2 hours. This 2 hour rating, however, may be reduced to 1½ hours where all walls, excepting fire walls are more than 15 feet from the legal boundary. We are not sure whether this situation applies in this instance, but nevertheless, the existing floors appear to have a healthy reserve of rating over and above the minimum requirement, notwithstanding the possibility that the cover to the reinforcing steel may in some instances be somewhat less than the minimum 25 mm requirement. No doubt there will be penetrations through the floors to accommodate the passage of services, ducts, etc. Some of these openings will need to be attended to, by way of provision of fire dampers for ducts, etc. and to ensure that only metal pipework or other suitable fire resistant materials pass through the floors to prevent fire bridging.

5. The main vertical supporting members primarily consist of brick and block masonry walls, together with masonry columns and other ornamentation. These members are all of massive section and of themselves quite readily provide more than the required minimum fire rating of 2 hours.

6. The main horizontal spanning elements other than the floors comprise massive steel beams which in the most instances are plaster or concrete encased. It has not been possible to ascertain the overall thickness of the encasing, but due to the general massive nature of the construction, one can assume that in general the horizontal elements will meet the 2 hour fire rating that is required for code compliance, although there may be odd isolated situations that will need upgrading.

7. The external perimeter walls have a relatively small proportion of openings in them and as the separation distances between other existing adjacent buildings are generous, the facade structure, infill panels, and window openings readily comply with the fire resistance ratings laid down in the code.

8. In summary thus, the floor areas and basic supporting fabric of the existing building comply with the current code ordinances. This being the case, any anticipated redevelopment automatically meets the main fundamental requirements for the fire risk area.

9. The existing building is currently occupied by a variety of tenants. Normally a 1 hour fire rated partition is required to separate adjacent tenancies and to separate any tenancy space and the routes and means of egress. It is noted that some of the existing corridors have plain glazing at high level, which is unacceptable and will need to be replaced by Georgian Wired Glass. Alternatively, the existing partitions could be extended in fire rated construction up to the underside of the floor above. It is also probable that a large proportion of the inter-tenancy partitions do not meet the minimum 1 hour rating and will need to be upgraded by the addition of further layers of Gibralter Board to ensure a full compliance. All doors giving access from individual tenancies in to the corridors would also need to be checked to ensure compliance with smoke stop standards.

Contd...over

10. In most instances there were no suspended ceilings in the building and thus no created ceiling spaces. However, in the roof space, ½ hour fire stops would need to be installed to provide areas less than 180 m^2, and with each stop no further apart than 15 m in any one direction. This could easily be effected by installing conventional 100 x 50 stud walls lined on each side with Gibralter Board. Alternatively, the third floor ceiling could be removed to expose the existing timber roof structure. This alternative has considerable merit for certain types of development such as restaurant or museum facilities.

11. The existing means of egress and widths of exit ways from the building are substandard in several aspects. The primary egress is by way of the corridors on each floor leading into the smoke protected lobbies which form part of the main central staircase. It was noted that the smoke stop doors, which separate the central stair lobby from the corridors either side, need to be upgraded to minimise the gap between the door itself and the enclosing frame. It is noted that these doors are currently swinging in both directions and upgrading could readily be effected by incorporating a planted timber stop restricting the door to one way opening in the direction of exit travel. The stairs themselves appear to conform in geometry and form of construction.

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c.c. File 2989

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YOUR REFERENCE:

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3 Sep. 81

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FIRE RESISTANT CONSTRUCTION AND EGRESS ASPECTS

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2. The building is located in the central fire risk area. This implies that it would need to meet the conditions of a "Type 2 construction" to fully comply with the requirements of Chapter 5. The floor plan area is approximately 935 m^2 (10,065 ft^2) and the base building comprises four storeys, above which rises the clock tower superstructure.

3. Commercial and industrial buildings such as office buildings, showrooms, shops for non-combustible and non-explosive materials are classified in the low risk division Group D1. A maximum floor plan area 1859 m^2 (20,000 ft^2) with unlimited storeys is allowed for this classification. General shops, restaurants, sale rooms, department stores, market buildings and workshops and workrooms manufacturing or storing materials for semi-hazardous processes are classified in the moderate risk division Group D2. In this classification a maximum floor area of 1394 m^2 (15,000 ft^2) with unlimited numbers of storeys is allowed.

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4. We understand that all of the main floors of the building consist of a 150 mm (6") reinforced concrete slab, which is overlaid in most instances by timber joists and decking to form the walking surfaces. A 150 mm thick concrete floor has a 3 hour fire resistance rating, assuming a minimum cover of 25 mm to the reinforcing steel. The maximum fire rating of a floor in a building classified as low risk Group D1 is 1½ hours and in a moderate risk Group D2 building is 2 hours. This 2 hour rating, however, may be reduced to 1½ hours where all walls, excepting fire walls are more than 15 feet from the legal boundary. We are not sure whether this situation applies in this instance, but nevertheless, the existing floors appear to have a healthy reserve of rating over and above the minimum requirement, notwithstanding the possibility that the cover to the reinforcing steel may in some instances be somewhat less than the minimum 25 mm requirement. No doubt there will be penetrations through the floors to accommodate the passage of services, ducts, etc. Some of these openings will need to be attended to, by way of provision of fire dampers for ducts, etc. and to ensure that only metal pipework or other suitable fire resistant materials pass through the floors to prevent fire bridging.

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6. The main horizontal spanning elements other than the floors comprise massive steel beams which in the most instances are plaster or concrete encased. It has not been possible to ascertain the overall thickness of the encasing, but due to the general massive nature of the construction, one can assume that in general the horizontal elements will meet the 2 hour fire rating that is required for code compliance, although there may be odd isolated situations that will need upgrading.

7. The external perimeter walls have a relatively small proportion of openings in them and as the separation distances between other existing adjacent buildings are generous, the facade structure, infill panels, and window openings readily comply with the fire resistance ratings laid down in the code.

8. In summary thus, the floor areas and basic supporting fabric of the existing building comply with the current code ordinances. This being the case, any anticipated redevelopment automatically meets the main fundamental requirements for the fire risk area.

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c.c. File 2989

31 August 1981 (Ref. 56259A.TXT,deh)

PRELIMINARY
SUBJECT TO REVISION

56259

31 August 1981

Messrs. Wargon, Chapman & Gurley,

Consulting Engineers,

40 College Hill,

AUCKLAND

Attention: Mr C.R. Gurley

Dear Sirs,

Re: PRELIMINARY SOILS INVESTIGATION,

FERRY BUILDING STRENGTHENING,

QUAY STREET, AUCKLAND.

For THE AUCKLAND HARBOUR BOARD.

Introduction

This report presents the results of our preliminary soils investigation, undertaken in conjunction with the proposal to strengthen the Auckland Harbour Board's Ferry Building. The Building is adjacent to Quay Street, Auckland, and bounded by the Waitemata Harbour, Ferry Wharf and Queens Wharf, as shown on the attached Plate 1, Site Plan.

Some subsurface information on this area had been obtained during our 1967 foundation investigation for Stage 1 of the Downtown Project (opposite the Building on the south side of Quay Street). That information was supplemented with research by Mr. Gurley and the results, including details of the existing foundations of the Ferry Building, are shown on Drawing SK 835, Sheet 1, dated Nov. 1980, and described in a Wargon, Chapman & Gurley report, dated December 1980.

The Building site is located on reclamation filling, placed in about 1905. The filling is retained along the north, east and part of the western sides of the site by a mass concrete seawall, as shown on the Site Plan.

The Building is apparently supported partly on concrete piles (driven down through the reclamation filling to the underlying rock), and partly on the seawall. The seawall was to be constructed directly on and keyed into the underlying rock. It was thought to be backed by a wedge of sandstone lumps (as shown on drawing SK 835), but whether this rock fill is present and if so how it relates to the piled construction is not known.

An earlier stage of reclamation filling was thought to be contained by a rock-filled timber breastwork, adjacent to the southern side of the Building. There may also be the remnants of a weighbridge along the eastern half of this southern side of the building, under the Quay Street footpath.

The purposes of our study were to provide some information on the reclamation filling underlying the Building, to check the condition of the mass concrete sea wall and its foundation, to confirm the depth to the underlying rock, and to provide preliminary geotechnical engineering design criteria for the proposed strengthening project.

Investigation and Testing

Subsurface conditions were explored by drilling three borings at the locations shown on the site plan. These positions were chosen to provide a cross-section across the site, and to correspond with accessible locations.

One of our engineers supervised the drilling on a full-time basis, logged the soils encountered, and obtained samples for examination and possible testing. Notes on the drilling and a summary of the abbreviations used on the boring logs are included on Plate 2, Notes and Abbreviations for Logs. Logs of the soils and rock encountered in the borings are presented on Plates 3-A to 3-C, Boring Logs. The soils are described in accordance with the Unified Method of Soil Classification, which is summarised on Plate 4, Method of Soil Classification.

Standard penetration tests were considered to be the most effective method to indicate the in-situ engineering properties of the reclamation filling materials. Where applicable, these tests were carried out at close intervals of depth in the borings, and the results are shown on the logs. The field values, shown on the logs, have also been corrected to allow for overburden and submergence values, with results as follows:

Boring	Depth	Field SPT	N, corrected
1	6.6	3	4
1	6.8	5	6
3	5.5	10	14
3	6.8	12	16
3	8.4	2	2

It was considered unnecessary to carry out extensive laboratory testing of the soil samples, due to the inherent variability of the reclamation materials. Particle size tests have been carried out on the sandy soils, from below the upper reclamation filling, and results are summarised on Plate 5, Grading Tests. Atterburg limit tests on two of these samples showed the fines to be non-plastic. Classification tests (liquid limit, plastic limit, and percent fines passing 75um sieve) were also carried out on a sample of soft clay from under the sandy soils in Boring 1, and results are shown on the boring log.

Subsurface Conditions

In making an assessment of subsurface conditions from a few borings, there is always a risk of undetected variations. In this case it is particularly important to remember that only three borings have been drilled, in part of an area where the reclamation methods and materials are known to have been extremely variable. However, the results obtained are generally consistent with available background data and the site geology, and are therefore described here as a basis for the conclusions of this preliminary report.

The site is blanketed with non-compacted fill, comprising soft to firm sandy clays, sandy silts and silty sands to a depth of approximately 5 metres. The fill is underlain by loose to moderately dense silty sands and some soft clays. Competent dark grey sandstones and siltstones of the Waitemata series rocks are present as bedrock below a depth of about 8.6 metres.

Boring 1 encountered cobble-sized basalt boulders, down to a depth of 5.5 metres below the footpath surface. The boulders are in turn underlain by the loose sands and soft clays. These boulders may be part of the rock-filled timber breastwork which defined the limit of the earlier stage of reclamation filling.

The seawall bounding the east, north and part of the west sides of the site is comprised of hard competent concrete, which contains some basalt cobbles. The bottom 400 mm of wall concrete appears to have been affected by seawater, as it has become whitened and somewhat softer than the unaffected wall above (such that it can be indented by a fingernail). Hard siltstone of the Waitemata series rocks was encountered immediately below the base of the wall, at 9.3 m depth.

Groundwater was encountered in the borings, and the water surface appeared to follow with tidal sea levels, but with a lag which resulted in water level differences which we noted as ranging up to about one metre.

DISCUSSION AND RECOMMENDATIONS

Previous Earthquakes

Over the past 100 years, the shocks from more than 50 earthquakes are recorded as having been felt in Auckland. This represents an average of one every two years. The closest recorded epicentre to Auckland of a significant earthquake was off Port Waikato in 1891. This earthquake had a Modified Mercalli intensity in Auckland of about 7.

Since the Ferry Building was constructed, in about 1908, available information indicates that the maximum seismic shock experienced in Auckland would have been equivalent to a Modified Mercalli intensity of about 5, arising from the Buller earthquake on 16 June 1929. A similar intensity of seismic shock may have been experienced during other earthquakes, for example one centred off Taranaki in 1949, but detailed information is not available. However, it does seem certain that the Ferry Building has not yet been subject to an earthquake of intensity equivalent to that being assumed for the design of the proposed strengthening.

There is evidence of some minor structural distress to the building, specifically as cracking in the archway along the north side of the building and in cross-walls at higher levels. Our inspection indicates that this could only be explained by a spreading of the building at ground level, such as by an outward movement of the top of the seawall, rather than by any foundation settlement. Perhaps this movement occurred during an earlier earthquake

Piled Foundations

The foundation piles, which are 0.45m-square precast-concrete driven piles, apparently had a design working load of about 110 tonnes. This is equivalent to an end-bearing pressure of about 5400 kPa. We have no detailed information on the construction of the piles, so, for purposes of this report, we have assumed that they were driven to near refusal on the underlying weathered Waitemata series bedrock. The piled foundations appear to have performed satisfactorily under the normal vertical loadings imposed by the existing building. This observation is consistent with our opinion that an end bearing pressure of 5400 kPa, under the normal working load condition, is reasonable for such piles driven to near refusal on the underlying bedrock.

As indicated, it is proposed to strengthen the Ferry Building so that its structure has better seismic resistance. This strengthening of the structure will of itself involve some increase in the normal vertical loadings on the foundation piles. We would expect that an increase in the end-bearing pressure of up to 10 percent, that is up to 6000 kPa, should only cause minor additional deformation.

With the added effects of the strengthening, and including allowance for seismic overturning, the maximum working load on a pile is expected to increase to about 200 tonnes. This is equivalent to an end-bearing pressure of about 9700 kPa. We are concerned that under this loading condition the piles may be overloaded. We would prefer to keep the maximum end-bearing pressure, for the dead plus live plus seismic working load condition, down to less than 9000 kPa.

We must point out that we cannot substantiate these allowable bearing pressure figures, other than by our judgement based on the performance of the building to date, information on the Waitemata Series bedrock obtained from the borings, and results of a full-scale pile loading test carried out some years ago for the adjacent Downtown project.

The net uplift capacity of a pile under the seismic working load condition may be assumed to be 5 tonnes. Note that in assessing the gross uplift capacity, which would include the added weight of the pile itself, the weight of the pile must be reduced to allow for bouyancy effects.

Seismic Liquefaction

When a loose saturated sand is subjected to ground vibrations, it tends to densify and decrease in volume. In the case of a fine sand, and particularly when silty fines are present, the water between the particles cannot drain so the reduction in volume causes an increase in porewater pressure. If the porewater pressure increases to the point at which it equals or exceeds the overburden pressure, then the effective stress between the particles becomes zero, the sand loses its inter-granular strength completely, and the soil adopts a liquified state analagous to that of a quicksand.

The grading and relative density of the silty sand at this site are such that we consider liquefaction must be considered a real possiblity under severe earthquake conditions. Based on the formulae included in a paper entitled "A Simplified Procedure for Evaluating

Soil Liquefaction Potential", by Seed H.B. and Idriss I.M., we estimate that the onset of liquefaction could correspond with a maximum earthquake ground movement acceleration of between about 0.07g and 0.12g, based on a minimum of 20 stress cycles, which is equivalent to an earthquake intensity of about 6 or 7 on the Modified Mercalli scale. For the design earthquake coefficient of 0.20, which is equivalent to an earthquake intensity of about 7 or 8 on the Modified Mercalli scale, it should therefore be assumed that liquefaction would occur.

We have considered the possibility of injection grouting to avoid this risk of liquefaction. However, the grading of the silty sands is too fine for this to be feasible, even with the use of chemical grouts.

Whilst the subsoils are confined by the seawall, any lateral spreading due to liquefaction cannot occur. However, there would be an increase in relative density of the silty sands which could cause a surface subsidence. If for example a 3-metre-thick layer was subject to an increase in relative density of 20%, then the resulting subsidence would be on the order of 500mm. However, if surface venting (sand boils) of the liquefied material were to occur, the subsidence could in the extreme be equal to the total thickness of the layer prior to liquefaction; that is, three metres in the example given. For this site, we consider this latter possibility to be most unlikely.

Seawall Foundation

The one boring drilled through the seawall indicates that it is founded on sound bedrock. A softer zone of concrete in the bottom of the wall is probably due to the effects of seawater, or the result of tremie placement of the concrete, or both. For assessing overturning of the wall under the seismic loading condition, we consider that the gross maximum toe bearing pressure (for unfactored working loads) should not exceed 2000 kPa.

The seawall is relatively rigid, and must be considered as unyielding if it is to fulfill its role of supporting the outer wall of the Ferry Building. It is therefore appropriate to use the "at-rest" coefficient for lateral pressure, Ko, which for this material may be taken as 0.60. This is applied to the effective vertical pressure; that is, to the gross vertical soil pressure reduced by the buoyancy effects of water pressure below the groundwater level.

In addition, the wall is subject to hydrostatic pressure due to any difference in water levels, from one side of the wall to the other, resulting from any lag in response to tidal variations of sea level - for design purposes this maximum difference may be taken as 1 metre.

Lateral Effects of Seismic Loading

Under the seismic loading condition there will also be a lateral dynamic loading on the seawall due to acceleration of the soil mass. For preliminary design purposes, we estimate that this may be taken as a uniform lateral pressure of 25 kPa acting on the full height of the wall. In the case of liquefaction, the combined static and dynamic lateral pressures due to the soil would be substituted by the dynamic effects of a heavy fluid with density equal to 1900 kg/m3.

The development of any lateral resistance due to passive pressure on the piles and foundation beams would require movement. However, movement cannot occur, because of the direct connection of the building to the relatively rigid seawall, without corresponding deformation of the building itself. Accordingly, the lateral seismic force from the building should also be transmitted to the top of the seawall during south to north earthquake loading. If the building did deform, then the proportion of lateral seismic loading taken through passive pressure would still in turn be transferred, through the upper soil, to the face of the seawall.

