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N. Z.I. E. news section

A supplement to " New Zealand Engineering " sent to all members of the N.Z. Institution of Engineers

President: R. A. J. Smith, B.E., C. Eng., M.I.C.E„ F.N.Z.I.E.

Secretary: R. W. K. Stevens, C.B.E.

The Secretary's Newsletter

ADDVICE TO MEMBERS ON WELFARE AND LEGAL AID

THE following statement was first published in the November 1968 issue of *New Zealand Engineering* and is reproduced by direction of the Council.

The object of the Institution is the advancement of the practice and profession of engineering. This rather broad statement includes a responsibility to assist members who find themselves in need of advice or help in problems arising from the practice of their profession.

When, and in what circumstances, will the Institution give guidance and assistance to its members and what will be the extent of such assistance? It is not practicable to cover all the issues that might arise, but the Council has adopted the following statement of policy, which embraces the principles applied in the past, and which is sufficiently flexible to cover any circumstances that can at present be foreseen.

Welfare advice

To the extent practicable, the Institution will, without payment, give advice on a welfare matter relating to a member's work or career as a professional engineer.

By a welfare matter relating to a member's work or career is meant one that falls outside the scientific, engineering or educational fields.

Legal aid

The Institution is not permitted by law to give legal assistance to members on matters arising from crime, slander or libel. In any other case where a point of principle affecting engineering or the profession is involved, which has not been previously considered, and which could be of interest to a number of members, the Institution may pay for such legal or other assistance. In cases where no point of principle is involved, the Institution will not provide financial assistance, but it will supply, on request, the names of legal advisers specialising in particular branches of legal work, providing such information is available.

ENGINEERING PROBLEMS AND FAILURES

The Council has for some time given thought to ways and means of stimulating the production and submission of short papers or articles dealing with engineering problems of an unusual nature, including failures. The Council feels that much value can be obtained from a knowledge of the solution to a difficult problem and of the reasons why a particular work did not turn out as planned. A letter is being sent to all branches recommending that they devote one general meeting a year to the presentation of short papers or case studies on engineering problems or failures and members are asked to support the scheme by the submission of suitable material.

STUDY OF ENVIRONMENTAL PROBLEMS BY BRANCHES

The Auckland Branch has suggested to the Council that branches be encouraged to set up committees to study environmental problems and has offered to make suggestions to other branches on the approach to, and conduct of case studies. A letter is being sent to branches commending this proposal for adoption and details of the Auckland Branch activities in this field are to be circulated.

N.Z.I.E. PRIZES FOR STUDENTS TAKING THE N.Z.C.E.

The following awards for the year 1971 have been approved:

(i) For completion of Year V with the most meritorious performance over the whole five years—Stephen Russell Inge, a senior technician with the N.Z.B.C., currently taking the second 3rofessional year at Canterbury University School of Engineering. Mr Inge was a student with the Technical Correpondence Institute.

(ii) For the student who has comfleted the first three years of the N.Z.C.E. course and gained top marks n Mathematics III—Chia Siang Lian, a brass-tape finisher with G. Methven and Co. Ltd., Dunedin, and a student at the Otago Polytechnic.

ADDITION TO THE CODE OF ETHICS

Some months ago the Waikato/Bay of Plenty Branch recommended to the Council that guidance be given to mgineers on matters affecting the environment by the promulgation of a :ode of practice. This proposal was referred to the ethics committee which .eported to the Council that the new N.Z.I.E. code of ethics, approved by he Council in December 1970, already ;ontained much of the material in the :ode of practice. The ethics committee recommended that the Waikato Branch :ase be met by the addition of a clause o the code of ethics. The Council adopted this recommendation and the :ode of ethics now contains the clause,

> "He shall endeavour to relate his work to the preservation or improvement of the environment."

The full code of ethics is published flsewhere in this section.

WELFARE RESEARCH

At the meeting of the executive committee for professional practice on 3 August, it was reported that the consultants carrying out the salary survey had submitted their first report covering comparisons between the salaries paid o professional engineers in New Zealand, Australia, Canada and the United Kingdom, as at April 1968, the latest late for which overseas statistics wzre Available. This report has not yet been considered by the welfare research committee.

The chairman of that committee drew attention to the need for immediate updating of the committee's terms of reference and drew attention to the fact that the welfare research committee was not shown in *Handbook No. 5* as a standing committee. It was agreed that this omission be corrected by the issue of an addendum slip to *Handbook No.* 5, and work on the preparation of terms of reference is proceeding.

COMMISSIONS OF INQUIRY INVOLVING ENGINEERING MATTERS

The Council believes that the Institution should be more closely concerned with the composition of commissions set up by the Government to inquire into accidents, failures and other occurrences with which members of the profession may be involved. A task committee set up to look into the question reported that its investigations showed that qualified engineers had served on all commissions on which the presence of a professional engineer was desirable and that such engineers had been expert in the particular field of inquiry. The task committee found no reason to recommend any change in the Commissions of Inquiry Act 1908, but considered that suitably qualified engineers should continue to be appointed to commissions of inquiry investigating engineering or technological matters and that their selection and appointment should be regularised and clarified with the assistance of the Institution because,

(i) Commissions of inquiry often have to be set up quickly and the Institution is in a good position to assist in the expeditious selection of suitably qualified engineers.

(ii) The Institution can give an independent view on the selection of suitably qualified engineers to serve on a commission of inquiry where administrative heads of departments may have difficulty in obtaining such an independent view if the inquiry involves the actions of engineers employed by that department.