The same problem occurs in reverse with north to south earthquake loading, and may be the more severe condition as it would develop tension across the base of the building. This may be the explanation for the minor cracking observed along the north side of the building. Perhaps the foundation grillage could even yield or separate from the wall due to tension across the base of the building, and in this case the seismic loading would then need to be resisted by passive pressure against the upper (non-liquefied) zone of reclamation filling, and any breastworks, out into Quay Street. For purpose of design calculations this may be assessed using a passive co-efficient for lateral pressure of 2.0, but with no additional allowance for friction under the base of the pile caps.

Another potential problem under seismic loading is that the foundation piles could be subject to high shearing forces at the interface between the upper reclamation fill and the lower zone of silty fine sand, which may be subject to liquefaction. If the piles are brittle they may fail in shear or bending at this interface.

General

This study indicates that there are potential problems of pile capacity, soil liquefaction and seawall stability, which will need to be resolved in any seismic strengthening of the Ferry Building. The general scope of these problems has been identified by these studies, and preliminary geotechnical design criteria are provided here as a basis for structural engineering feasibility studies.

If the project proceeds to final design, then we consider that further site investigation, involving more drilling and testing, will be required. Further data and analyses will also be required to enable the review and amplification of the preliminary design criteria.

The following plates are attached and complete this report.

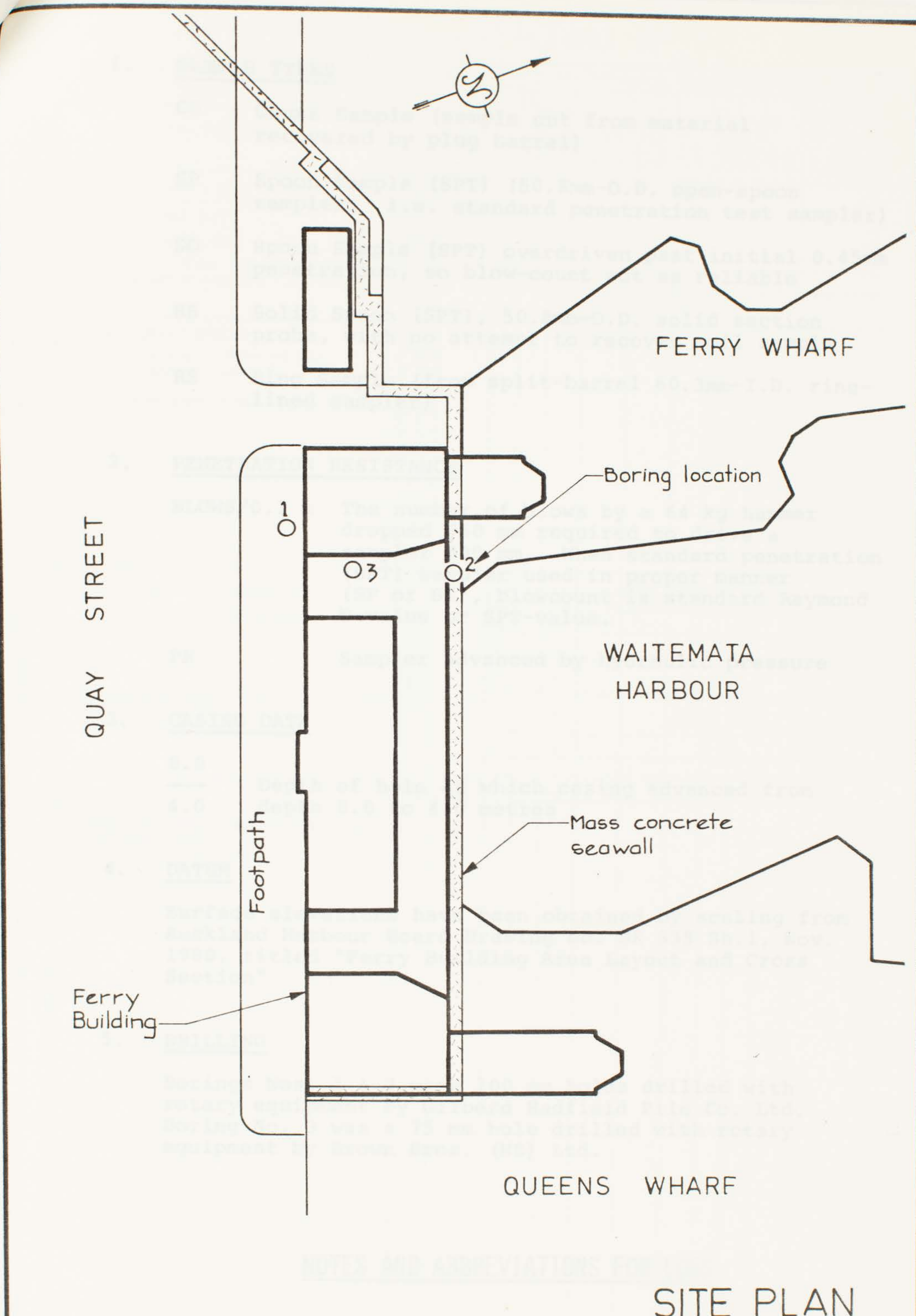
Plate 1	Site Plan
Plate 2	Notes and Abbreviations for Logs
Plates 3-A to 3-C	Boring Logs
Plate 4	Method of Soil Classification
Plate 5	Grading Tests

Yours faithfully,

p.p. Brickell, Moss & Partners

David E. Hollands

JOB 56259 CLIENT Wargon Chapman & Gurley LOCATION Quay St BY N.M.B. CHECKED DATE



SITE PLAN

Scale 1:500

Brickell, Moss & Partners

JOB 56259 CLIENT Auckland Harbour Board LOCATION Ferry Bldg, Auckland BY NM-B CHECKED DEH

1. SAMPLE TYPES

CS	Chunk Sample (sample cut from material recovered by plug barrel)
SP	Spoon Sample (SPT) (50.8mm-O.D. open-spoon sampler; i.e. standard penetration test sampler)
SO	Spoon Sample (SPT) overdriven past initial 0.45mm penetration, so blow-count not as reliable
SS	Solid Spoon (SPT), 50.8mm-O.D. solid section probe, with no attempt to recover soil sample
RS	Ring Sample (from split-barrel 60.3mm-I.D. ring-lined sampler)

2. PENETRATION RESISTANCE

BLOWS/0.3 m	The number of blows by a 64 kg hammer dropped 760 mm required to drive a sampler 300 mm. When standard penetration (SPT) sampler used in proper manner (SP or SS), blowcount is standard Raymond N-value or SPT-value.
PH	Sampler advanced by hydraulic pressure

3. CASING DATA

$\frac{0.0}{4.0}$ Depth of hole at which casing advanced from depth 0.0 to 4.0 metres

4. DATUM

Surface elevations have been obtained by scaling from Auckland Harbour Board Drawing No. SK 835 Sh.1, Nov. 1980, titled "Ferry Building Area Layout and Cross Section"

5. DRILLING

Borings Nos. 1 & 2 were 100 mm holes drilled with rotary equipment by Gilberd Hadfield Pile Co. Ltd. Boring No. 3 was a 75 mm hole drilled with rotary equipment by Brown Bros. (NZ) Ltd.

NOTES AND ABBREVIATIONS FOR LOGS

Brickell, Moss & Partners

JOB No. 56259 LOCATION Ferry Building, Auckland FIELD LOG BY NMB FINAL LOG BY NMB DATE 17.6.81 DRILLING CONTRACTOR Gilbert Ha...

SURFACE ELEVATION 5.2 DATUM AHB

GROUNDWATER DEPTH 1.43* DATE 6.5.81

FIELD: * Water level appears to correspond with tide level

DRILLING DATE(S) 4 - 5.6.81

BORING No. 1

Drilling method	Casing depth	Stratigraphy	Log	Elev / Depth (metres)	Type	Sample lost / disturbed	Blows per 0.3 metres	Dry density Mg/m³	Natural water content %	Liquid limit	Plasticity index	% fines (<75 µm)	Type of test	Parameters (Su kPa, c' kPa, φ' degrees)	Other
Plug Barrel	0	PAVEMENT (bitumen + base course)		4.8 / 0.4											
Plug Barrel		Brown SILTY SAND (SM) and SANDY CLAY (CL) (moderately dense FILL, with pieces of weathered mudstone & siltstone)	FILL		CS										
				3.2 / 2.0	SP		1								
Rotary Plug		Dark grey BASALT COBBLES (hard, vesicular, inferred size 100mm to 300mm) (with some brown SCORIA, hard)	FILL	-0.3 / 5.5											
Plug Barrel		Dark grey ORGANIC SILTY SAND (SM) (loose, with pieces of wood, organic, some OL)			CS				29.7			43			
					CS				26.1			36			
					RS		PH	1500	29.1			34			
					SS		3								
				-2.1 / 7.3	SS		5								
		Grey SANDY CLAY (CH) (soft, organic material present)			SP		0			60	35	78			
				-3.3 / 8.5	SO		9								
	0	Grey SILTSTONE (very stiff, moderately weathered Waitemata Series)		8.7	SP		50								

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SURFACE ELEVATION 5.0 DATUM AHB

GROUNDWATER DEPTH * DATE

* Not recorded

BORING Nº. 2

DRILLING DATE (S) 5-6-7-8.5.81

Depth (m)	Drilling method	Casing depth	Stratigraphy	Log	Elev / Depth (metres)	Sample type	Sample lost / disturbed	Blows per 0.3 metres	Dry density Mg/m³	Natural water content %	Liquid limit	Plasticity index	% fines (< 75 µm)	Type of test	Parameters Su kPa, c' kPa, ø' degrees	Other
0 – 2.5	Tricone Bit	0	Grey CONCRETE (hard, homogeneous; concrete seawall)	CONCRETE WALL												
2.5 – 9.3	Diamond Coring		(with grey BASALT COBBLES, hard, vesicular, size 100mm to 300mm contained within concrete)	CONCRETE WALL	-3.9 / 8.9	CORE SAMPLE RETAINED										
8.9 – 9.3	Diamond Coring		White CONCRETE (firm to stiff, affected by seawater)	CONCRETE WALL	-4.3 / 9.3	CORE SAMPLE RETAINED										
9.3 – 10.4	Plug Barrel	0	Grey SILTSTONE (very stiff, moderately weathered Waitemata Series)		-4.7 / 9.70	SP		50								

JOB Nº 56259 LOCATION Ferry Building, Auckland FIELD LOG BY NMB FINAL LOG BY NMB DATE 17.6.81 DRILLING CONTRACTOR Gilberd

BRICKELL MOSS & PARTNERS

PLATE 3·B

JOB Nº 56259 LOCATION Ferry Building, Auckland FIELD LOG BY NMB FINAL LOG BY NMB DATE 17.6.81 DRILLING CONTRACTOR Brown Bros.

BORING Nº 3

SURFACE ELEVATION 5.4 DATUM A.H.B.

GROUNDWATER DEPTH 3.65* DATE 27.5.81

*Water level appears to correspond with tide level

DRILLING DATE(S) 27.5.81

DRILLING METHOD: Plug Barrel (0 – 9.3 m)

CASING DEPTH: 0

STRATIGRAPHY	LOG	ELEV / DEPTH metres
Concrete PAVEMENT		5.2 / 0.2
Brown SANDY CLAY (CL) and SANDY SILT (MI) (soft - firm, FILL, pieces of brick, organic material) (grades grey, with brown layers) (small pieces of broken mudstone) (pieces of brick) (small pieces of sandstone and siltstone, shells, piece of scoria)	FILL	0.0 / 5.4
Grey SILTY SAND (SM) (loose) (bands of firm - stiff silt) (grades dark grey, moderately dense) (very loose, with shells)		-3.3 / 8.7
Yellow CLAYSTONE (hard)		9.1
Grey SILTSTONE (very stiff, moderately weathered Waitemata Series)		9.3

TYPE	SAMPLE LOST DISTURBED	BLOWS PER 0·3 METRES	DRY DENSITY Mg/m^3	NATURAL WATER CONTENT %	LIQUID LIMIT	PLASTICITY INDEX	% FINES (<75 µm)	TYPE OF TEST	PARAMETERS Su kPa c' kPa ϕ' degrees	OTHER
SP		2								
CS										
SP		3								
SO		5								
CS										
SP		8								
SO		5								
CS										
SP		10								
CS				32.1	NP		45			
SP		12		28.1			30			
CS				41.5	NP		52			
SP		2		30.8			42			
SP		50								

BRICKELL MOSS & PARTNERS

PLATE 4

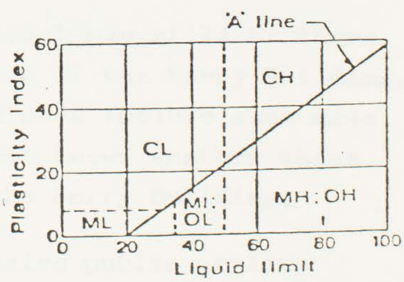
JOB 56259 CLIENT Auckland Harbour Board LOCATION Ferry Bldg, Auckland BY N.M-B CHECKED

Major division		Symbol	Description
COARSE GRAINED SOILS (less than 50 percent fines)	GRAVELS	GW	Well graded gravels or sand-gravel mixtures, less than 5% fines.
		GP	Poorly graded gravels or gravel-sand mixtures, less than 5% fines.
		GM	Silty gravel, gravel-sand-silt mixtures.
		GC	Clayey gravel, gravel-sand-clay mixtures.
	SANDS	SW	Well graded sands or gravelly sands less than 5% fines.
		SP	Poorly graded sands or gravelly sands, less than 5% fines.
		SM	Silty sand, sand-silt mixtures.
		SC	Clayey sand, sand-clay mixtures.
FINE GRAINED SOILS (more than 50 percent fines) SILTS & CLAYS	LL < 50	ML	Inorganic silts, rock flour, sandy or clayey silts of low plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
		OL	Organic silts and organic silty clays of low to medium plasticity.
		MI	Inorganic silts or clayey silts of medium plasticity.
	LL > 50	MH	Inorganic silts, micaceous or diatomaceous silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of high plasticity, organic silty clay, organic silt.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.

CLASSIFICATION	EQUIVALENT SIEVE SIZE B S	A S
COBBLES	8 in - 3 in	200mm - 75mm
GRAVEL,	3 in - 3/16 in	75mm - 4.75mm
coarse	3 in - 3/4 in	75mm - 19mm
fine	3/4 in - 3/16 in	19mm - 4.75mm
SAND,	3/16 in - No 200	4.75mm - 75 μm
coarse	3/16 in - No 7	4.75mm - 2.36mm
medium	No 7 - No 36	2.36mm - 425 μm
fine	No 36 - No 200	425 μm - 75 μm
FINES, silt & clay	below No 200	below 75 μm

COHESIONLESS SOILS	
RELATIVE DENSITY	'N' (SPT) VALUE. blows/ft.
VERY LOOSE	0 to 4
LOOSE	4 to 10
MODERATELY DENSE	10 to 30
DENSE	30 to 50
VERY DENSE	Above 50

COHESIVE SOILS		
CONSISTENCY	UNDRAINED SHEAR STRENGTH. p.s.f.	kPa
VERY SOFT	0 to 250	0 to 12.5
SOFT	250 to 500	12.5 to 25
FIRM	500 to 1000	25 to 50
STIFF	1000 to 2000	50 to 100
VERY STIFF	Above 2000	Above 100



METHOD OF SOIL CLASSIFICATION

(UNIFIED CLASSIFICATION SYSTEM)

BRICKELL MOSS & PARTNERS

WCG

J. Spencer Nicholls, BE, MICE, MNZIE
CONSULTANT

XXXXXXXXXXXXX XXXX
P O Box 47-215, PONSONBY 760-772

December, 1980

FERRY BUILDING - FOUNDATION SITUATION

1. CUSTOM STREET/QUAY STREET RECLAMATION:

The history of reclamation for the area Custom Street to Quay Street is summarised in report No.5250 of July, 1967 by Brickell Moss Rankine & Hill. This report was carried out in conjunction with the 'A.H.B. Central Area Properties Redevelopment' and includes records of bores to establish stratification of the underlying Waitemata sedimentary deposits below, in particular, the present Air New Zealand Building. This building is located South and slightly West of the Ferry Building, separated by the approx. 30.5 metre (100 feet) boundary-to-boundary width of Quay Street.

The reclamation work described extended to about the North side of Quay Street and was terminated by a rock-filled timber breastwork. The Ferry Building seems to have been located immediately North of this breastwork, i.e. immediately BEYOND the original reclamation area. Presumably the rationale was that the Ferry Building should not be built over or supported on the pre-existing breastwork but that it should be located as close to it as pile-driving operations for the (then) new building would permit.

If this is so, then the old breastwork may still be there under the footpath immediately South of the Ferry Building. Further to the East the breastwork does include some mass concrete anchor-blocks but it is not known whether these occur in the section adjacent to the Ferry Building.

The footpath area does carry extensive public utility services. The A.H.B. Drawing Office is currently compiling a consolidated record of these. The Chief Draughtsman (Mr. Sinclair) has made enquiries but is not aware of any

.... 2/

occasion when the breastwork was encountered in the course of utility trenching. Auckland City Council has local authority jurisdiction over Quay Street including the footpath. My understanding is that it also has jurisdiction over the building itself but not over the Ferry Basin (North) or the wharves East and West. I have not checked the legal issues on this because they have not seemed important.

2. THE CONCRETE QUAY WALL:

A.H.B. Drg. No. 18A dated 9.V.06 shows the stepped mass concrete sea wall built generally beyond and to the North of the earlier reclamation. Drg. SK835/1 of November, 1980 shows relationship to building. One leg of this wall is under and supports the North facade of the Ferry Building. Another returns under and supports the Eastern facade. However, the Western return is about 6m outside and beyond the Western facade. This width provides for vehicular access to the Ferry Tees (wharves). The outside face of the wall is clearly visible. Part of the inside face is visible from a small basement under the North-East corner of the building. There is thought to be another basement under the South-East corner and the A.H.B. Construction Engineer (Mr. Goord) is checking on access to this.

In places, the external wall face has weathered at old pour joints to a depth of 30-50mm and revealed a cement matrix and aggregate grading of indifferent quality.

The old drawings show the wall keyed into 'rock' at a level 4.5m below Quay Street. There is evidence (see below) which may suggest some movement of the wall particularly near the centre of the building. Hydrographic records of mud levels in the Ferry Basin (immediately North) are available but not rock levels. If the rock levels drop away drastically then some present building damage would be explainable and geoseismic stability of the wall foundation would become an issue. In any case it is desirable to sample rock below the wall foundation but I prefer to do this after "prickings" of rock levels in the basin are available. Mr. Goord is looking into this.

.... 3/

IF the rock levels and quality seem satisfactory and if ther are no major voids in the wall -
THEN Vertical loads on the wall (gravity plus seismic overturning) are not likely to be problems. Seismic shear parallel to the line of each wall is not likely to be a problem either but seismic shear perpendicular to the wall face is. The fact that there is no return wall under the Western facade combined with the high aspect ratio of the building suggests a torsional problem in transferring seismic shears from Quay Street level down to rock.

Incidentally, the Quay Wall Drawing also shows a (then existing?) weighbridge under (the now) Quay Street footpath running almost half the length of the building from about the Eastern end.

3. THE BUILDING:

The (quite detailed) superstructure drawings by Architect Wiseman do not show any details of foundations below brick haunchings to walls. There is a drawing showing "Plan of Concrete Foundations" together with figures for superimposed superstructure gravity loads. This is the very sort of co-ordination document one would expect the Architect to issue if the foundations were done under a separate contract for which he was not primarily responsible. There is another Wiseman drawing showing layout, sizes and loads on piles together with some 'counterforts' to support superstructure elements over the set-backs of the sea-wall.

Other A.H.B. files suggest that detailed design and construction of the foundations was carried out by the Ferro Concrete Co. of Australia Ltd., under a separate contract. F.C.C. also built the adjacent Queen's Wharf and Ferry Tees at about the same time. Indeed the precise demarcation between the three structures is not obvious but the single storey North wings of the building (certainly the North-Western one) are likely to be supported on the wharf structures.

One notes that the period 1906-1910 was an intense period of

.... 4/

of major expansion of port facilities and that the F.C.C. had a substantial part in this at a time when reinforced construction would have been in its infancy.

Messrs. Sutton and Angus (former Chief Engineers) believe the F.C.C. may have originated from Sydney and I am arranging for some enquiries there.

The building does show some movement damage to the arches over the collonade running along the North side and in the first floor brick partitions immediately above. It is not yet clear whether this problem indicates movement in the piles or in the quay wall. If in the piles then the possibilities seem to be:

* long term consolidation of Waitematas
* inadequate driving set
* driving damage to pile or tip
* long term structural deterioration of piles or footing beams exposed to salt water environment.

It may not be possible to arrive at a precise detailed diagnosis but it is necessary to tie this down as much as possible by:

* digging test-pits to examine the condition of foundation beams and upper parts of piles
* by making some geomechanic estimates based on the properties of the Waitematas and some guesswork as to the pile-driving equipment used.

Some new piles may be required to rectify the existing problems and/or to deal with the seismic overturning forces. Seismic forces will be an outcome of superstructure strengthening proposals which have not yet been developed in detail (but see my report of August, 1980). Overturning forces are not likely to be so large as to involve major nett tension - indeed this will be one of the objectives of detailed superstructure proposals. If geomechanics shows the ultimate pile capacity under short-term (seismic) load to be substantially higher than under long-term loads then seismic overturning may not be an issue.

Diaphragm capacity at Quay Street level may be an issue and the architectural documents are somewhat vague on ground floor slab details and slab-wall connections. Any information available when opening test pits should be recorded.

Seismic shear capacity below Quay Street level may involve:

* Raker piles, or
* Concrete walls - e.g. by 'slurry trench' technique, or
* Some form of stabilisation of ground perhaps outside the building plan area together with some anchors from the building at high level.

Raker piles are not favoured because of ductility problems under overload although it may be possible to use them if they provide a fairly conservative level of strength relative to the "as upgraded" superstructure.

It may be possible to provide some new basement spaces the walls of which would carry the seismic shears down 4.5m to rock. Construction dewatering is a consideration. Long term damp-proof occupancy would be difficult and expensive.

4. OBJECTIVES OF SUBSURFACE EXPLORATION:

4.1 To assess causes of present damage at Northern collonade.

4.2 To permit assessment of capacity of existing piles.

4.3 To evaluate techniques for adding new piles.

4.4 To permit assessment of capacity of new piles.

4.5 To permit assessment of technique for new elements to resist North-South seismic shear acting on, say, the Western half of the building but including tower foundation. Perhaps also for East-West shears from the South facade.

5. SUBSURFACE EXPLORATION PROGRAM:

5.1 Rock levels in Ferry Basin. Survey the top edge of the Quay wall (plan and level) to see whether variation along length is haphazard (setting out error) or systematic

....

(movement). Any systematic 'tilt' in piers above? Mr. Goord, Mr. Nicholls.

5.2 Shallow drilling of foundation beams in association with drilling of masonry superstructure - Mr. Goord, Mr. Nicholls.

5.3 Shallow test pits, to examine existing foundation beams and upper parts of piles.

5.4 Deep drilling program. Generally along lines proposed by Brickell Moss & Partners (letter 3.12.1980, file 56259) but reconsider drilling locations in light of results from 5.1, 5.2. Perhaps simultaneous with 5.3.

pp CCG

Colin Gurley

DISTRIBUTION:

A.H.B.	Chief Engineer	- Mr. Le Clerc
	Assistant Chief Engineer	- Mr. Bray
	Construction Engineer	- Mr. Goord
	Chief Draughtsman	- Mr. Sinclair
A.C.C.	City Design Engineer	- Mr. Leadbeater
	City Structural Engineer	- Mr. Kratky

Wargon Chapman & Gurley - Mr. Gurley
- Mr. Nicholls

Brickell Moss Rankine & Hill - Mr. Hollands
(Takapuna Office)

Enclosed - Copy Brickell Moss letter dated 3.12.1980

~~ALEXANDER WARGON~~, MSc, CE, MNZIE, FIE Aust, FASCE.
ROBERT F. CHAPMAN, BE, ASTC, MNZIE, FIE Aust.
COLIN R. GURLEY, MEng. Sc., BE (Hons.), MNZIE, MIE Aust. MASCE.

WARGON CHAPMAN AND GURLEY
REGISTERED ENGINEERS
CONSULTING STRUCTURAL & CIVIL ENGINEERS

27 SYMONDS STREET, AUCKLAND, N.Z. **PHONE 797-584**

10 September 1980

The Chief Engineer
Auckland Harbour Board
PO Box 1259
AUCKLAND

Dear Mr Seagar

Re: Ferry Building

Further to our report on an overall scheme for seismic strengthening of the Ferry Building, we have now to report our proposals for sampling and testing the structurally relevant materials as they presently exist.

You will recall that the earlier report suggested a clear distinction between:

* Initial testing of masonry superstructure
* Full testing of masonry superstructure
* Foundation investigation

This letter is principally concerned with the testing of the superstructure although you will need to bear in mind the foundation investigation in assessing total investigation budgets.

Testing of the masonry is a matter for which there is little directly relevant precedent. Investigation costs are potentially such that it will be necessary to maintain a careful scrutiny of proposals and to carefully monitor the progress of testing to ensure that the results achieved are of direct practical value.

Ch. NZIG Mk
Consulting
Eng.

So as to keep my own judgement clear on these issues I have asked Mr J.S.F. (Spencer) Nicholls to assist me by drawing up an outline for the program. His notes are attached and having discussed them with him in some detail I am satisfied that they are appropriate.

Mr Nicholls suggests a budget of $6500 for the initial masonry investigation. This phase is intended to go only as far as required to establish the feasibility and viability of testing proposals. My feeling is that his figure may prove a little restrictive but there is some elasticity depending on the extent to which the Board is able to supply labour and plant from its own resources.

2/...

10 September 1980

The full masonry testing program I would expect to be left until after there is some definite committment to procede with strengthening. It would aim to supply detailed quantitative information for design and construction and it would become interwoven with the design program.

Foundation testing will initially involve a substantial drawing office effort in compiling the best available "as built" drawings from your own records and other sources mentioned in earlier correspondence. These should be carried as far as possible before bringing in drilling rigs. We need to discuss with you the extent to which your own staff are able to undertake this work in consultation with ourselves.

Yours faithfully

Colin Gurley

Colin R Gurley
WARGON CHAPMAN AND GURLEY

AUCKLAND FERRY BUILDING

BRICK TESTING PROGRAMME - NOTES 4.9.80

1. The tests outlined on Sheets 1,2 & 3 show a full programme to enable structural design to proceed to completion. Initial testing to enable design to commence would be considerably briefer and less complex. (See later clause.)

2. Assembling a test procedure for this project has no clear precedent. U.K. practice and the current Wellington work on the University 'Hunter' building is generally looking at demolition of the main fabric and retaining only an original facade. European work (Italy, Greece) has restored many antique masonry structures, but not specifically for earthquakes and they generally avoid adding any externally applied forces.
Refer - FONDEDILE, S.p.A. Statice Restoration of Historic Monuments.

3. The recently available 'Tentative Los Angeles Ordinance and Testing Programme' (Schmid, Kariotis, Schwartz) is an excellent example of a programme assembled to give results suitable for use in the design process. This programme was for a particular type of building, with particular materials and the investigators had free use of 'sample' buildings. The programme is thus very useful, but only partly relevant to our situation.

4. Generally, testing throughout the structure must be sufficient to give realistic mean values and these values to have sufficient credibility to enable their confident use throughout the structural design process. Hopefully, the test programme would yield sufficient results that the designers knowledge of the buildings structure fabric would be as complete as for a new, modern, conventionally schemed structure.

5. The tests listed are a combination of laboratory and insitu tests and only attempting what is outlined will give guidance on the method that will ultimately provide the most credible results.

Generally, the insitu tests will have advantages over the laboratory tests. These laboratory tests will suffer badly from:

a) sample breakage and actual sample loss, or
b) sample cracking and material disturbance during removal that will lead to unusable results.

6. The tests as listed will give direct results for compressive and shear strengths and hence allow assessable figures for:

a) tension
b) diagonal tension/compression
c) reduction factors for large elements, e.g. piers, etc.

<u>but not creep</u>.

Creep tests will be necessary if prestressing techniques are proceeded with as part of the remedial measures. However, conventional creep tests are generally carried out in a laboratory in controlled temperatures and humidity and continue for say, 10 or 12 years. Further investigation of the possibilities of creep testing are proceeding.

7. <u>INITIAL TEST PROGRAMME</u>:

We recommend this be carried out well in advance of any serious structural analysis and planning of permanent remedial works.

These tests will generally cover:

a) Hand sampling throughout the structure using hand drills and power drills.
b) Exploratory use of a SCHMIDT hammer.
c) Some limited diamond coring and laboratory testing to establish the method viability.
d) An attempt to remove a wall brickwork sample for compressive testing.
e) Investigation of local hydraulic ram availability.

Richard Fenwick
Spencer Bright
Peter Taylor
Ceramco

f) Investigation of local laboratory facilities and diamond drilling equipment.

g) Investigate creep testing.

The final extent of this work could vary daily, depending on the results observed, however we estimate a suitable maximum budget of $6,500.00.

8. FULL BRICKWORK TEST PROGRAMME:
See notes attached - Addendum 1.

9. An onsite inspection of the visible building fabric was carried out on 4 September, 1980 by the writer in company with Mr. Alfred Way, retired Master Bricklayer, well-known and recommended by Amalgamated Brick Co. Ltd. of New Lynn, for his wide knowledge of brickwork and bricklaying over the last four decades. We enclose a brief report of that discussion - Addendum II.

J. S. F. NICHOLLS Registered Engineer

AUCKLAND FERRY BUILDING

ADDENDUM I

BRICKWORK TESTING PROGRAMME 4.9.80

A. BRICKS : Remove sample bricks from existing structure. Take samples from inner and outer wythes and bricks of varying colour.

1. Laboratory test to assess compressive and shear strengths.
2. Laboratory test to assess range of strengths in the structure.
3. Analyse results to assess mean brick strengths, floor by floor.
4. Laboratory test for any particular properties.
   e.g. excessive moisture penetration,
   excessive moisture movement,
   any thermal movement.

Investigate use of 'SCHMIDT' hammer to aid in above.

B. MORTARS : Remove samples of both cement and lime mortars using diamond coring equipment. Cut, trim and 'cap' as necessary and laboratory test to give:

1. Compressive strengths
2. Mean compressive strengths.

Investigate the use of chemical analysis per an Industrial Chemist (Dr. Sprott).
Investigate the expected strengths deducible from the contract specification clauses.

C. BRICKWORK :

1. Assess rupture strength of mortar 'in shear'
   - a) Using diamond drum coring tools & lab. tests
     - in outer wythes
     - in inner wythes
   - b) Using insitu tests by selective mortar removal Refer L.A. page 17
     - in outer wythes
     - in inner wythes (where feasible)

C. BRICKWORK: (Cont..

2. Assess rupture strengths of the brick & mortar composite in compression
   - a) Cut and remove & laboratory test sample say 600 x 600 x 325
     - in outer wythes
     - in inner wythes
   - b) Using an 'INSITU' face test Refer L.A. page 23 — in outer wythes only

Test C.2b - measure strain and stress during the test. This test can only assess a maximum likely face compression resistable by the outer wythes of the sampled wall.
i.e. A useful check on the walls capacity to resist 'face' loads.

Test C.1a - check local test machine sample size limitations.

Tests C.1&2 Ensure a sampling from both the main building cross walls and the tower walls.
Ensure the sampling identifies both the lime and cement mortars.

D. SANDSTONE: Sample using diamond coring tools as necessary to clarify:

1. Mean compressive/shear strengths.
2. Extent of ageing deterioration.
3. Condition of typical joints.
4. Extent of water penetration.
5. Whether deteriorated areas are repairable or will need replacement.

Investigate creep properties.

E. METAL FIXINGS/ANCHORAGES:

1. Decide likely types of any anchorages envisaged for remedial scheme. Arrange tension and shear strengths tests as necessary.
2. Examine any existing anchorages or fixings and assess condition.

F. BUILDING WALLS ALIGNMENT:

1. Inspect structural walls, piers and slabs to assess:
   a) Horizontal or vertical misalignments.
   b) Any set-backs NOT shown on drawings.
   c) Any likely wall eccentricities.
   d) Any wall cutting or chasing for service runs in structurally critical areas.

J. S. F. NICHOLLS Registered Engineer

AUCKLAND FERRY BUILDING

ADDENDUM II

SUMMARY OF SITE VISIT TO ABOVE BUILDING ON 4 SEPTEMBER, 1980.

Those present being - Mr. Alf Way, retired Master Bricklayer
Mr. Colin Gurley, Consulting Civil/ Structural Engineer
Mr. Spencer Nicholls, Consulting Civil/ Structural Engineer

BRICKS:

Facings - appeared to be red, machine pressed to specification (frogs, even each side could be expected), local supply?

Backings: cream, machine pressed? local.

All laid in English and Garden wall bond.

Condition: Facings generally good. Marked weathering on the tower where an area shows deterioration, from erosion and dampness penetration, but all repairable.
Backings and interior brickwork all looks sound and good condition.

MORTAR:

Lime mortar (1:3) looked good as per the specification. Fine ground, scoriaceous, raked out easily, but well laid - i.e. in area examined no evidence of air pockets.

Cement Mortar (1:2) & Pointing - not examined this inspection.

DAMPCOURSING:

This looked for, but not located. What material? could be tar, asphalt, lead, slates ? current condition? expected reaction to increased compression from a prestressing force?

PLASTER TO STAIRWELLS:

Soft, fine, easily scratch removed.

STRONGROOM: East wing, top floor, solid cement walls. Actual support structure not evident.

SANDSTONE: Source assessed as probably ~~Piermont~~ Pyrmont, Sydney. Most corners on North face weathered to an arris. Some surface deterioration evident where the applied coating has broken down. This coating peeling extensively on all North-facing large flat areas.

Sandstone Jointing: It was observed that these joints could be dry and only the joint arris pointed for weather exclusion.
Mortar cross-keying identified in parapet stones.

ROOF: Apparently re-roofed with DECRAMASTIC tiles in the last decade. Much aggregate loss from the tiles, tile rusting is evident and we were advised by the Caretaker that the fixings have not proven resistant to wind and weather (since completion of the adjacent tower blocks?). Interior timber framing looked good.

WALL INTEGRITY: The 36" walls examined looked true to line and square. Bricklayer Way considers these would be of a regular quality throughout and would not expect any reduction in quality towards the centre.

SUGGESTED FURTHER INFORMATION SOURCES:

Building Centre - Bill Hitchcock, retired Engineer.
Old building centre or the Mackie Logan Building. We advised that this is a prime example of early Auckland brickwork.
St. Matthews Church - restoration procedures.

SUGGESTED FURTHER INFORMATION SOURCES: (Cont..

Clarence Albert Anderson, retired Architect, father of Caretakers wife, has tested levels throughout structure over several years.

Dr. Tony Bryant - Auckland University School of Engineering - Creep testing.

J. S. F. NICHOLLS Registered Engineer

McMahon Burgess & Yeates
Consulting Engineers & Geologists

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Telephone (02) 411 5588 Telex AA26784

13th August, 1980.

Our Reference PJNP/GJ.

Messrs. Wargon, Chapman & Gurley,
25 Symonds Street,
AUCKLAND. NEW ZEALAND.