(iii) The investigations into engineering and technological failures or accidents can lead to findings causing distress and loss of professional standing to members of the Institution, and therefore, the Institution should be concerned that the members of the commission are suitably qualified.

(iv) The Institution should show a public responsibility in the field of investigating engineering and technological failures and be seen to show this sense of responsibility by assisting in the selection of engineering members of commissions of inquiry.

The Council accepted the recommendations of the task committee and resolved to offer to the Minister concerned the names of suitably qualified engineers as soon as the need for such a commission becomes apparent.

CODE OF ETHICS

Rule 18.2 states:

Each member shall so conduct himself as to uphold the dignity, standing and reputation of the profession.

In furtherance thereof:

1. He shall not misrepresent his competence nor, without disclosing its limits, undertake work beyond it.
2. He shall not attempt to supplant another member already engaged, except where a duly constituted appeal system operates.
3. He shall not compete with another member on the basis of professional remuneration.
4. He shall discourage criticism in public of the work of another member.
5. Except when reporting on subordinate employees, he shall not review the work of another member without taking reasonable steps to ensure that such member is informed.
6. He shall not disclose any confidential information of his employer.
7. He shall disclose to his employer any financial interest in any process or third party connected with his work. Without his employer's prior approval he shall not accept from or give to any such third party anything of substantial value.
8. He shall endeavour to relate his work to the preservation or improvement of the environment.
9. When practising as a consulting engineer, he shall conduct his affairs in accordance with the current Institution Code of Professional Practice for Consulting Engineers.
10. In connection with work in a country where professional practices are codified he may substitute such practice for any of the above provisions.

—By order of the Council.

Candidates for Election

Any member wishing to communicate with the secretary on the subject of these elections should do so not later than the twelfth day of the month following publication.

For election as Members:

Allen, M. L.; Dewhurst, H.; Ewens, Lt/Cdr. H. J.; Gallot, N. L. J.; Kay, M. G.; King, H. R. D.; Jakobsson, B.; McIntosh, W. D.; Milne, J. T. L.; Oetiker, N. K.; Skidmore, A. H.; Smyth, L. S. W.

For election as Associate:

Cox, M. C.

For admission as Graduates:

Bathgate, A. P.; Chang, W.; Cottle, D. E.; Deverall, W.; Frost, M. W.; Haselden, A. N.; Hewlett, P. J. H.; Jacka, P. M.; Jones, G. D.; Lee, H. P.; Phoon, K. H.; Sinclair, W. S.; Tang, K.; Taylor, R. G. R.; Thomson, G. L.; Watson, P. L.

For admission as Students:

Dove, R. W.; Rolfe, K. A.; Sleep, W. J.

Members for transfer to Fellows:

Bewley, L. D.; Grinlinton, R. McR.

Graduates for promotion to Members:

Campbell, P. L.; Davie, G. E.; Farley, P. J.; Lear, D. G.; Smirk, A. H. C.

Student for promotion to Associate:

Prescott, I. L.

Student for promotion to Graduate:

Gillott, G. N.

ANNUAL REPORT FOR 1970 OF THE N.Z.I.A./N.Z.I.E. JOINT STANDING COMMITTEE

Members

N. Y. A. Wales, chairman; D. Bruce-Smith, deputy chairman; G. W. Butcher, honorary secretary; G. Laurenson, A. L. Andrews, R. P. R. Gulliver, P. E. Weston, R. G. Freeman.

In attendance: S. W. Mitchinson, professional officer, N.Z.I.A., J. G. Excell, director, A.C.E.N.Z.

At the last meeing in 1969 F. E. Kerswill resigned and R. G. Freeman, Wellington partner of Stephenson and Turner, was appointed to the committee. Mr Freeman's wide knowledge of professional conditions in both Australia and New Zealand should prove invaluable to this committee.

Meetings

All members of this committee hold senior positions in private practices and in view of the heavy demands on their time it was agreed that three meetings a year should suffice. However, further meetings could be quickly convened should the occasion warrant.

General

Numerous subjects were discussed during the year, the most important of these being:

(i) *Consulting engineers' revised conditions of engagements and scale of fees*

The committee was first advised on 9 December 1969 that the N.Z. Institution of Engineers were proposing to amend their conditions of engagement and scale of charges.

The N.Z. Institute of Architects was officially advised on 15 April 1970 of the impending increase in the scale of charges. The amount of the increase was not known at that stage.

On 15 June 1970 the N.Z.I.A. replied requesting a delay of three months in promulgating the revised Document B. This request was granted by the N.Z.I.E.

On 11 August 1970, a draft of the conditions of engagement was tabled. Comments on the conditions of engagement were prepared by the professional practice committee N.Z.I.A. and by private architects. These comments were sent to the N.Z. Institution of Engineers. The Institution's fee task committee considered each item and, where they agreed, amended Document B accordingly.

At the meeting on 18 November 1970 the final printing of the conditions of engagement and scale of fees (Document B 1970) was tabled.

This was the first occasion that the question of conditions of engagement and scale of fees had been discussed. The architect members appreciated the opportunity to comment on these documents before the final draft was undertaken.

It was stated that the final document would be revised from time to time and members of both professions were invited to forward to the committee any constructive comments for consideration.

(ii) *Fees for reinforced masonry and precast panels*

Clarification of the N.Z.I.E. fee scale relating to the use of reinforced masonry and precast panels had been requested by architectural and engineering firms.