Attention: Mr. C. Gurley.

Re: ENGINEERING PROPERTIES OF HAWKESBURY SANDSTONE.

Dear Mr. Gurley,

As discussed over the 'phone on the 8th August, 1980, I enclose a copy of my paper on the Engineering Properties of Hawkesbury Sandstone. This paper covers general engineering properties and there is some further information available regarding properties such as creep, modulus, moisture effects and physico-chemical deterioration.

Regarding the question of prestressing the building in Auckland to provide structural stability, I would be happy to produce an initial report summarising the relevant existing information and setting out suggested testing specific to the sandstone in your structure. Such a report would take about two weeks to complete and would involve about 12 hours of my time and a certain amount of junior engineer and drafting time.

Our estimated cost for this initial report is $700.00.

Yours faithfully,
McMAHON BURGESS & YEATES,

P.J.N. PELLS.

Sent surface mail
and received
Auckland 10/09/80
WCG: File AB/85
Ferry Building.

c.c. Mr. A. Wargon,
Wargon, Chapman and Associates,
245 Sussex Street,
SYDNEY. N.S.W.

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Reprint

of Paper published in

THE AUSTRALIAN GEOMECHANICS JOURNAL

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Measurement of Engineering Properties of Hawkesbury Sandstone

P.J.N. Pells*

SUMMARY The sandstones of the Hawkesbury formation underlie much of Sydney and are thus of considerable engineering importance. This paper presents a study of some of the significant engineering properties of these sandstones based on laboratory and field measurements. While investigating these various properties certain problems were encountered which are generally applicable to rock materials. Those chiefly concern the application of the effective stress principle to rock behaviour and the nature of the load deformation curve in the region beyond peak strength. These problems are discussed in some detail. The properties studied include

(i) Strength and complete load-deformation behaviour in uniaxial and triaxial compression.

(ii) The effect of degree of saturation on the uniaxial, triaxial and tensile strength.

(iii) Tensile and point load strength.

(iv) Shear strength of fresh joints.

(v) In-situ deformation and seismic properties.

Some data are also given regarding creep and permeability, as measured in the laboratory.

1 INTRODUCTION

The Triassic Hawkesbury Sandstone occupies an area of some 12,500 square kilometres in the Sydney Basin (Geology Society Australia, 1969). As much of Sydney is directly underlain by this sandstone, it is of considerable engineering significance. It has been tunnelled through for railway, water supply and sewer services, has been excavated to form spectacular cuttings on the freeways and supports the tower blocks of central Sydney and the arches of the Harbour and Gladesville bridges.

The engineering properties discussed in this paper were measured on samples from Manly, Helensburgh and from six quarried blocks of sandstone used to provide suites of near identical specimens. Most of the laboratory tests were performed on NX size core (54 mm diameter). Some size effect studies were undertaken using core that varied from 25 mm to 76 mm diameter. Other data were obtained from engineering investigation reports produced by the Snowy Mountains Authority, Australian Rock Engineering Consultants and Ground Test Pty. Ltd.

2 THE HAWKESBURY SANDSTONE

While the formation does contain a certain amount of shale, interlaminated with fine sandstone or occasionally in thick layers, it consists dominantly of quite massive sandstone beds of up to 15 metres in thickness. Strong cross-bedding is common. Although slope, tunnel and foundation stability problems have arisen at specific sites there is no doubt that in general the formation provides a good medium for engineering structures.

The sandstone is composed primarily of subangular quartz grains with an argillaceous matrix and some siderite cement. Secondary silica occurs mostly as overgrowths around grains and the development thus of crystal faces imparts a glistening effect to the rock. The degree of overgrowth development is variable and has an important bearing on the strength and stiffness properties of the material. Well developed overgrowth of detrital grains results in a strong, interlocked structure. Siderite is sometimes found in sufficient quantity to bind quartz grains together. The spaces between the quartz grains contain sericite and clay, as illustrated in Figures 1a and 1b. Where the degree of overgrowth is small, many of the grains are separated by the agrillaceous materials resulting in lower strength and greater compressibility. While the secondary silica does not act as a true cement SEM studies show (vide Figure 1c) that true cementation is quite widespread. The nature of this rather strange 'organic' looking cementing material is not yet clearly understood. Examination with an electron probe indicates it to be a potassium aluminium silicate but both its structure and its mode of attachment to silica grains are difficult to understand.

According to Standard (1969) the composition based on analysis of widely spaced samples from the whole outcrop, is as follows:

Detrital grains	
Quartz	68%
Others	4%
Matrix clay (70% Kaolinite, 20% Illite)	20%
Secondary silica and siderite	8%

* Mr. Pells is a Lecturer at the School of Civil Engineering at the University of Sydney.
(Paper P3555, submitted 27 April 1976).

However, it should be noted that quite wide variations in composition may occur even over one site, as illustrated by the data in Table I obtained during the foundation investigations for the MLC Centre (Australian Rock Engineering Consultants, 1973)

TABLE I

THIN SECTION ANALYSIS OF SANDSTONE SAMPLES FROM THE MLC CENTRE (ARENCO, 1973)

Depth metres	Frame-Quartz %	Sericite and clay %	Unaltered mica %	Siderite %	Voids %
11.2	38.4	52.4	0.6	8.6	
11.8	59.9	35.5	2.8	1.8	
12.9	73.1	17.9	0.6	0.7	7.7
16.9	60.6	36.0	0.8	2.6	
17.1	72.7	24.5	1.0	1.3	0.6
19.3	76	15.2	0.4	1.1	7.3
20.9	66.4	24.8	0.8	5.2	2.8

3 THE RELEVANCE OF THE EFFECTIVE STRESS PRINCIPLE

Before proceeding to examine the properties of Hawkesbury sandstone it is necessary to consider the existing evidence regarding the relevance of the 'soil mechanics' concept of effective stress to the strength and deformation behaviour of sandstone. Two aspects must be considered.

The first is with regard to the applicability of the Terzaghi effective stress equation to the strength of fully saturated sandstone. In the form originally proposed by Terzaghi (1936) the neutral stress (equal to the pore pressure) is considered as a basic hydrostatic stress existing in both the solid and fluid phases. The effective stress arises exclusively from the solid skeleton and is given by

$$\sigma' = \sigma - u_w$$

where

σ = total stress acting in a given direction at any point in the material

u_w = pore water pressure

While early investigations regarding the applicability of the effective stress principle to saturated rocks caused some confusion, due mainly to incorrect measurements of pore pressure in less permeable rocks, enough evidence has now been accumulated to show that the principle, as embodied in the above equation, is correct for both sedimentary and some very low porosity igneous rocks (Lane, 1970). Many explanations have been attempted for why the simple Terzaghi effective stress equation appears valid for materials of extremely low porosity, where one would expect the contact area to be significant (Skempton, 1960). Hubbert and Rubey (1959) in their widely quoted paper (e.g. Jaeger and Cook, 1969) attempt to show, in direct contradiction to Skempton (1960), that the contact area does not enter the effective stress equation at all. However, the mathematics from which Hubbert and Rubey draw this conclusion is completely unsatisfactory. The same is true of the analysis given by Harza (1949) who drew the same conclusion as Hubbert and Rubey.

(a) Overgrown quartz grains with clay matrix

(b) Clay platelets forming the matrix

(c) Cemented quartz grains

Figure 1

It appears that there are two reasons why valid experimental evidence shows that contact area does not enter the effective stress equation for the strength of very low porosity rocks. These are:

(i) Although the volumetric porosity is very low individual grains remain to a large extent separated from one another by very narrow cracks. (Sprunt and Brace, 1974).

(ii) Many materials which make up the solid grains of rock show virtually no increase in strength with confining pressure and, as shown by Skempton (1960), for such materials the effective stress equation with respect to shear stress does not depend on contact area.

The second aspect is with regard to the dramatic difference between the strength of the sandstone in the dry and saturated states, as illustrated in Table II and Figure 2.

TABLE II

Block No.	General Description		Uniaxial compressive strength	
	Grain size	Amount of clay matrix	Saturated MPa	Oven Dried MPa
1	Medium	Average	18.0 (5)	43.4 (2)
2	Medium	High	11.7 (4)	39.2 (2)
3	Fine	Average	21.6 (2)	53.9 (2)
6	Medium	Average	25.0 (7)	56.0 (3)

Note: The number in brackets is the number of specimens tested.

This well known phenomenon has been studied by many workers using a variety of rocks (a summary is given by Vutukuri et al, 1974). The phenomenon is very often ascribed to physico-chemical effects in pores and crack tips (Vutukuri et al, 1974, Colback and Wiid, 1965). However, Chenevert (1970) has suggested that the effect can to a large extent be explained by considering the negative pore pressures, set up in the rock when water is removed, together with the Terzaghi effective stress equation. Recalculating Colback and Wiid's data, as shown in Figure 3, he suggests these fit the hypothesis of high suctions at full saturation. However, the shear strength parameters implied by Figure 3 are obviously wrong which is not really surprising since it is certain that the specimens were not fully saturated. If most of the strength gain observed in rock as it dries is due to increased effective confining pressure, then for numerical calculation one must consider the Bishop-Aitchison equation for effective stress in partially saturated soil (Bishop et al, 1960), extended into the negative pore pressure range. If the pore air pressure is taken as atmospheric, this equation reduces to:

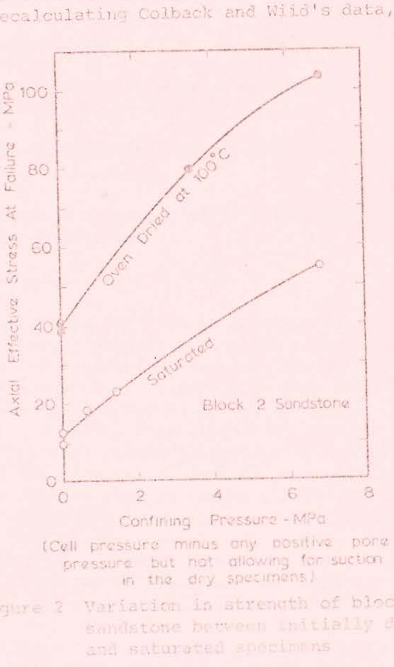

Figure 2 Variation in strength of block two sandstone between initially dry and saturated specimens

$$\sigma' = \sigma - \chi u_w$$

where

χ = a parameter with a value less than unity which is a function of the degree of saturation

u_w = pore water pressure, in this case a negative value (suction)

Values of χ can be calculated for ideal packings of granular materials (Donald, 1960). However, with real materials χ can only be determined if it is assumed that the effective stress shear strength parameters are unaffected by the process of desiccation (the data in Figure 3 imply χ values of 0.15 to 0.26). This may not be a valid assumption as it has been long known that the hardness of ceramics and rock forming minerals is a function of the chemical environment. Many workers have followed the theme suggested by Rebinder et al (1948) that the environment influences the surface free energy of any solid, thus affecting the ease with which cracks can propagate. However, more recent work indicates that surface active environments influence near-surface dislocation mobility and that the micro-hardness of many non-metallic solids is a maximum in environments when they are at their iso-electric point (zero surface charge) (Macmillan et al, 1975). Changes of 30% to 40% in the hardness of materials like quartz, microcline, Al_2O_3 and MgO are achieved by changing the surface charge. Detailed discussion of this subject is not appropriate in this paper but it should be noted that the proponents of chemomechanical phenomena as being the main cause of strength gain due to des-

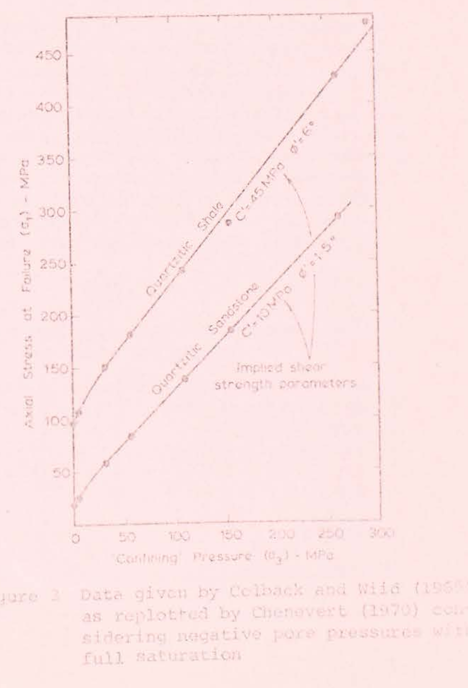

Figure 3 Data given by Colback and Wiid (1965) as replotted by Chenevert (1970) considering negative pore pressures with full saturation

ication never consider the effects of suction (matrix and solute) and visa versa. It may well be that the two schools of thought can be related through consideration of solute suction but for the present the general picture regarding the influence of environment on strength and hardness of rock remains confused.

4 STRENGTH AND DEFORMATION BEHAVIOUR UNDER UNIAXIAL AND TRIAXIAL STRESS

(i) Test Procedure

A study of the complete load deformation curves for the Hawkesbury sandstone under uniaxial and triaxial compression was undertaken using specimens cored in the laboratory from quarried blocks. All tests were conducted on specimens prepared with a length to diameter ratio of between 2.1 and 2.8. Most specimens were 54 mm diameter. The specimen ends were ground, using a CSIR type precision grinding jig (Benzinger, 1973), to closer tolerances than specified by the ISRM (1972). Specimens were tested between steel end pieces of the same diameter as the specimens, with a spherical seat being incorporated in the upper end piece. Saturated specimens were produced by subjecting them to quite high vacuum under water for an hour and then leaving them to soak for 24 hours. The process was repeated and then the specimens were stored under water until tested. All tests were conducted using a closed loop servo-controlled machine (INSTRON) either at constant rate of axial strain or constant rate of lateral strain. Test durations varied from 30 minutes to 5 hours.

The uniaxial tests were quite conventional except for the strain control used to effectively create a very stiff testing machine. Deformations were measured using both resistance wire strain gauges cemented to the specimens and LVDT's clamped across the full specimen length.

Three types of triaxial test were used. In soil mechanics terminology these were:

a) Unconsolidated undrained tests with pore pressure measurement

b) Consolidated undrained tests with pore pressure measurement

c) Drained tests.

It is most important in tests of this nature to ensure that the pore pressures within the specimens are as measured or controlled externally (Brace and Martin, 1968). The problems involved in this are discussed in detail by Bishop et al (1960) and by Bishop and Henkel (1962). Correct testing rates (for controlled rate of strain tests) can be calculated provided the compressibility and permeability of the material being tested is known. The results of these calculations are summarised in Table III for the sandstones from blocks 1 and 2. This indicates that drained tests on Hawkesbury sandstone can satisfactorily be conducted within a reasonable length of time. However, the data given in Table III do not tell the whole story for the case of undrained testing because the sensitivity of the pore pressure measuring system completely dominates matters. Both null indicator and transducer systems were used in the tests discussed here. Calculations show that a test duration of at least 2 hours is required to enable reasonably accurate pore pressure measurements to be made. A transducer with very small volume change characteristics must be used.

(ii) Pre-Peak and Post-Peak Deformation Properties

Figure 4 shows complete loading curves, under uniaxial and undrained triaxial compression, of saturated specimens from a single block of sandstone. Figure 5 gives complete unconfined loading curves for specimens from Manly, saturated and dry, and for saturated specimens from block 6.

In the case of uniaxial compression it should be mentioned that once the load on the post-peak curve had reduced to about 80% of the peak value, considerable variation in the load displacement curves, for apparently identical specimens, was recorded. This is because by this stage the specimen is separated by facture zones into discrete volumes of essentially intact material. Thus the load-deformation behaviour is not a material property but is an expression of the energy interaction between the intact blocks and the narrow fracture zones. The post-peak curve will take on any shape depending on the volume involved in fracture relative to the volume of intact material. However, the early part of the post-peak curve for the uniaxial case is reasonably reproducible regardless of whether the test is run at constant rate of axial strain or lateral strain. In the case of triaxial compression the complete post-peak curve is reproducible simply because failure of the sandstone under the confining pressures used here always occurred with the formation of a single diagonal shear plane.

Although, as discussed above, it may be a difficult property to characterise, the brittleness of rock is of utmost importance. The ratio of the post-peak to the pre-peak stiffness provides a possible index for this property. Table IV gives this ratio, at a stress level of 80% of peak strength, from uniaxial and drained triaxial tests. It should be noted that in the case of undrained triaxial tests the post-peak stiffness is reduced as a result of the gradual decrease in pore pressure with increasing displacement (see Test 14, Figure 4).

(iii) Uniaxial Compressive Strength

Uniaxial strength and deformation data obtained from various sources are summarised, using the Deere-Miller type plot, in Figure 6. That many of the results fall in the low modulus region is because the secant modulus at a stress level of only 6MPa has been used. This modulus is more applicable to civil engineering problems than the more commonly used tangent modulus at 50% of uniaxial strength. Uniaxial strength data reported by O'Brien (1969) from the Warragamba dam (16 to 57 MPa) and by MacKenzie (1969) from the Opera House site (21 to

TABLE III

THEORETICAL DURATIONS FOR DRAINED AND UNDRAINED TESTS

Material	Permeability cm/sec	Secant modulus GPa	Poisson's ratio	Time for 95% equalization seconds	Time to failure for 95% dissipation seconds
Block 1	1.0×10^{-6}	2.0	0.4	3	190
Block 2	3.7×10^{-7}	4.0	0.4	4	240

47 MPa for soaked samples) fall within the range given in Figure 6. It is worth noting that, as shown in Figure 7, there is some correlation between the direct wave velocity (V_p) and the uniaxial strength of saturated specimens.

An aspect that has not been studied in detail here is the degree of anisotropy of the strength properties. Some results in this connection are available from boreholes drilled at different inclinations at sites in the Eastern Suburbs Railway tunnels. These data indicate that the uniaxial strength is the same for specimens cored normal and parallel to the bedding but can be reduced by a factor of anything between 0.2 and 0.8 for specimens cored at 45° to the bedding.

As already mentioned there is considerable difference between the dry and saturated strength of this sandstone. In order to investigate the degree to which this phenomenon is associated with increase in effective stress in the dry specimens, tests were conducted on specimens prepared at known suctions. Some specimens were conditioned by allowing them to dry out very slowly while using psychrometer probes (Richards, 1965) to measure the total suction. Other specimens were stored at controlled humidities until they reached constant weight and the suction then calculated (Richards, 1965). The relationship between suction and degree of saturation was determined as shown in Figure 8. It is important to note the rapid increase in suction as the degree of saturation drops below 20%.

The specimens prepared to known suctions were tested in uniaxial compression in the standard manner (ISRM, 1972) except that they were sealed in plastic to inhibit moisture change during the testing process. Making the assumption that the strength gain of dried specimens is only due to pore water suction one can calculate the χ values necessary to match the uniaxial strengths of dry specimens to the failure envelope for saturated specimens. These values are given in Figure 9 which shows that while these values are outside the range where idealized models and soil mechanics data indicate possible values, they are not unreasonable. Invoking the

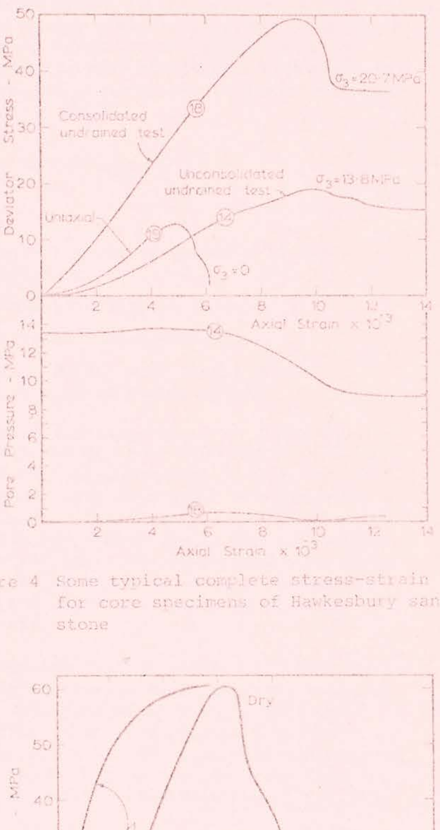

Figure 4 Some typical complete stress-strain curve for core specimens of Hawkesbury sandstone

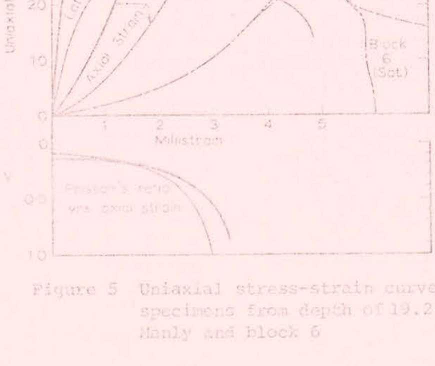

Figure 5 Uniaxial stress-strain curves-specimens from depth of 19.2 m, Manly and block 6

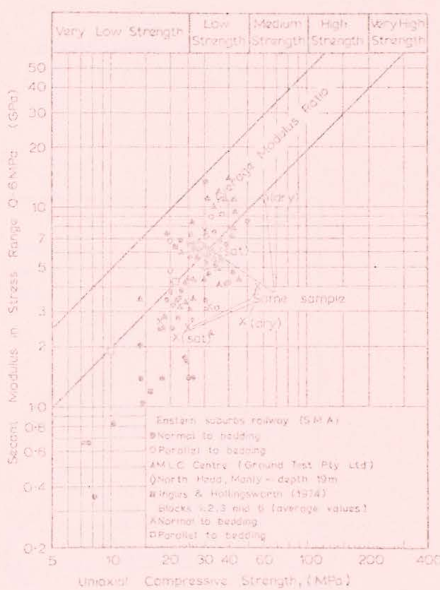

Figure 6 Summary of strength and modulus data for Hawkesbury sandstone

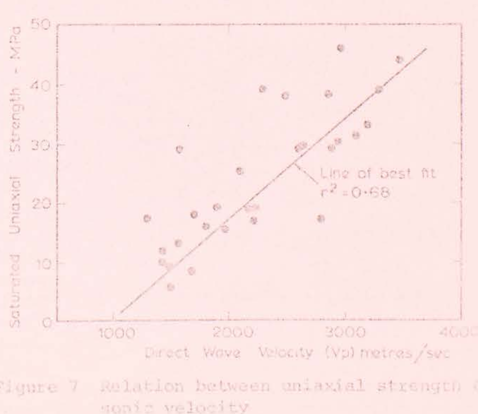

Figure 7 Relation between uniaxial strength and sonic velocity

principle of effective stress thus does provide a reasonable explanation of the strength gain associated with dessication but a more detailed study is required to determine to what extent chemomechanical factors, as discussed earlier, may also be influencing this behaviour.

In discussing triaxial test data in terms of effective stresses it is best to make use of Skempton's equation for pore pressure changes due to total stress changes, namely

$$\Delta u = B\Delta\sigma_3 + BA(\Delta\sigma_1 - \Delta\sigma_3)$$

where $\Delta\sigma_1$, $\Delta\sigma_3$ = changes in the principal stresses

B, A = Skempton's pore pressure parameters. (Note $AB = \bar{A}$)

When there is no change in the deviator stress the pore pressure change within a specimen depends only on the parameter B, which is a function of the relative compressibility of the solid skeleton to the pore water, as given by the following equation (Bishop & Henkel, 1962).

$$B = \frac{1}{1 + n\, C_w / C_c}$$

where

n = initial porosity (about 0.15 for Hawkesbury SST)

C_w = compressibility of water

C_c = skeleton compressibility

$= 3(1 - 2\nu)/E$ (.18 to 2.4 m^2/GPa depending on stress level)

Thus the theoretical value for B at low effective confining pressures is about 0.95 but with increasing effective confining pressure it will decrease rapidly to values less than 0.3. The parameter A is equal to 0.33 in the elastic range but generally cannot be evaluated theoretically due to the non-linear volumetric behaviour associated with shear strains. It can be expected to vary considerable depending on the effective confining pressure and the shear strain.

Figure 10 shows the typical undrained pore pressure response, with increasing cell pressure, measured on saturated specimens of Hawkesbury sandstone. The high initial value of B is partly due to membrane penetration and not just skeleton compressibility, but the pore pressures are very real and are of great importance when considering triaxial

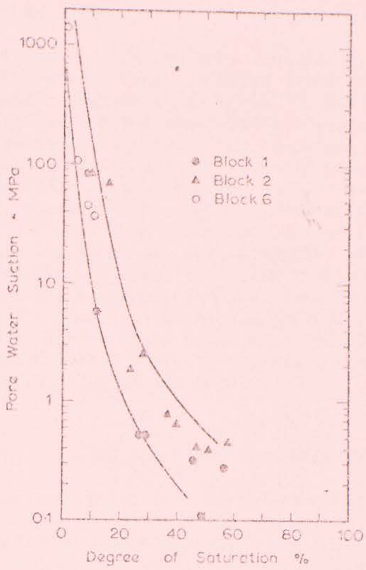

Figure 8 Relationship between suction and degree of saturation

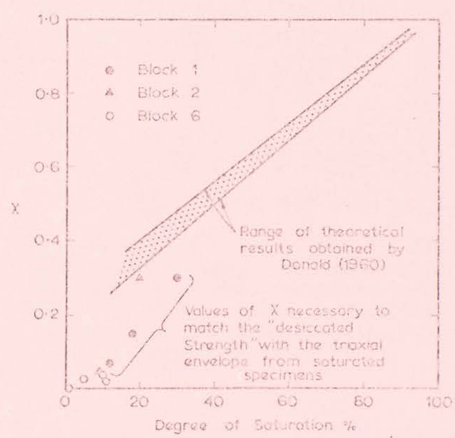

Figure 9 Relationship between χ factor and degree of saturation

TABLE IV

BRITTLENESS MODULAR RATIOS* (BMR) AT STRESS LEVELS OF 0.8 PEAK DEVIATOR STRESS (54MM ϕ × 110MM LONG SPECIMENS)

Specimen	Effective Confining Stress MPa	Tangent Modulus at 80% of Peak Stress GPa	BMR*
6 - 8	0	9.7	-2.7
6 - 16	0	5.0	-1.8
6 - 18	20.6	5.1	-3.6
6 - 19	0	4.0	-4.6
6 - 25	10.3	5.0	-14.3
Manly(sat)	0	16.0	-9.4
Manly(dry)	0	22.1	-3.6

* Brittleness Modulus Ratio $= \dfrac{\text{falling slope of load deformation curve}}{\text{loading stiffness}}$

tests conducted on saturated specimens, where no consideration is given to drainage. Often the very dramatic curve in the Mohr envelope obtained from triaxial tests on rock arises from ignoring the pore pressure created during application of the confining pressure. Interpretation of unconsolidated undrained tests, such as shown in Figure 4, in terms of total stress yields a very strongly curved Mohr envelope which is simply approaching the $\phi = 0$ condition and has nothing to do with the curved envelope predicted by the Griffith criterion.

Typical pore pressure response to change in deviator stress is shown in Figure 4. Such results have been used to determine the relationship between $\bar{A}$ at failure ($\bar{A}_f$) and effective confining pressure shown in Figure 10b. This behaviour is very typical of an overconsolidated or dense soil. This similarity also holds for the pore pressure behaviour on cyclic loading (vide Figure 11). As in the case of dense sand (Rowe, 1975) the maximum pore pressure does not coincide with the stage of maximum cyclic shear stress and dilatency occurs between points C and D to increase the strength.

Peak shear strength parameters were determined for six different blocks of sandstone. For blocks 1 to 5 drained triaxial tests were used while undrained tests were used for block 6. Failure in these triaxial tests took the form of sliding on a single "plane" and in those tests conducted in the stiff testing machine it was possible to determine the strength parameters for sliding on such a shear induced fault.

The test results are summarised in Table V. These results indicate an increase in the values of c' and ϕ', obtained from drained tests, with increasing uniaxial strength. The undrained tests on block 6 yielded a lower value of ϕ' than would be indicated by the drained tests on the other five blocks.

Also shown in Table V are the values of ϕ predicted using the empirical equation for sandstone described by Bieniawski (1974). This equation seems satisfactory in that it is reasonably conservative for the Hawkesbury sandstone.

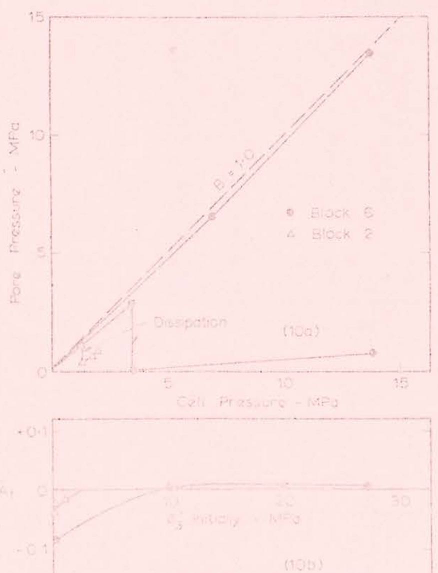

Figure 10 Pore pressure behaviour of saturated specimens under triaxial test conditions

(v) Creep

This aspect has not been considered in detail in the present study. O'Brien (1969) reports rather vaguely that this can amount to 50% of axial strain. This was not the case with tests conducted in this investigation where a few specimens were loaded to between 50% and 95% of their peak strength and then allowed to creep for up to 2 days. However, tests reported by the Snowy Mountains Authority (1969) show that creep can be significant in specimens loaded at low stress levels, as shown in Table VI. At such low stress levels pores in the rock are still closing and probably a fair portion of the load is transmitted through the clay matrix. Thus greater creep would be expected than at high stress levels where the quartz grain skeleton carries much of the load.

5 TENSILE AND POINT LOAD STRENGTH

Diametral point load tests were conducted, as outlined by the ISRM (1972), on saturated and oven dry specimens from blocks 1, 2 and 6. The specimens from blocks 1 and 6 were cored normal to the bedding. The block 2 specimens were cored paralled to the bedding and the specimens split diametrically when tested normal to the bedding but split longitudinally at somewhat lower loads when tested parallel to the bedding.

The results of these tests are given in Table VII which also gives data from Brazilian tests conducted on block 6. The empirical equation

$$\sigma_c = 24\ I_s$$

where

σ_c = uniaxial strength

I_s = point load index for NX specimens

gave in this case consistently low estimates of the unconfined strengths. It must, however, be noted that relatively homogeneous isotropic blocks of sandstone were used here and these results cannot be generalized to cases where the sandstone is strongly bedded.

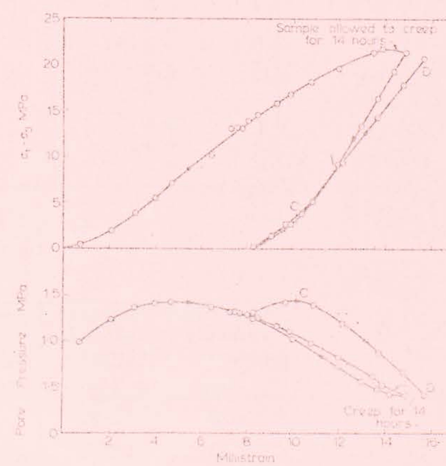

Figure 11 Pore pressure response in cyclic undrained triaxial test (σ_3 = 1.95 MPa) block 2

6 SHEAR STRENGTH OF ARTIFICIAL JOINTS

Artificial joints were formed across 100 mm square blocks, either by tension fracture to produce a 'rough' joint, or by diamond saw to yield a smooth joint. Surface profiles were measured with attention being paid to irregularities with a base length of 10 mm or greater. The average roughness angle on a 10 mm base length for the tension joints was 5°.

Figure 12 gives the failure envelopes obtained for both joint types tested air dry. The smooth joints yielded the so called 'basic friction angle' (Hoek and Bray, 1974) of 34.5° while the rough joints yielded a 'cohesion' intercept of 0.44 MPa and a friction angle of 40°. The fact that the rough friction angle equals the basic value plus the measured roughness angle was probably fortuitous in this case. Considerably more work is required before a satisfactory method is established for predicting the effective friction angle of rough joints in Hawkesbury sandstone on the basis of field roughness angle measurements and laboratory tests. However, the data given here do give some idea of the properties of fresh fractures in this sandstone.

7 IN-SITU DEFORMATION PROPERTIES

While laboratory test data are of value is designing structures in, or on, a generally massive rock like Hawkesbury sandstone it is clearly the in-situ properties that are of ultimate concern. This applies particularly to the in-situ deformation

TABLE V

Block No.	Uniaxial strength	Peak shear strength		'Residual' strength		Predicted* peak
		c'	ϕ'	c_r	ϕ_r	ϕ
	MPa	MPa	degrees	MPa	degrees	degrees
1	18.0	3.7	48	1.2	39	43
2	11.7	2.4	45	–	–	41
3 & 5	21.5	3.9	49	–	–	44
4	32.5	5.0	53	–	–	46
6 **	25.0	6.0	41	2.0	36	45

** Undrained tests (all other data from drained tests)

* Using equation $\dfrac{\tau_m}{\sigma_c} = 0.75 \left[\dfrac{\sigma_m}{\sigma_c}\right]^{0.9} + 0.1$ (Ref. 4)

TABLE VI

Uniaxial Strength	Stress range	Short term secant modulus	Number of days creep	Ratio Long term modulus / Short term modulus
MPa	MPa	GPa		
8	1.4	0.2	21	0.9
9	1.0	1.4	10	0.8
13	1.4	1.4	13	0.8
19	1.0	1.4	26*	0.5
22	1.0	3.5	20	0.8
24	1.0	0.7	41*	0.5
26	1.0	2.4	42*	0.6
28	1.4	10.7	19	0.9
28	1.0	7.2	10	0.9
35	1.0	6.9	7	0.9
45	1.4	6.2	8	0.9

* Creep still continuing

TABLE VII

Block No.	Conditions	Brazilian strength	Point load strength Index (I_s)	Predicted uniaxial strength	Measured uniaxial strength
		MPa	MPa	MPa	MPa
1	sat		0.64	15.4	18.7
2	sat		0.37	8.9	11.7
	dry		1.60	38.4	39.2
6	sat	0.7	0.8	19.2	25.0
	dry	1.7			

modulus which may be quite different from the laboratory value. The plate loading tests conducted by the Snowy Mountains Authority in the Eastern Suburbs Railway tunnels provide useful data in this regard. The plates used varied from 200 mm to 430 mm diameter so that the volume loaded was comparable to, say, an end bearing pile. Table VIII summarises the data obtained in the tunnels. The moduli calculated from the plate loading tests would be lower than the true values due to surface distrubance but higher than the true values because, although conducted in a tunnel, the tests were interpreted using the solution for a plate load on a half-space. If it is assumed that these factors cancel one another then the results in Table VIII indicate that the in-situ modulus can be estimated using a reduction factor of 0.65 to 0.8 on the core modulus at the appropriate stress level. As the rock masses at the test sites have Geomechanics Classification Ratings (Bieniawski, 1975) of 65 to 70 these data show a much smaller reduction to be applicable than suggested by Bieniawski. This highlights one of the flaws in Bieniawski's system as the Geomechanics Rating is influenced by both the uniaxial strength and the water conditions, factors which have no bearing on the relationship between field and laboratory modulus.

8 IN-SITU SEISMIC VELOCITY

The velocity of direct waves through a rock mass is used in the estimation of rippability while the ratio of this field velocity to the laboratory value measured on core has been used as an index of rock quality (Deere et al, 1966).