A sub-committee, consisting of T. R. Svendsen, N.Z.I.A., and R. C. Amos, N.Z.I.E., has considered the problems involved and has given the joint standing committee a preliminary report.

Rules for the specific division of architecture and structure for sophisticated forms of block and precast construction are not practicable as such a division must vary with the particular project and with the individuals of the particular design teams.

Further information is still to be obtained and it is hoped that a practice note will be published in both the *N.Z.I.A. Journal* and *New Zealand Engineering* in the near future.

(iii) *Limited liability—corporate practice.*

The N.Z.I.E, have for some time had a sub-committee investigating the important question of limited liability practice. With the number of architect engineering practices in existence, it is obvious that this must be a combined exercise between the two professions. The engineers have provisionally agreed to an exchange of information.

The acceptance of limited liability in Australia gives a fair lead as to its likely acceptance in New Zealand. (Refer to the article "Limited liability in Australia" in the *R.I.B.A. Journal,* May 1970.)

(iv) *Municipal Corporations Act*

When the Architects Bill (now the Architects Act 1963) was before the Statutes Revision Committee of the House of Representatives, clauses 55 and 56 of the Bill were deleted. These provided that when a local authority was undertaking building work in excess of a specified value, the services of a registered architect were to be mandatory.

This was a parallel to a similar provision in the Engineers Registration Act.

The Statutes Revision Committee expressed the opinion that provision in the Bill, together with those in the the Engineers Registration Act, should be more properly in the Municipal Corporations Act and the Counties Act.

Since 1968 there has been some misunderstanding regarding the N.Z.I.E.'s position in this connection. The present situation is that the Institution is willing to assist the N.Z.I.A. in achieving its objective of a statutory requirement that local authorities employ an architect on projects above a specified estimated cost. It has not, however, specifically said that it was willing to support a move to amend the Engineers Registration Act. The secretary of the joint standing committee has written and advised the secretary of the N.Z.I.A. accordingly and has asked what assistance the N.Z.I.A. would require of the Institution to achieve its objectives. It is hoped that this matter may be taken a stage further during the coming year.

(v) *Conditions of engagement of architects and professional consultants*

The draft of a document prepared for the Royal Australian Institute of Architects and Building Industry Advisory Council headed " Conditions of engagement of architects and professional consultants" was tabled and consideration has been given to this document to see whether it has any application to New Zealand conditions.

(vi) *Dade Practices Amendment Bill*

This item was discussed and it was noted that there was a worldwide trend to regard scales of fees as a restrictive practice. A watching brief should be maintained on this development.

(vii) *Task committee reports*

(a) Project briefs: Little progress has been made on this subject over the past year. Further information is being obtained from the Victorian Branch of the Royal Australian Institute of Architects.

(b) Package deal: A brief report on the subject was received at the December meeting 1969 and a final report has been requested from this committee.

(c) Cost information: An Auckland committee has agreed to prepare a report on this subject.

Motion of thanks

On behalf of the committee, the chairman thanked members of both professions for their valuable assistance and work on the task committees. He suggested that the experience gained by serving on these committees and the respect engendered by mutual discussions leading to a common goal served to strengthen the ties between the two professions.

Election of officers

Officers for 1971, elected 17 November 1970:

D. Bruce-Smith, chairman; R. G. Freeman, deputy chairman; P. E. Weston, honorary secretary.

—N. Y. *A. Wales Jr.,*
Chairman, Joint Standing Committee, 1970.

Changes in the Roll of Members

The following additions to and changes in the Roll of Members result from recent decisions of the Council, subject to confirmation under the provisions of rule 7.1 where applicable.

ADDITIONS

Fellow

F. Dr W. A. Fairhurst, C.B.E., LL.D., C.Eng., F.I.C.E., F.I.Struct.E., M.I.H.E., M.Soc.C.E.(France), 12 Kia Ora Flats, Pukuatua Street, Rotorua.

Members

M. K. B. Cassey, B.E.(Civil), 14 Edison Place, Auckland 5.
M. N. F. Falloon, B.E.(Mech.), C.Eng., M.I.Mech.E., Nutter Road, Muriwai Beach, Auckland.
M. H. J. Feickert, C.Eng., M.I.Mar.E.. 106 Donald Street, Wellington 5.
M. J. R. McGimpsey. C.Eng., A M.I.Chem.E., 15 Rapaki Road, Christchurch 2.
M. D. MacGregor, V.R.D., C.Eng., F.I.Struct.E., M.A.S.C.E., 75 Solefields Road, Sevenoaks, Kent, United Kinedom.
M. D. F. Palmer, C.Eng., M.I.Prod.E., M.I.M.H., 136 East Coast Road, Milford, Auckland.
M. Lt.-Cdr. R. C. C. Pearce, C.Eng., A.M.I.Mar.E., 4 Ringwood Street, Torbay, Auckland.
M. D. Stone, B.Sc.(Hons)(Elect.Engrg)(Leeds), C.Eng., M.I.E.E., 32 York Street, Masterton.
M. A. N. Trower, C. Eng., M.I.Mar.E., 17A St. George Street, Timaru.