Some initial studies of these seismic velocities have been made at three widely spaced locations in the southern zone of the Hawkesbury formation. Cores were taken, in extensive surface exposures, down to depths of 15 metres. These were logged in terms of RQD and then laboratory velocities were measured at approximately 0.2 m intervals. At all three sites the RQD averaged about 90%. In-situ velocity measurements were made using a hammer seismograph with source to geophone spacings up to 50 metres.

At the first two sites the water table was at the surface and the field velocities of 2700 m/sec and 2800 m/sec were close to the average laboratory values. The third site on a low ridge, probably well above the water table, yielded a velocity of 1700 m/sec (about 0.6 of the laboratory value). While many more results are needed before a general statement can be made it does appear that the seismic velocity ratio is too dependant on the degree of saturation of the rock mass to be a reliable indicator of the degree of jointing in the sandstone layers.

With regard to the evaluation of rippability it should be noted that dry sandstone yields a lower seismic velocity than the same rock saturated. Yet the dry material is more difficult to rip due to its significantly greater strength. Thus the seismic velocity is not a sufficient indication as to rippability and consideration should be given to including other rock properties in this evaluation as suggested by Weaver (1975).

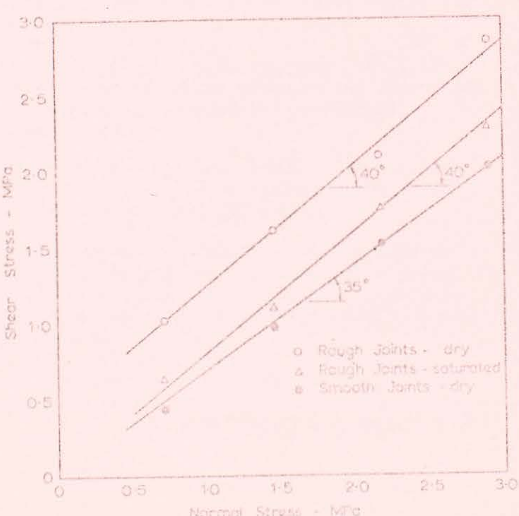

Figure 12 Shear strength measurements on artificial joints

9 CONCLUSIONS

An examination has been made of the strength and deformation properties of Hawkesbury sandstone, with particular regard to the principle of effective stress. It is shown that, in many respects, the sandstone behaves like an overconsolidated or dense soil.

The considerable strength difference between dry and saturated specimens appears to be largely due to the effect of high negative pore pressure in dry specimens but chemomechanical factors probably also play some role in this phenomenon.

In triaxial tests on saturated specimens high pore pressures may develop on application of the cell pressure. Unless this factor is considered complete erroneous failure envelopes may be obtained. In drained tests the rate of application of the deviator stress is not as critical as ensuring full drainage of the pore pressures induced on application of the cell pressure.

The load deformation curve in the post-peak region is not a unique property of the sandstone because the specimen is separated by fractures into discrete volumes of intact material which are essentially elastic. The load deformation behaviour is thus a function of the specimen geometry, independant of specimen end conditions, as it is simply an expression of energy interaction between the intact blocks and

TABLE VIII

Test site	Location	Field modulus E_f GPa	Laboratory modulus E_ℓ GPa	Ratio E_f/E_ℓ
1	Roof	1.5	2.4	0.6
	Floor	3.2	4.5	0.7
2	Roof	0.6	0.6	1.0
	Floor	2.1	3.8	0.5
3	Horizontal-south	1.8	3.1	0.6
	Horizontal-north	1.7	3.4	0.5
4	Horizontal-south	2.5	5.9	0.4
	Horizontal-north	2.2	4.1	0.5

the narrow fracture zones. The post-peak deformation behaviour would only be a unique material property in a material which remained homogeneous at all stages. Whether a homogeneous brittle behaviour exists in reality is doubtful.

The Mohr envelope in terms of effective stresses is close to linear for confining pressures up to 30 MPa. Values of ϕ' in excess of 40° were obtained on all samples tested, with drained tests yielding values at least 5° higher than those from undrained tests. The angle of friction of tension fractures in fresh sandstone was found to be about 40° while smooth pre-cut joints have friction angles of 35°.

A survey of available data indicates that the uniaxial strength of most Hawkesbury sandstone falls in the range of 20 to 50 MPa. The material has a low modulus ratio at stress levels of up to 6 MPa and is anisotropic. Permeability appears to be in the region of 0.1 to 0.3 m/year but overall permeability is, as with all rock masses, controlled by the jointing.

In estimating the in-situ deformation modulus of massive layers of this sandstone it seems that 0.5-0.8 times the core modulus (at the correct stress level) should be used.

10 ACKNOWLEDGEMENTS

The work described in this paper forms part of a general program of research into the deformation of soil and rock masses being carried out at the University of Sydney under the general direction of Professor E.H. Davis. The work is supported by a grant from the Australian Research Grants Committee.

The assistance of Mr. K. Larymore in connection with much of the testing described here is acknowledged.

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Our Ref: AK 185

CONFIDENTIAL

Wells - Green

14th July 1980

The Chief Engineer,
Auckland Harbour Board,
P.O. Box 1259,
AUCKLAND.

Dear Mr Seagar,

Re : Ferry Building

I have, as agreed, had meetings on this with the Auckland City Design Engineer (Don Leadbeater) and (separately) with the District Structural Engineer, Ministry of Works (Ian Armstrong).

Attached is a copy of a letter dated 9th July from Don Leadbeater. Don is particularly conscious of the "...difference in thinking..." mentioned in his Paragraph 2 which seems to have been identified in the course of some of ACC's own loan applications. At the same time the attitude expressed in his Paragraph 3 assumes that his sole role is that laid down in 5301A of the MC Act. His responsibilities would change if Council became financially involved in the Ferry Building under the terms of its own District Scheme. Further discussion centred on the technical feasibility of our own proposals for prestressing the existing masonry. I think that a fair concensus of that discussion would be that the approach could well be valid and appropriate. However it does go outside existing New Zealand experience and practice and it therefore requires substantial and detailed investigation to ensure that it is soundly based.

I have some nine pages of handwritten notes on matters discussed with Ian Armstrong. Although extensive they are not particularly specific and it is clear that Ian does not want to be drawn into too detailed a level of discussion until our investigations have gone very much further. He emphasises that the discussions are entirely of a private nature until such time as his office is formally consulted by the Local Body Loans Board. Further he makes it clear that any official detailed review would require as input very complete documentation related to:-

* Investigation of ALL alternatives and a statement of available options.

* Future use of the building. Ownership and responsibilities both legal and moral. Objectives of strengthening. Role of ACC and ARA?

* Detailed evaluation of "Historic Building" status. Is the National Historic Places Trust interested? Specific and detailed indentification of those features essential to preservation of historic character.

Associated Offices at:- P.O. Box 1074, Christchurch, Phone: 67-953
243 Sussex St., Sydney N.S.W. Phone: 29-2071.

.../2

* Whether the historic fabric of the building can survive a "damaging" earthquake in a repairable state.

* Detailed description of the building "as existing" including, particularly, the foundations and the soils with reference to microzone effects.

He looks for a report which considers a number of different schemes each of which are capable of supplying a number of different degrees of improvement. He avoids statements which might be taken as establishing policy, but I suspect that if the Board's objectives are predicated on the historic merit of the building, then he will be looking for a high degree of protection for the "historic features". Perhaps he wants to consider an earthquake with a return period of 300 to 500 years. I have some doubt that this would be feasible technically at whatever cost. However he does invite me to take up the seismological aspects with his own head office (Glogau) and/or with DSIR Wellington (Skinner).

He asks for very detailed studies on "...the practical and technical adequacy of prestressing brickwork..." and on the performance of such a material in relationship to the objectives stated in the above paragraph. He does not seem to think that the quality of the existing masonry is a major factor in this regard.

He mentions the present requirements for closely spaced horizontal steel in new buildings and regards this as a matter of some importance. He notes that the relevance of the Danish research (see our own report) to earthquake situations has not be established.

IN CONCLUSION : It is clear that Ian looks for very much more detailed assessment on a number of matters. Several of these involve aspects which could reasonably be described as research of a fairly basic nature. Research matters within MOWD are handled by head office and it would now be quite proper for us to take these aspects up with them directly. This would require a one/two day visit to Wellington, but I think that it would be premature to do this until some progress has been made in sampling and testing the existing brickwork. It would be timely too to start assembling available recorded data on the existing foundations and soils. Some relevant data may be available from the construction of the Air New Zealand Building as well as the records relating to the Ferry Building itself and to adjacent harbour Works.

We now await your further instructions.

Yours faithfully,

C. Gurley
WARGON, CHAPMAN & GURLEY

ALEXANDER WARGON, MSc. CE, MNZIE, FIE Aust, FASCE
ROBERT F. CHAPMAN, BE, ASTC, MNZIE, FIE Aust
COLIN R. GURLEY, MEng. Sc., BE (Hons.), MNZIE, MIE Aust, MASCE

WARGON CHAPMAN AND GURLEY
REGISTERED ENGINEERS
CONSULTING STRUCTURAL & CIVIL ENGINEERS

27 SYMONDS STREET, AUCKLAND, N.Z. PHONE 797-584

11 March 1980

The Chief Engineer
Auckland Harbour Board
Box 1259
AUCKLAND

Dear Sir

re : Ferry Building

We have Mr Goord's letter of 20.12.79 and we attach copy of DSIR report on one of the samples of reinforcing wire taken from the floor slabs. The wire is 5 mm diameter and seems to be ordinary hot rolled mild steel. One naturally expects a moderately high yield point in such fine rolled sections as compared to larger sections of the same material.

The existing floor slabs are quite lightly reinforced. Furthermore the details of the timber super floor observed with Mr Goord just before Christmas lead us to conclude that the slabs were intended only as self supporting fire resistant ceilings and that the timber floor was intended to carry occupancy live loads directly to supports.

The slabs may have some unutilised gravity load capacity over and above their own weight and this can be assessed by modern analytical techniques once the layout of supporting rolled steel beams is known. It is quite likely, however, that such an assessment will still conclude that the slabs cannot, on their own, safely carry normal occupancy live loads. This simply means that the existing timber floor has a continuing structural gravity load junction.

A more serious concern is the credibility of the slabs as structural diaphragms and, in particular, the ability of slabs to receive earthquake fall loads from the very heavy facades and transmit these into the primary earthquake resisting structure. You may like to think of this as akin to the construction of a plate web girder with a thin weak web. Such an element could normally be made to work if the web was provided with stiffeners at all major load attachments so that loads were distributed uniformly into the body of the web and not allowed to concentrate and cause local failures.

In this case the web stiffeners could take the form of spaced reinforced concrete strips or bands running across and bonded to the top of the existing slabs. Such strips would terminate at structural elements used to brace the facades. The design and construction of the strips would need to have regard for the gravity load function of the existing timber super-floors. We do not see any particular problems in such an approach other than the further cost implication.

The layout of the rolled steel floor beams needs to be ascertained before much further progress can be made and we would like to press on with this as soon as possible.

Yours faithfully

Colin Gurley.

WARGON CHAPMAN AND GURLEY
Colin R Gurley

DEPARTMENT OF SCIENTIFIC AND INDUSTRIAL RESEARCH
AUCKLAND INDUSTRIAL DEVELOPMENT DIVISION

REPORT

MCS:MDB

ON

FERRY BUILDING REINFORCING STEEL

CLIENT:

CLIENT'S REFERENCE:

A.I.D.D. REFERENCE: 90/1370

Wargon Chapman and Gurley,
27 Symonds Street,
AUCKLAND 1.

Attention: Mr C. Gurley

INTRODUCTION

This Division was requested to examine a piece of reinforcing rod taken from the Ferry Building in Auckland. Tests were done to determine the mechanical properties on the wire submitted.

INSPECTION

The wire sample was tested on an Instron tensile testing machine. The results are as below. The yield point can only be taken as being an approximate value.

Yield Point tons/in^2	Ultimate Tensile Strength tons/in^2	Reduction Area
21.5	37.2	approximately 40%

These tensile tests were confirmed by hardness tests done by a Vickers hardness machine using a 10 kg load which indicated a tensile strength of approximately 40 tons/in^2. The hardness tests were carried out on a micro specimen of the wire which had previously been prepared for metallographic examination. The micro structure revealed that the wire was a low carbon steel with small areas of fine pearlite in a ferrite matrix. The structure indicated that the wire had been hot rolled.

"Specimens submitted in connection with this report will be disposed of unless collected within 8 weeks of the date below."

DATE: 3 January 1980

M.C. Sanders

EXAMINING OFFICER.

CHECKED BY R Thomas

[illegible] WARGON, MSc. CE, MNZIE, FIE Aust, FASCE
[illegible] F. CHAPMAN, BE, ASTC, MNZIE, FIE Aust
COLIN R. GURLEY, MEng. Sc., BE (Hons.), MNZIE, MIE Aust, MASCE

WARGON CHAPMAN AND GURLEY
REGISTERED ENGINEERS
CONSULTING STRUCTURAL & CIVIL ENGINEERS

27 SYMONDS STREET, AUCKLAND, N.Z. **PHONE 797-584**

3rd June, 1980.

The Chief Engineer,
Auckland Harbour Board,
P.O. Box 1259.
AUCKLAND

Confidential

Dear Mr. Seagar,

Re: Ferry Building

I refer to our discussions of Thursday 29th concerning sections 1 to 5 of our report.

Our attitude is that the report establishes a decision/design framework which poses the major relevant questions and provides our own current best estimate of the answers.

It provides a basis for technical negotiation with the City Council and with MOWD. In the event that such negotiations are unfavourable then it indicates the probable direction in which proposals would need to be modified.

The City Design Engineer (Don Leadbeater) has just returned from a trip to the USA to assess attitudes there on historic buildings and he will be looking at our Ferry Building report next week.

The District Structural Engineer MOWD (Ian Armstrong) is on leave till Tuesday June 3. He has indicated that he will discuss the report with me on a personal and informal basis but that he will not make any written reply without a letter from you to the effect that an application to the Local Body Loans Board is "foreseeable".

Neither of these men are likely to commit themselves very far until some clear conclusions emerge from a coring, sampling and testing program. My understanding is that you want at this stage to establish that the cost of such a program is warranted.

One likely contentious issue will be the application of the Danish theories on shear strength to masonry subject to earthquake loads. It will be necessary to check on this by reviewing the Danish approach in the

Contd:-

light of the existing body of experimental data on shear walls tested in New Zealand. Perhaps some further testing will be necessary. This would be basic research with applications going well beyond the Ferry Building and it therefore seems unreasonable to suggest that it should be funded by the Harbour Board. We will use our best efforts to have it taken up by others. In terms of the Ferry Building the issue at stake is the need for "shotcrete" curtains covering internal walls.

You have noted that we have not looked at the foundation situation. Logically proposals for foundation strengthening should be based on the outcome of superstructure proposals. It will be necessary, first of all, to compile an "as existing" drawing based on the best available historical information. It seems that some of the old Ferry Building documents may be "Alternative Proposal" documents and we are not clear as to what was actually built. Your office also has some earlier documents describing civil structures under or close to the Ferry Building. It is physically clear that there has been some settlement damage. The historical data is likely to need to be supplemented by test bores or pits.

We note however that soft (Waitemata) rock is believed to be only 4 to 5 m below Queen St. and that the superstructure investigations so far do not seem to predict any major foundation uplift problems. It is quite likely that some new piling will be required to take overturning compression and/or to correct the settlement problem. It may be that there are unsound areas or cavities in the old structures which require drilling and grouting. Finally it will be necessary to look at the adequacy of the Ground (Quay St.) floor slab and high level footings considered as a structural diaphragm.

Overall, however, our present impression is that foundation problems are likely to be substantially less critical than issues with the superstructure and that it would be premature to initiate detailed foundation investigations until some clear direction emerges with the superstructure.

Yours faithfully,

Colin Gurley.

Colin Gurley

for WARGON CHAPMAN AND GURLEY

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27 SYMONDS STREET, AUCKLAND, N.Z. PHONE 797-584

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The Chief Engineer,
Auckland Harbour Board,
P.O. Box 1259,
AUCKLAND

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1. Analysis

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* Base building at Quay St. level : 1.4 to 1
* Tower at Main Roof level : 3.7 to 1

These results do not surprise us. They confirm that the dynamic characteristics of the tower are quite different to those of the base building and, therefore, that seismic stresses, particularly at the junction of the two, can only be realistically estimated by dynamic computer studies. Such studies can be quite expensive and, in our view, it would be premature to commission them until there is some committment to strengthening of the building based on definite structural proposals.

Floor levels in the building and tower have been measured by your surveyors and the third floor has been identified as a reinforced concrete slab with a timber superfloor. From these investigations it is clear that the original architectural drawings cannot be relied on to be entirely accurate. Presumably changes authorised during construction would have been recorded in a "Site Instruction Book". We have had no success in locating such a book through your own records, through the Wiseman family or through various historical sources.

2. Los Angeles Reports

The material collected by Derrell Goord during his recent visit is most helpful particularly the very extensive and well-documented back-ground material on the proposed L.A. City Ordinance on "Earthquake Hazard Reduction in Existing Buildings". It is clear that the committee drafting that ordinance included a wide range of structural engineers from private and public sectors together with representatives of building owners, the community and other professions. Several of the engineers are known to us personally or by reputation and we believe that the material represents a well-informed and well-documented attempt to define some sensible and honest compromise between hazard reduction, on the one hand, and economics on the other.

By way of comparison we would estimate that the L.A. provisions are less severe than (say ½ to ⅔ as severe as) the current standard (NZS4203) applied to the construction of new buildings in Auckland. On the other hand it seems to be significantly more severe than the discretionary standards sought by Auckland City Council for reconstructed old buildings. The latter difference is not large enough to be surprising and, in part at least, it reflects the lower seismic risk exposure in Auckland.