Graduates

Grad. N. A. Anderton, B.E.(Hons) (Elect), P.O.Box 36, Waipukurau.
Grad. N. J. Barclay, B.E.(Hons)(Mech), Civil Aviation Division, Ministry of Transport, Private Bag, Wellington.
Grad. R. D. Beetham, B E.(Civil), 4A Bloomfield Terrace, Lower Hutt.
Grad. W. L. Busch, B.E.(Civil), 39 Wadestown Road. Wellington 1.
Grad. W. D. C. Clark, B.E.(Civil), 143 Marine Parade, Eastbourne.
Grad. W. de Boer, B.E.(Chem), Grad.I.Chem.E., P.O. Box 109, Drury, Auckland.
Grad. P. T. Finlay, B.E.(Civil), 48 Ellesmere Crescent, Takaro, Pamlerston North.
Grad. C. L. Foster, B.E.(Elect), District Engineers Office, Post Office, Greymouth.
Grad. B. P. Gibbs, B.E.(Civil), 9 Sophia Street, Rotorua.
Grad. K. F. Hosking, B.E.(Mech)(Hons), No. 4 R.M.D., Christchurch.
Grad. D. W. Langdon, B E.(Hons) (Civil), Ministry of Works, Tauranga.
Grad. C. G. Lloyd, B.E.(Elect), Flat 3, 69 Elizabeth Street, Rotorua.
Grad. D. C. MacDonald, B.E.(Civil), 15 Fairholme Avenue, Epsom, Auckland 3.
Grad. F. W. Matthewson, B.E.(Hons)(Elect), 9 Carlton Mill Road, Christchurch 1.
Grad. S. R. H. Miles, B.E.(Hons) (Civil), 39 St. Stephens Avenue, Parnell, Auckland.
Grad. E. Nagoya, B.E.(Elect), Nakorovatu, Matailobau, Naitasiri, Fiji.
Grad. A. J. Naughton, B.E.(Elect), 30 Kaihuia Street, Northland, Wellington 5.
Grad. K. M. 011ivier, B.E.(Civil), 20 Durham Street, Brooklyn, Wellington 2.
Grad. Dr M. N. Rao, B.E.(Civil) (Andhra), M.Sc.(Engrg), D.Phil., A.M.I.E.(India), School of Engineering. P.O. Box 2175, Auckland.
Grad. R. J. Redmayne, B Sc.(Hons) (Engrg) (London), Grad.I.C.E., 27 Allen Terrace, Linden, Wellington.
Grad. R. J. Simpson, B.E.(Hons) (Elect), Student member I.E.E., 74 Cannon Hill Crescent, Christchurch 8.
Grad. B. J. Thompson, B.E.(Mech), 17 Kelso Place, Invercargill.
Grad. J. C. Wilson, B.E.(Civil), 60 Somerfield Street, Christchurch 2.

Students

Stud. W. J. R. Burrett, 92 Colwill Massey, Auckland 8.
Stud. K. G. Peacock, 22 Anthony Crescent, Hamilton.

PROMOTIONS

Members to Fellows

F. R. D. G. Monk, C.Eng., M.I.C.E., M.I.P.H.E., M.I.W.E., Camp, Dress & McKee, One Centre Plaza, Boston, Massachusetts 02108, U.S.A.
F. R. H. Newton, D.F.C., C.Eng., M.I.C.E., A.M.Inst.T., District Engineer's Office, N.Z. Railways, Auckland.

Graduates to Members

M. H. B. Adcock, C.Eng., M.I.C.E., Downer & Co. Ltd., Private Bag, Invercargill.
M. D. S. Evans, B.E.(Hons)(Civil), C.Eng., M.I.Struct.E., Palmer & Turner, 1906 Prince's Building, Hong Kong.
M. B. L. Halliday, B.E.(Civil), 9 Pipitea Street, Wellington.
M. M. A. Packer, B.E.(Civil), 31 Haumia Street, Johnsonville, Wellington.
M. R. D. Sullivan, B.E.(Civil), 22 Clipper Place, Christchurch.
M. D. R. Young, B.E.(Civil), Downer & Associates, Private Bag, Turangi.

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Leading article

Papers and articles

General

Cover picture

The Westport breakwater. By courtesy of Wilkins and Davies Construction Co. Ltd.—see p. 266.

NEW ZEALAND Engineering

THE JOURNAL OF THE N.Z. INSTITUTION OF ENGINEERS Vol. 26, No. 9, September 1971

Cheaper by the hundred

ELSEWHERE in this issue is a note from the N.Z.I.E. Water Supply Committee on the metric units that they are recommending for use in the water industry in New Zealand. This is the first publication of this sort in this journal; we expect that it will be the first of many, as more people come to grips with the problems of metrication in their particular fields.

The complexity of these problems is, we feel, only just beginning to be appreciated by engineers in this country. Difficulties with standardisation have always been with us, even when there has been only a single and long-established system of units to work with; the old joke about the plumber who always has to go back to the workshop for something he has forgotten has its roots in the almost incredible variety of sizes and types of equipment used in the water industry over the years. Only now (and by really hard work on the part of those concerned) are these varieties being reduced to something like a reasonable number. No doubt there are many other industries with a similar tale to tell. The question now is whether the advent of metrication will make this multiplicity of sizes and types worse again, while the old imperial sizes run parallel with new, metric ones, or whether the opportunity can be grasped for further standardisation so that the inevitable changeover period can be smoothed and the new metric system be a real improvement. Such a desirable state of affairs can be achieved only if real thought is given early to the problems of the changeover by all the interested groups in an industry.

The Institution is taking a number of steps to help engineers in all fields with the metrication programme. The 1972 conference is to have a period set aside for papers on a number of aspects of metrication including, incidentally, the proposals of the Publications Committee on the way the change of units should be handled by authors of technical papers in *New Zealand Engineering.* The conference session will also cover the use of the non-gravitational units that comprise the SI system, together with a general approach to metric education and the engineer. A metrication sub-committee was set up some time ago by the Executive Committee for Engineering Science; the sub-committee members are E. H. Hitchcock, chairman of E.C.E.S. and chief technical adviser of the Standards Association of New Zealand, I. D. Stevenson, the chairman of the Metric Advisory Board, and H. W. Robertson, assistant chief power engineer of the Ministry of Works. This sub-committee is prepared to examine proposals—such as that by the Water Supply Committee—put forward by any group of the Institution before sending them to the Metric Advisory Board as official recommendations of the N.Z. Institution of Engineers. The task of the sub-committee is just beginning; if the government's target date of 1976 for "substantial conversion" to the SI metric system is to be met by anything like a subtstantial sector of the country, then a lot of spadework must be done in the next year or two. Already work is being planned in many engineering offices for construction in 1975 and 1976, or later; the question is being asked whether such work should be designed in metric quantities. Until some more thinking is done, industry by industry, on the way that the changeover should be handled, such a question is a very difficult one to answer.

Some time ago, the Institution's branches and divisions were informed of the existence of the metrication sub-committee, and were invited to consider areas where they could put forward metrication proposals which the sub-committee could then consider transmitting to the Metric Advisory Board. Activity is now beginning to be generated. We all have a part to play in making this inevitable change and in hastening the day when, at last, things by tens and hundreds will be cheaper than by dozens.

Design of *hollow skew deck spans as applied to Kilmore and Madras Street bridges*

R. C. FENWICK
B.E.(HONS.), PH.D. (MEMBER)

J. A. INCE
B.E. (MEMBER)

This paper describes the purpose of constructing the Madras Street and Kilmore Street bridges in close proximity across the Avon River and the design difficulties imposed by this situation. The highly skewed deck systems constructed in prestressed concrete presented problems of analysis which the design attempted to satisfy. Subsequent study has been made of associated creep behaviour and of methods of digital computer analysis.

J. A. INCE has been divisional engineer (design) with the Christchurch City Council since 1963. He is responsible for an office of 50 engineers, surveyors, engineering assistants and draughtsmen engaged on all classes of civil and structural design.

He was born in Levin and from 1952 until 1959 was with the Ministry of Works, spending two years in the hydro-electric design office, two years at the Taumarunui residency and four years on the Ohakuri power project.

In 1960 he joined the Christchurch City Council as staff engineer (design), a position he held until 1963.

He has been a committee member of the N.Z.I.E. Canterbury Branch since 1969.

1. GENERAL DESCRIPTION

THE Madras Street bridge was built in 1967-68 as a replacement for an old multi-span wooden bridge which was constructed over 80 years ago. During the last few years of its life it had been subject to a severe load restriction owing to excessive decay. Since its construction this timber bridge had satisfied

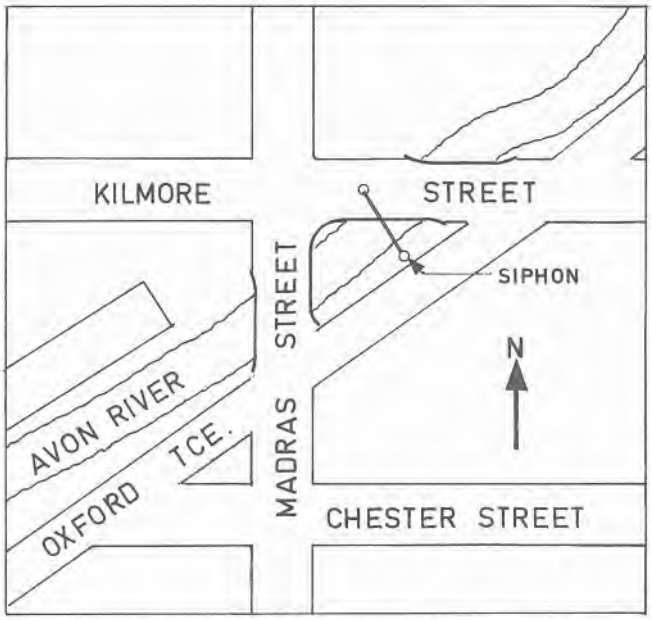

Fig. 1: General locality of Kilmore and Madras Street bridges.

the needs of the traffic crossing the river in this locality. It was aligned to suit the north-south movements, while the low density east-west movements along Kilmore Street were adequately catered for by following a zig-zag route across this bridge.

A major change in this arrangement was planned in 1967, when it was decided to introduce a one-way street system to improve traffic conditions in the central part of Christchurch[1]. The only suitable pair of streets

This paper was first received 18 Ocober 1970, and in revised form on 2 May 1971.