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The first major issue is that of the overall strength of the building particularly at Ground Floor level where many major bearing walls are arched out (or beamed out) to provide collonades, shops etc.. Sketch SK5 shows nett shear areas and correlates seismic base shear with a range of brick shear strengths.

Council's letter dated 8 May 1979 implies the view that the Ferry Building does fall within the terms of S301A of the Municipal Corporations Act. To the extent that this raises a legal issue we do not think we can comment with any degree of certainty. To us the wording of the statute is so vague and technically imprecise that any Court sitting on a relevant matter is likely to have to contend with a very wide range of expert testimony. We can say that Council's view seems reasonable on technical grounds. Shear cores have yet to be taken from the existing building but it seems unlikely that the building, as it now stands, would even meet one-half of the L.A. standards.

We have also looked at various interpretations (of S301A) published by senior engineering staff of Christchurch City Council. The Ferry Building does not appear favourably in the light of those interpretations.

Auckland City Council has a discretionary power (under S301A) in respect of the standards to be adopted in strengthening buildings. They have advised that, in general, they will look for the standards of NZSS 1900 i.e. the standards applied to new buildings in the late 1960's. We have already noted that, for the Auckland area, these standards are somewhat less severe than the L.A. standards and a good deal less severe than the current standards for new buildings in Auckland. As in L.A., the underlying philosophy is to reduce danger to life without necessarily attempting to protect the buildings concerned from damage.

One relevant requirement of NZSS 1900 is that all masonry be reinforced except only for infill panels enclosed by structural frames. There may be some discretion, on Council's part, as to the maximum centres of reinforcing steel to be provided but, in any case, the totality of steel required under this provision may well be more significant than that required to achieve any specific overall lateral strength.

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4. Existing Floors

The specification for the building (page 22) provides for an "alternate Tender" utilising:-

* Reinforced floors of cement shingle concrete (1-4-1) 6" thick with
* No. 6 gauge bright crimped wire lattice 3" mesh and
* No. 26/8" x 6" x 35lbs. R.S. joists to each of 1st, 2nd and 3rd floors upturned and encased in concrete.

The wire gauge is, presumably, B.S.W.G. in which case No. 6 gauge corresponds to 0.192 inch diameter. This would give a steel area of 246 mm^2/metre width or 0.16% of gross concrete area. This is fairly low compared to minimal current requirements of 0.25% of gross area for mild grade steel. It implies, first of all, that some care will be necessary in transmitting concentrated forces into the slab regarded as a diaphragm.

The spec. gives no information on the plan location of the R.S. joists. I have sighted one such joist running N-S on the third floor. The best guess seems to be that they run N-S so that the slabs span E-W and are supported at up to 9'6" maximum centres. If this is so, then, depending on the yield strength of the steel and on the placement (at midspan and over supports) the slabs may be marginal under service gravity loads even by modern ultimate strength criteria.

We recommend that the slab be opened up at, say one or two locations on each of the major floors to check the mesh actually used, to check consistency and placement and to take samples for mechanical testing. If the wire is hard-drawn then it may well be high tensile. Mr. Neville Miller of the AIDD Division of DSIR (phone 34-116) is prepared to be present to advise on the sampling and testing arrangements.

We would like also to check on actual plan locations of the RS joists wherever possible without disrupting occupancies or damaging floor coverings.

5. Tower

The precise strength requirement of the tower at about existing main roof level depends, in part, on the dynamic analysis mentioned under Paragraph 1. However, there is a need for some vertical reinforcement (perhaps post-tensioned) at the four corners of the tower to provide ductile bending strength and the precise amount may not be particularly critical in drawing up an initial budget. Of rather more concern is that the tower north wall is discontinued just below existing roof level and that there is no structural connection between the tower and the remainder of the building above third floor level. Thus:

* For N-S earthquake the tower cantilevers from 3rd floor effectively as a U-shaped section and
* For E-W earthquake the tower will be subject to significant torsional problems because of the lost shear-strength of the discontinued wall.

It seems essential then to replace the lost shear strength of the north wall above third floor level. It may be possible to go further and to effectively support the tower (considered as a vertical cantilever) at main roof level and to integrate a solution to this problem with that of paragraph 6 below. It will be necessary to look at strength of connections to existing and/or new diaphragms at main roof and third floor levels.

6. Existing Parapets

Council's letter names, as "particular hazards":-

* The 4'8" high perimeter parapet and
* The 5' 6" high ornamental gables to Quay St.

The latter, we think, means the heightened rectangular elements directly over the two main arched public Quay St. entrances each side of the tower. These elements are much heavier than the typical parapets - the flanking sub-elements, in particular, are 3'9" thick. In fact the building is a long narrow one and the existing roof structure does not seem to be a very effective diaphragm so that these elements may behave as vertical cantilevers above third floor level i.e. to a cantilever height of about 22 ft. Council is, in our view, correct to be particularly concerned about this matter although we would not necessarily consider the parapet problem more immediately critical than the tower torsion problem of the previous paragraph 5. It would be possible to correct this particular problem by short-term solutions providing structural steel portal frames across the building and/or steel bracing tieing the parapets back to third floor. We would prefer to resolve this problem within the overall problem and, in particular, to integrate it with a solution to the tower torsion problem.

7. Facades

The existing facades are very heavy and quite intricate. Whether or not, by some addition of reinforcement, they are to be used as major lateral load elements it will, at least, be necessary to tie them back to major structure to prevent them falling out of the building and endangering life in Quay St. particularly. This is likely to be a major exercise in its own right and one in which it may be quite difficult to find some reasonable compromise between the historic value of the building, on the one hand, and earthquake hazard reduction on the other. There seems no doubt that the existing facades of the building constitute almost all of its historic significance and that any drastic alteration to the external appearance of the building would destroy that historic significance.

8. Foundations

Our understanding is that most of the perimeter of the building is supported on massive concrete walls which go down from just below Quay St. level about 14 ft. to rock. The details are to be found, not on the Ferry Building drawings themselves but on various other sets related to adjacent quay and wharf construction. This matter seems less immediately critical and we have therefore, for the moment, concentrated on problems related to the superstructure above Quay St. level.

The interior of the building appears to have been supported on timber piles and there is clear evidence of settlement cracking:-

* In the arches of the E-W collonade on the north side of the building and
* In the N-S bearing walls north of the corridor on the north side i.e. directly over the collonade arches.

The second of these seems to explain Council's comment concerning "masonry bearing walls which are badly cracked at first floor level" although we do not think that we would have used the word "badly" in this context. Certainly, however, there are foundation settlement problems and, if any further substantial investment in the building is to be made then they will have to be rectified. It only makes sense to do so in co-ordination with foundation strength requirements established as an outcome of proposals worked out for strengthening the superstructure.

9. Further Action

The next steps are:-

* To initiate sampling and testing of the existing masonry and concrete slabs and to determine, so far as now possible, plan locations of R.S. joists.

* To evolve and secure Council's agreement in principle, proposals for the minimum amount of work necessary to tie various parts of the building together so that the building at least acts as a whole unit in an earthquake.

The second step includes such matters as:-

* Minimum "basketing" reinforcement of masonry generally.

* Pinning back of the facades.

* Reliable gravity load support to R.S. floor joists.

* Diaphragm action at main roof level and tower/ building connections.

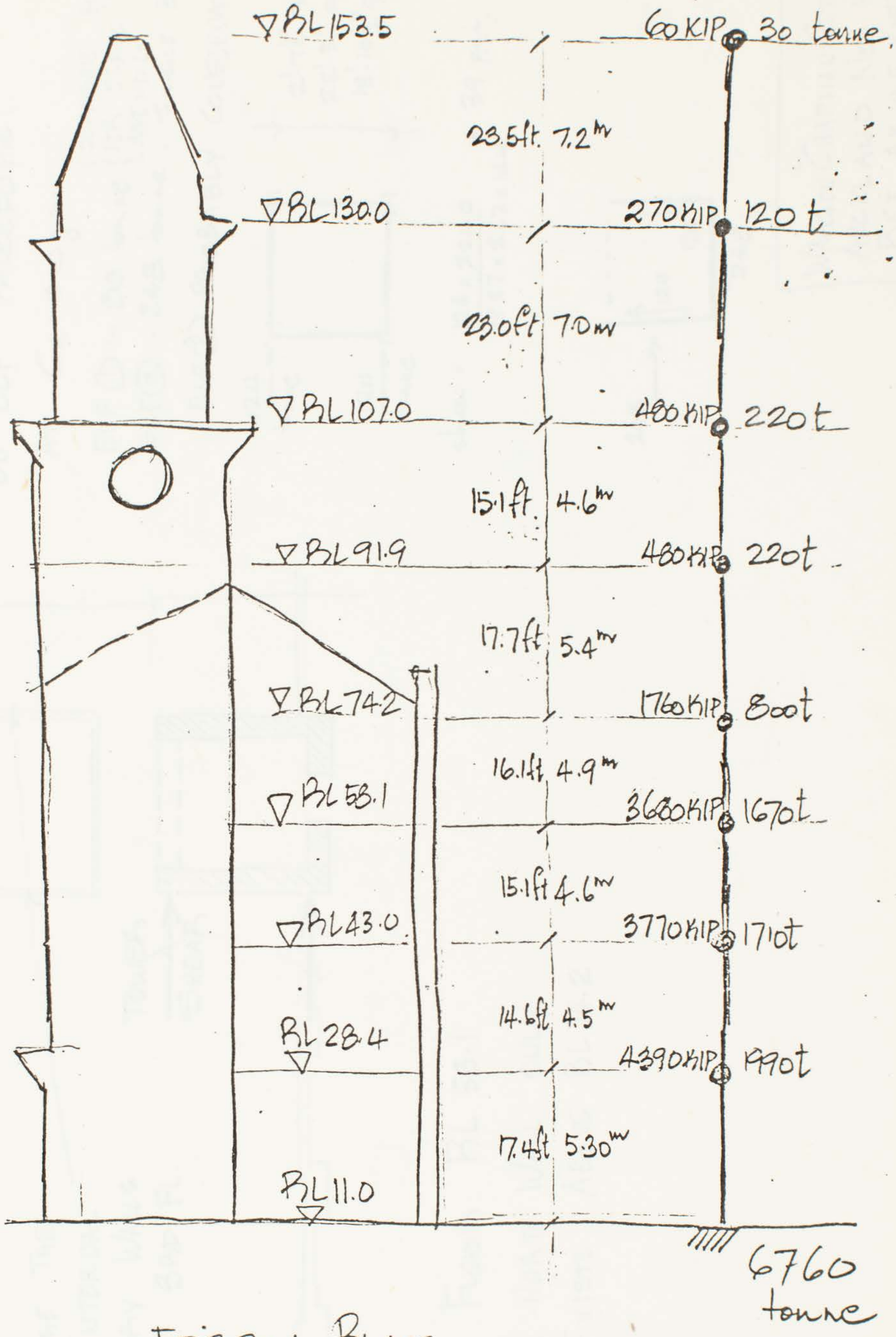
It is quite likely that the strengthening work indicated by such requirements will also be sufficient to provide an acceptable degree of overall lateral strength or, if not, that it will nevertheless account for the major part of strengthening costs.

Yours faithfully,

C R Gurley

WARGON, CHAPMAN & GURLEY

Colin R. Gurley



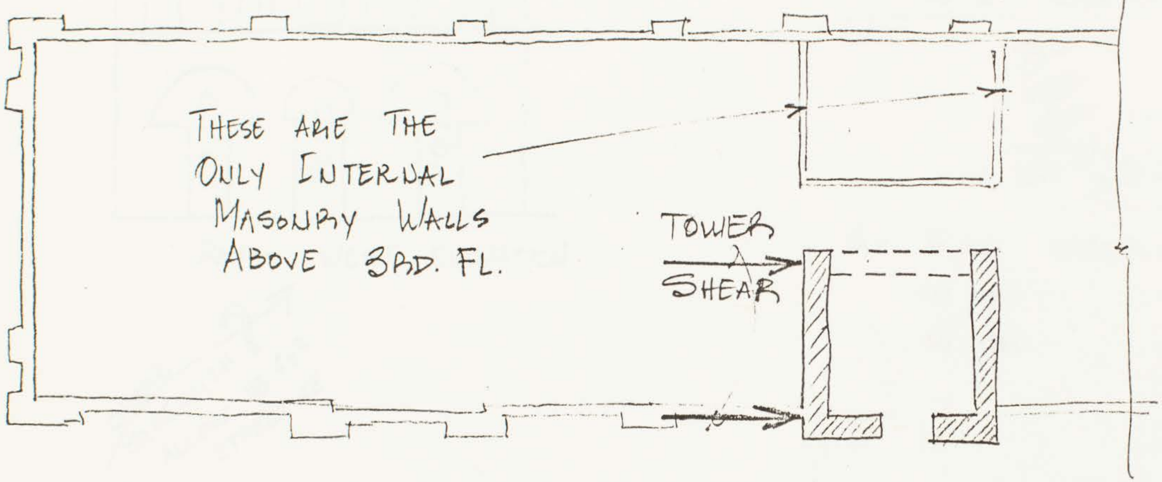
FERRY BLDG.
DEAD LOAD SUMMY.

WARGON CHAPMAN GURLE[Y]
AUCKLAND NOV 197[?]
REF AK 185 / SK4

NORTH.

FERRY BUILDING.

TOWER TORSION PROBLEM.



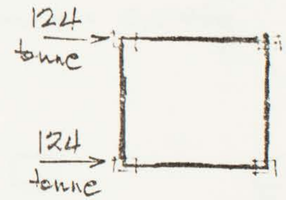
TOTAL TOWER SHEAR DEPENDS ON ELF PROCEDURE:

AT $C_d = 0.10g$:-

ELF ① :- 83 tonne { SEPARATE BLDGS PER SEAOC 'SETBACK' APPENDIX

ELF ② :- 248 tonne. - SINGLE BLDG.

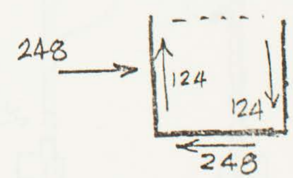
ELF ② PROBABLY CONSERVATIVE



$$\text{shear} = \frac{124 \times 2240}{18.87 \times 2.62 \times 144} = 39 \text{ psi.}$$

THIRD FLOOR RL 58.1

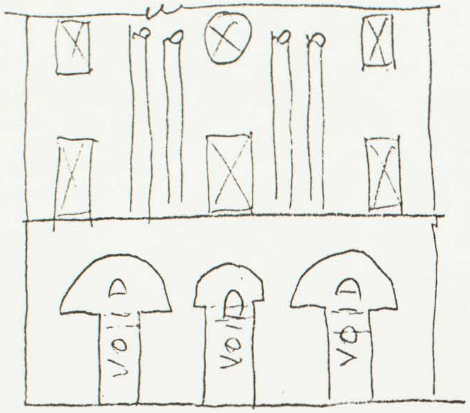
TOWER NORTH WALL ONLY
EXISTS ABOVE RL 74.2



WARGON CHAPMAN GURLEY
AUCKLAND NOV 1974
REF AK165/SK

WARGON CHA
AUCKLAND
REF AK185/

FERRY BUILDING
GROUND FLOOR PLAN
N-S SHEAR AREAS



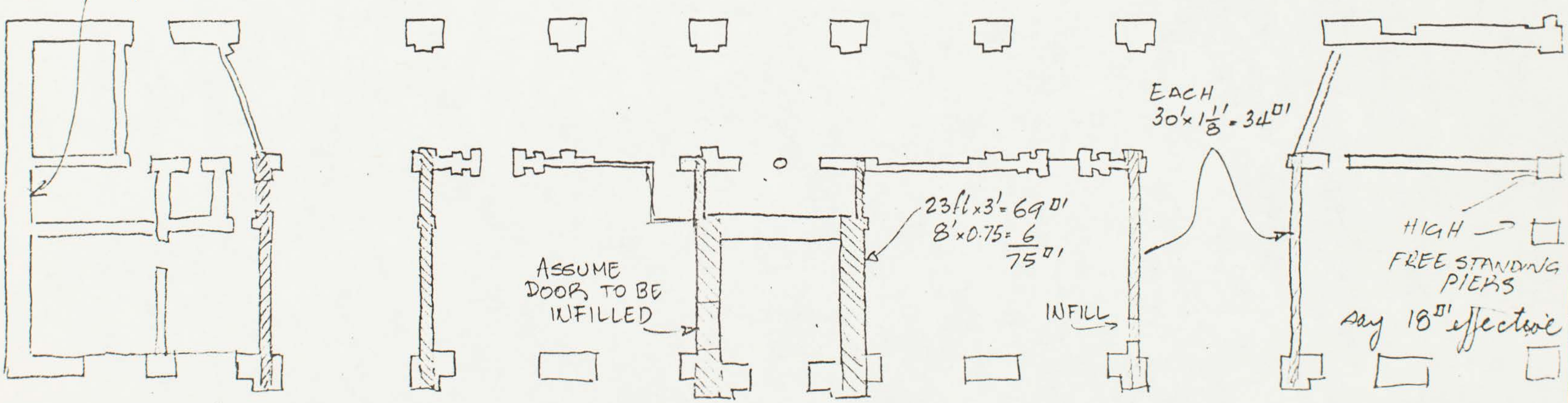
PART WEST ELEVATION

$$
\begin{array}{r}
75^{\square'} \\
68^{\square'} \\
18^{\square'} \\
\hline
161^{\square'} \times 2 = 322^{\square'}
\end{array}
$$

$W_t = 6760$ tonne

AT 5 psi SHEAR = 102 tonne = 0.0151 g
10 psi = 204 tonne = 0.030 g
15 psi = 306 tonne = 0.045 g

SHEAR STRENGTH ? ALLOW 18 ft² EFFECTIVE



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BGC 12/12/79

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* No. 6 gauge bright crimped wire lattice 3" mesh and
* No. 26/8" x 6" x 35lbs. R.S. joists to each of 1st, 2nd and 3rd floors upturned and encased in concrete.