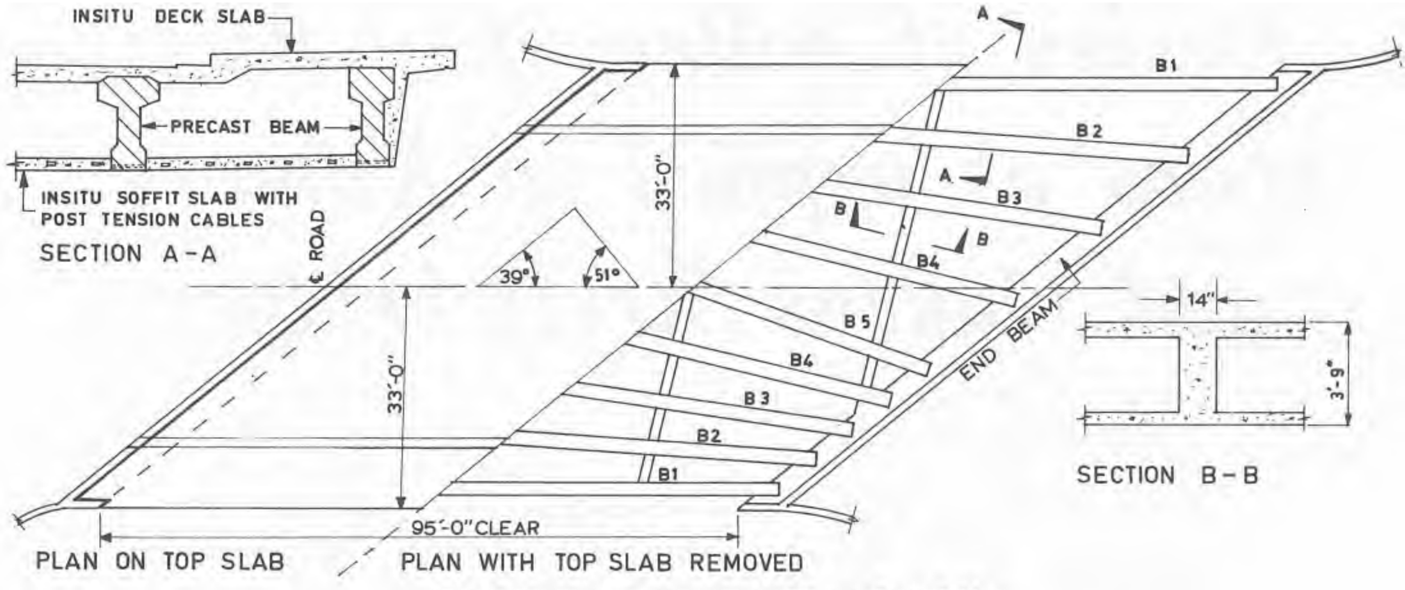

Fig. 3: Structural arrangement for Kilmore Street bridge.

for the east-west traffic movements just north of the city centre were Kilmore and Salisbury Streets. To allow Kilmore Street to carry one-way traffic it was essential to eliminate its discontinuity at the Avon River, and for this purpose the second bridge was built in 1969. A general plan of the site is shown in Fig. 1.

There were unique difficulties associated with the site of these two bridges. Kilmore and Madras Streets intersect at right angles very close to the river so it was not practical to realign the road layout, and only minor adjustments could be made to the river position. As a result both bridges were designed to span the river with considerable skew. For the Madras Street structure with span of 75 ft the skew was 39°, and for Kilmore Street the span was 95 ft and the skew 51°.

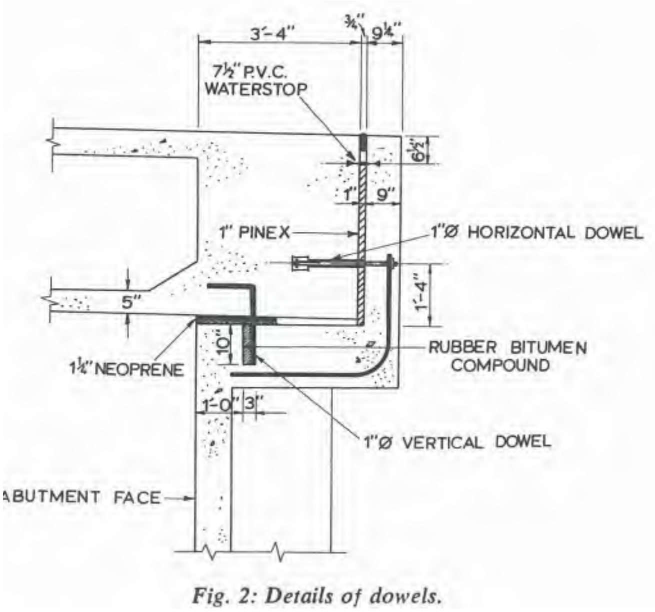

Fig. 2: Details of dowels.

Geometric considerations of road shape and river level associated with desirable aesthetics of the skew spans led to the selection of single-span structures with minimal depth to span ratios. A model was constructed to test the design in relation to the surroundings. A further limiting factor in the design of the Kilmore Street structure was the presence of a large 90-year-old sewer siphon which influenced the position of the west abutment.

Apart from this complication the foundation pads and abutments were straightforward. They were founded on dense gravels, and were designed as free-standing vertical buttressed walls mounted on reinforced-concrete foundation pads. The precast beams, which formed the backbone to both decks, were supported on neoprene bearing pads mounted on these buttressed walls. To secure the deck under the action of severe earthquake loading the end beams were tied into the abutment walls by a number of vertical and horizontal dowels, which are illustrated in Fig. 2. The horizontal dowels, which were designed to carry the shock loading in the direction of the traffic movement by axial tension, had one end fixed in the abutment back wall and the other end in the deck was attached to a plunger located in a cavity filled with a bitumen compound. Under the slow application of load arising from temperature or shrinkage movements the bitumen can flow round the plunger allowing movement to occur, but under dynamic loading the assembly should act as a rigid tie. Earthquake forces at right angles to the traffic direction act on the side walls which frame the bearing seat and on the dowels (in shear). Vertical forces are resisted by shear in the horizontal dowels and by jamming between the back walls.