The wire gauge is, presumably, B.S.W.G. in which case No. 6 gauge corresponds to 0.192 inch diameter. This would give a steel area of 246 mm^2/metre width or 0.16% of gross concrete area. This is fairly low compared to minimal current requirements of 0.25% of gross area for mild grade steel. It implies, first of all, that some care will be necessary in transmitting concentrated forces into the slab regarded as a diaphragm.

The spec. gives no information on the plan location of the R.S. joists. I have sighted one such joist running N-S on the third floor. The best guess seems to be that they run N-S so that the slabs span E-W and are supported at up to 9'6" maximum centres. If this is so, then, depending on the yield strength of the steel and on the placement (at midspan and over supports) the slabs may be marginal under service gravity loads even by modern ultimate strength criteria.

We recommend that the slab be opened up at, say one or two locations on each of the major floors to check the mesh actually used, to check consistency and placement and to take samples for mechanical testing. If the wire is hard-drawn then it may well be high tensile. Mr. Neville Miller of the AIDD Division of DSIR (phone 34-116) is prepared to be present to advise on the sampling and testing arrangements.

We would like also to check on actual plan locations of the RS joists wherever possible without disrupting occupancies or damaging floor coverings.

5. Tower

The precise strength requirement of the tower at about existing main roof level depends, in part, on the dynamic analysis mentioned under Paragraph 1. However, there is a need for some vertical reinforcement (perhaps post-tensioned) at the four corners of the tower to provide ductile bending strength and the precise amount may not be particularly critical in drawing up an initial budget. Of rather more concern is that the tower north wall is discontinued just below existing roof level and that there is no structural connection between the tower and the remainder of the building above third floor level. Thus:

* For N-S earthquake the tower cantilevers from 3rd floor effectively as a U-shaped section and
* For E-W earthquake the tower will be subject to significant torsional problems because of the lost shear-strength of the discontinued wall.

It seems essential then to replace the lost shear strength of the north wall above third floor level. It may be possible to go further and to effectively support the tower (considered as a vertical cantilever) at main roof level and to integrate a solution to this problem with that of paragraph 6 below. It will be necessary to look at strength of connections to existing and/or new diaphragms at main roof and third floor levels.

6. Existing Parapets

Council's letter names, as "particular hazards":-

* The 4'8" high perimeter parapet and
* The 5' 6" high ornamental gables to Quay St.

The latter, we think, means the heightened rectangular elements directly over the two main arched public Quay St. entrances each side of the tower. These elements are much heavier than the typical parapets - the flanking sub-elements, in particular, are 3'9" thick. In fact the building is a long narrow one and the existing roof structure does not seem to be a very effective diaphragm so that these elements may behave as vertical cantilevers above third floor level i.e. to a cantilever height of about 22 ft. Council is, in our view, correct to be particularly concerned about this matter although we would not necessarily consider the parapet problem more immediately critical than the tower torsion problem of the previous paragraph 5. It would be possible to correct this particular problem by short-term solutions providing structural steel portal frames across the building and/or steel bracing tieing the parapets back to third floor. We would prefer to resolve this problem within the overall problem and, in particular, to integrate it with a solution to the tower torsion problem.

7. Facades

The existing facades are very heavy and quite intricate. Whether or not, by some addition of reinforcement, they are to be used as major lateral load elements it will, at least, be necessary to tie them back to major structure to prevent them falling out of the building and endangering life in Quay St. particularly. This is likely to be a major exercise in its own right and one in which it may be quite difficult to find some reasonable compromise between the historic value of the building, on the one hand, and earthquake hazard reduction on the other. There seems no doubt that the existing facades of the building constitute almost all of its historic significance and that any drastic alteration to the external appearance of the building would destroy that historic significance.

8. Foundations

a mixture of R.C.? piles and

Our understanding is that most of the perimeter of the building is supported on massive concrete walls which go down from just below Quay St. level about 14 ft. to rock. The details are to be found, not on the Ferry Building drawings themselves but on various other sets related to adjacent quay and wharf construction. This matter seems less immediately critical and we have therefore, for the moment, concentrated on problems related to the superstructure above Quay St. level.

The interior of the building appears to have been supported on timber piles and there is clear evidence of settlement cracking:- > RC?

* In the arches of the E-W collonade on the north side of the building and
* In the N-S bearing walls north of the corridor on the north side i.e. directly over the collonade arches.

The second of these seems to explain Council's comment concerning "masonry bearing walls which are badly cracked at first floor level" although we do not think that we would have used the word "badly" in this context. Certainly, however, there are foundation settlement problems and, if any further substantial investment in the building is to be made then they will have to be rectified. It only makes sense to do so in co-ordination with foundation strength requirements established as an outcome of proposals worked out for strengthening the superstructure.

9. Further Action

The next steps are:-

* To initiate sampling and testing of the existing masonry and concrete slabs and to determine, so far as now possible, plan locations of R.S. joists.

* To evolve and secure Council's agreement in principle, proposals for the minimum amount of work necessary to tie various parts of the building together so that the building at least acts as a whole unit in an earthquake.

The second step includes such matters as:-

* Minimum "basketing" reinforcement of masonry generally.

* Pinning back of the facades.

* Reliable gravity load support to R.S. floor joists.

* Diaphragm action at main roof level and tower/building connections.

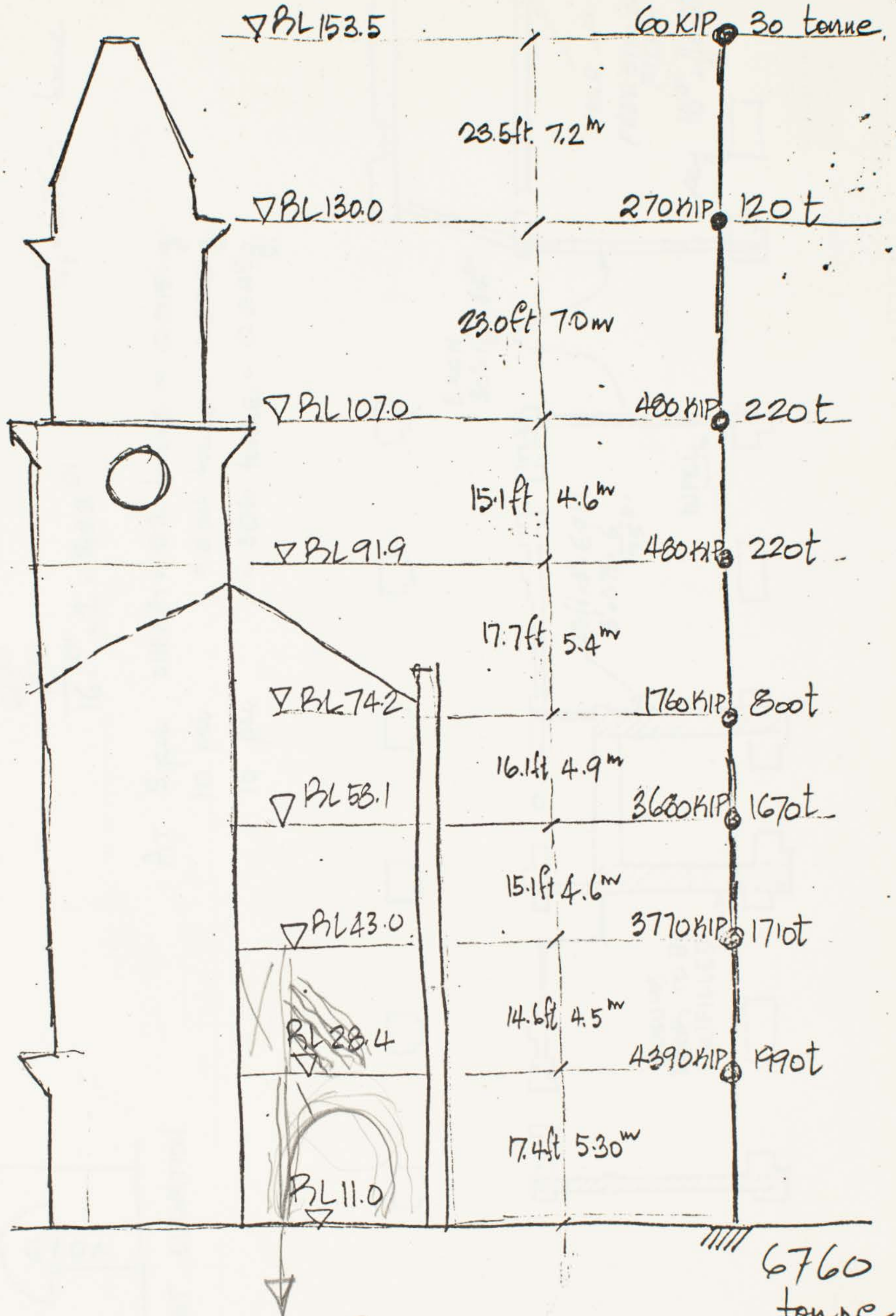
It is quite likely that the strengthening work indicated by such requirements will also be sufficient to provide an acceptable degree of overall lateral strength or, if not, that it will nevertheless account for the major part of strengthening costs.

Yours faithfully,

Colin Gurley

WARGON, CHAPMAN & GURLEY

Colin R. Gurley



FERRY BLDG.
DEAD LOAD SUMMY.

WARCIONI CHAPMAN GURLEY
AUCKLAND NOV 1979
REF AK185/SK4

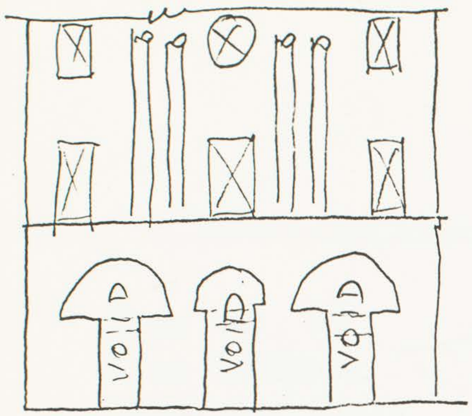
WARGON CHAPMAN
AUCKLAND NOV 7
REF AK185/SK5

FERRY BUILDING
GROUND FLOOR PLAN
N-S SHEAR AREAS

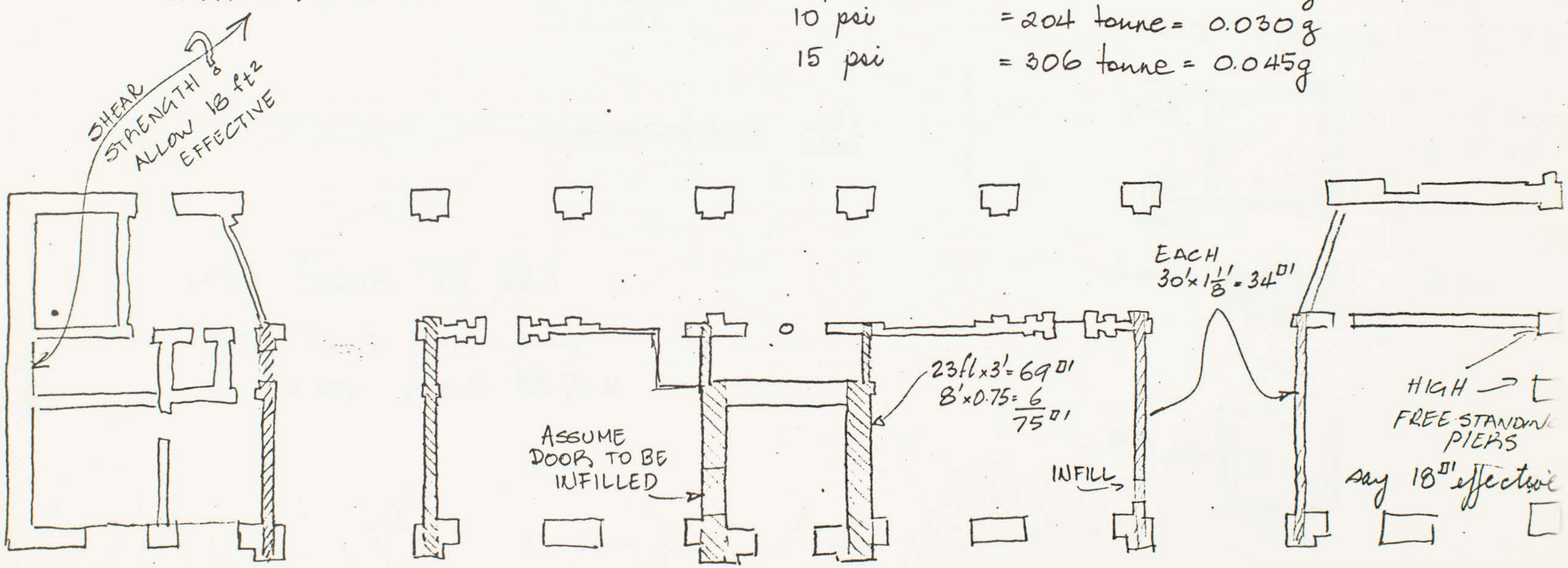
75 □'
68 □'
18 □'
161 □' × 2 = 322 □'

$W_t = 6760$ tonne

At 5 psi SHEAR = 102 tonne = 0.0151 g
10 psi = 204 tonne = 0.030 g
15 psi = 306 tonne = 0.045 g

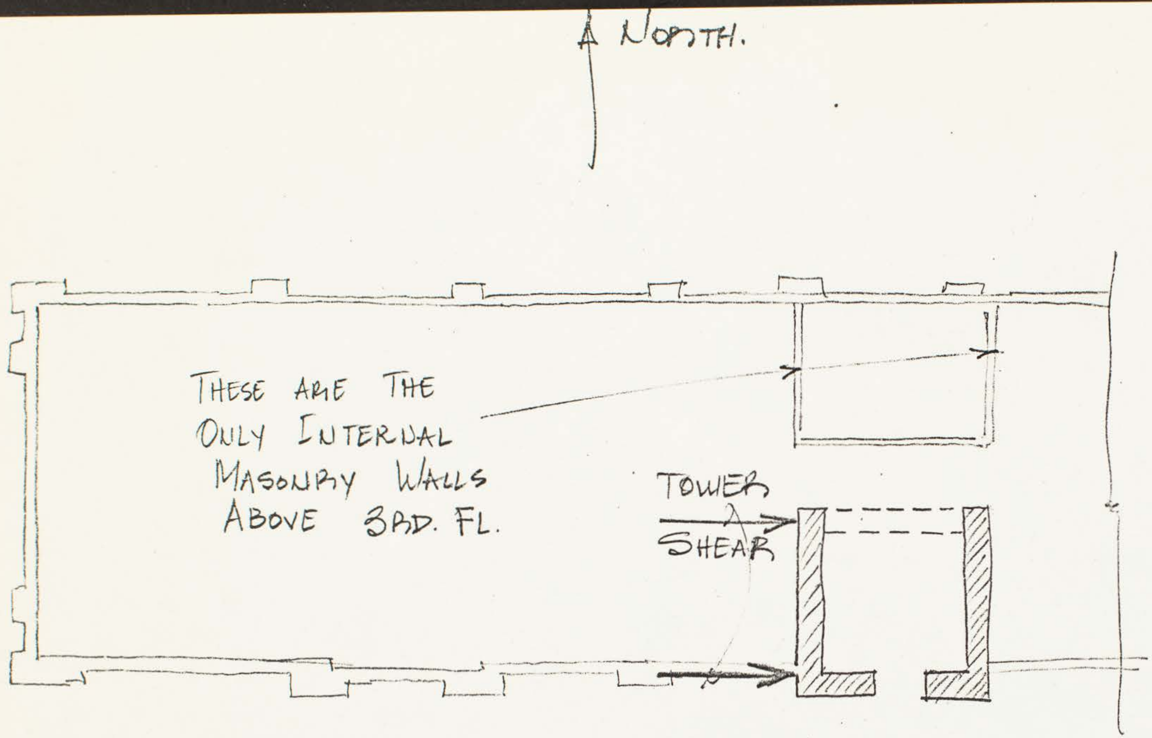


PART WEST ELEVATION



FERRY BUILDING.

TOWER TORSION PROBLEM.



THIRD FLOOR RL 58.1

TOWER NORTH WALL ONLY EXISTS ABOVE RL 74.2

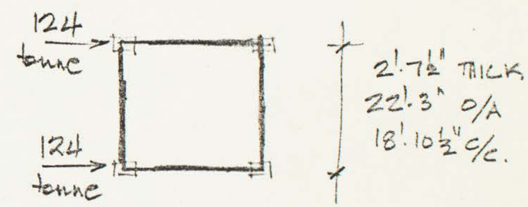
TOTAL TOWER SHEAR DEPENDS ON ELF PROCEDURE:

AT $C_d = 0.10g$:-

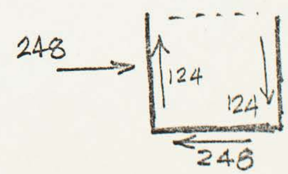
ELF ① :- 83 tonne { SEPARATE BLDGS. PER SEAOC 'SETBACK' APPENDIX

ELF ② :- 248 tonne - SINGLE BLDG.

ELF ② PROBABLY CONSERVATIVE !!



$$\text{shear} = \frac{124 \times 2240}{18.87 \times 2.62 \times 144} = 39 \text{ psi.}$$



WARGON CHAPMAN GURLEY
AUCKLAND NOV 1979
REF AK185/SK 5

Contracts

Letting.

Conditions

General