2. THE STRUCTURAL FORM OF THE DECK

In a number of previous designs for river bridges in Christchurch use has been made of precast pretensioned beams acting compositely with *in situ* concrete to give a hollow structure. This form has an advantage over *in situ* decks in that the need for false work staging in the river is virtually eliminated. Although the structural arrangements used for the Kilmore and Madras Street bridges follows the general form of the earlier bridges, it differs in a number of important aspects. The new features include the use of stage-stressed beams at wider centres. For the Madras Street bridge they were placed parallel to one another at 6 ft centres, but for the Kilmore Street structure the non-parallel arrangement shown in Fig. 3 was used for the following reasons.

The amount of flexural reinforcement required in a bridge depends on its orientation. With parallel beams the steel arrangements (1) and (2) shown in Fig. 4 are possible, while (3) corresponds to the adopted scheme at the centre of the deck. An idea of the relative efficiencies of these alternatives may be obtained by determining the quantities of reinforcement required to resist the bending moments with each. For a slab with an aspect ratio a/b =0.67 (see Fig. 4) and a skew of 52.5° (compare Kilmore Street aspect ratio=0.68 and skew of 51°) the bending moments at the central point of the slab may be found from tables[4] for different loading cases. Table I shows the relative quantities of reinforcement required per square foot at the central point of the slab for a uniformly distributed load. These values have been obtained by using the theory developed by Wood[10] and Armer[11]. They clearly show that arrangement (2) is impracticable. The adopted scheme (3) appears to be the most efficient, but the difference between it and scheme (1) is relatively small.

With the parallel arrangement of beams the dead load is initially carried on a 96 ft span, while for the adopted scheme the spans vary between 72 ft and 96 ft. With the first arrangement the dead-load bending moments are higher and calculations show that approximately 50% extra precast concrete is required.

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After a year as an assistant engineer with the Christchurch City Council, which was spent on bridge design, he left for the United Kingdom and was employed by Mott Hay and Anderson of London. During the three years which Richard Fenwick spent overseas he was involved with the design of complex elevated motorway structures. He returned to New Zealand in 1970 to take up his present appointment.

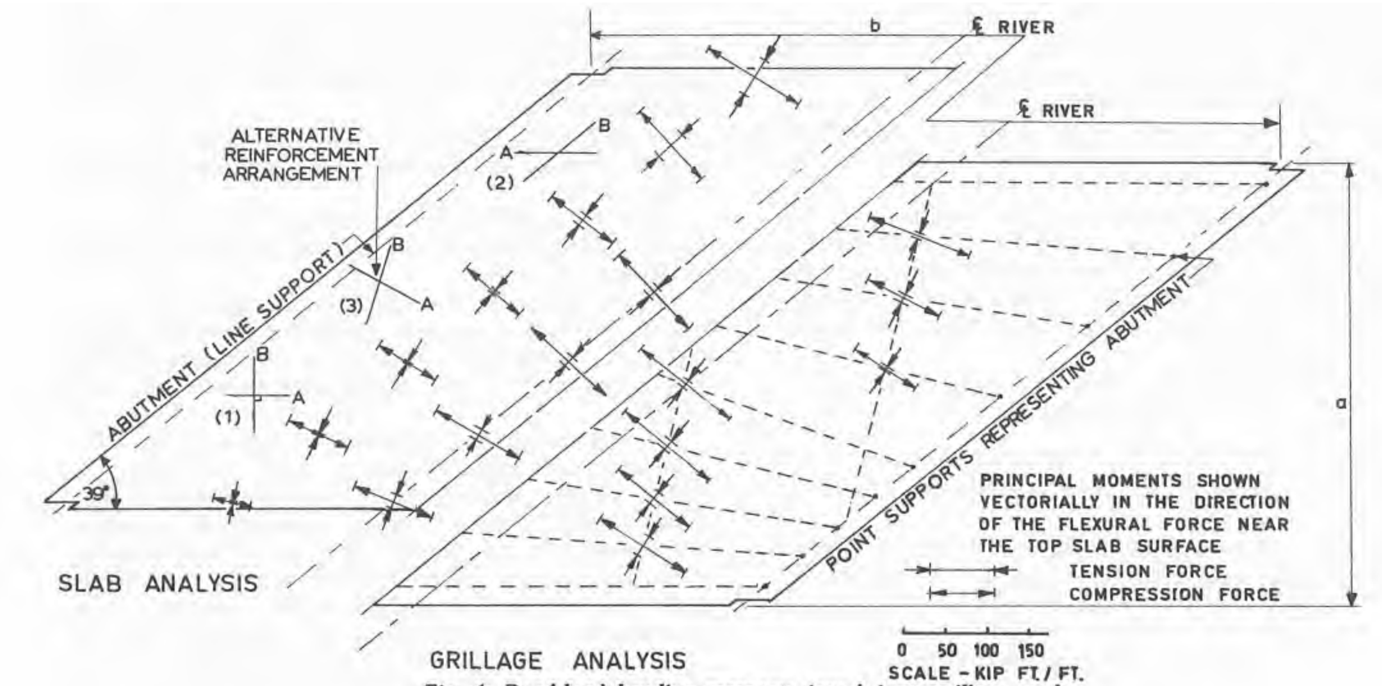

Fig. 4: Dead-load bending moments found from grillage, and slab analyses.

TABLE I

RESULTS OF ANALYSIS OF CENTRAL POINT OF SLAB FOR DIFFERENT REINFORCEMENT ARRANGEMENTS

Bending moments at central point of slab (see Fig. 4)

m_x= 16.39L m_y, 23.43L m_{xy}=22.50L

Principal moments

m_n=42.91L m_q=-2.6I. Θ=49.1°

Steel equivalent moments for reinforcement design

Arrangement	Direction A bot.	Direction A top	Direction B bot.	Direction B top	Sum rein. moments	Belalive quantities
1	39.4L	— 4.7I.	45.9L	0.0L	90.0L	1.16
2	202.5L	0.0L	150.3L	—2.6I.	355.41L	4.6
3	46.7L	0.0L	26.8I.	—3.9I.	77.4I.	1.0

The advantages and disadvantages of the non-parallel arrangement of beams used for Kilmore Street may be summarised as:

The beams are more closely aligned to the directions of the major principal moments in the structure and this leads to a more efficient reinforcement arrangement.

The amount of transverse reinforcement required in the soffit slab was reduced, and the wider spacing of beams allowed it to be lapped between the bottom flanges. This eliminated the practical problem of threading the reinforcement through several ducts, though it meant that many short bars of differing lengths were required.

Creep of the concrete leads to a redistribution of the bending moments. The extent of this redistribution is less with the adopted scheme than with the parallel beam arrangement. This aspect is discussed in a later section.

The reduction in length of the internal spans reduced the dead-load bending moments, and this led to a reduction in the quantity of precast concrete required. From the erection point of view the reduced weight of the internal beams proved to be an advantage. The edge beams, which were the heaviest, at 27.2 tons, were close to the limits of the crainage available in Christchurch.

With the adopted scheme there were five different beams, all of which were cast in the same mould after modification to allow for the different positions of the web ducts. Minor errors in the theoretical soffit shape resulted from casting all beams off the same platen.

3. SEQUENCE OF CONSTRUCTION FOR THE BRIDGE DECKS

The main stages in constructing the deck were:

The precast prestressed beams were lifted into place.

The end beams and soffit slab were cast against boxing which was supported from the main beams.

The remaining cables in the main beams and the cables in the soffit slab were stressed.

The top slab and the transverse diaphragms were cast.

All the post-tension cables were grouted.

With this sequence of construction the dead load of the structure was initially carried in the following way:

(a) The precast beams supported their own dead load in addition to that of the soffit slab, and;

(b) The soffit slab acting compositely with the precast beams carried the dead load of the top slab and the diaphragms.

4. REDISTRIBUTION OF LOADING WITH CREEP

With the construction sequence described above it will be realised that initially the dead load acting on each beam or composite beam and slab may be found frdm statics. However, creep causes this initial load distribution to change.

Creep redistribution occurs in a concrete structure if its form is changed after part of the load has been applied. The extent of this redistribution may be very considerable, and although it is unlikely to cause an appreciable reduction in the static ultimate load capacity it is possible that the onset of cracking and the fatigue strength could be detrimentally affected. Several methods of assessing the redistribution arising from creep are available 3.

Two aspects should be considered; the first is the redistribution of the forces and stresses within a section, and the second the change in the load paths in a structure.

Consider the top flange of the precast beams immediately after the *in situ* concrete has been added. The precast concrete is subject to a compressive stress of about 1,500lb/in^2 while the surrounding *in situ* concrete is initially unstressed. The two concretes are bonded together and thus creep movements in the highly stressed precast concrete are restrained by the *in situ* concrete. This action leads to a transfer of forces in the section which results in a reduction of the stress differentials between the concretes.

As previously mentioned creep can cause the load paths to be redistributed in a structure. The steps outlined below may be used to assess the extent of this redistribution:

Analyse the structure in its initial condition but imagine it to be constrained against any creep movements by ties.

As the concrete tries to creep it is restrained by the ties which consequently carry an increasing portion of the load.

With the removal of the ties the load previously carried by them is released on to the structure in its final form. Thus the load paths in a structure change with creep only if the initial and final forms are different.

Simple analytical expressions derived from the "rate of creep method" may be used to predict the extent of creep redistribution[2]. Experimental work has shown that this method gives a reasonably accurate assessment of creep effects[3]. The creep characteristics of the concrete are defined by the parameter Φ, which is the ratio of the creep strain to the elastic strain in an element of concrete subject to a uniform constant stress. The proportion of the load carried by the structure in its initial condition after all creep has occurred is given by the expression and the remainder of the load (l-e -Φ) is carried as though it had been applied after the structure had been completed. For normal concretes the value of Φ lies between 1.5 and 3.0. Its actual value depends on many factors which include the cement content, the age of the concrete, the volume to surface ratio and the drying characteristics of the environment. The precast beams were several months old when the soffit and top slabs were added, and as these slabs further insulate the beams from water loss the creep factor applicable to this case is likely to be in the range of 1.0 to 1.5. With a value of 1.25, Equation (1) indicates that when the creep is complete only 29% of the load will be carried in the way in which it was applied, with the remaining 71% being carried as though it had. been applied after the structure had been completed. Even with this low creep factor the majority of the load is subject to redistribution. Examination of creep tests shows that even if the beams were a year old before the *in situ* concrete was cast 50% of the load could still be subject to load redistribution.

In carrying out an analysis for the effects of creep redistribution it is necessary to allow for the different elastic and creep properties of the *in situ* and precast concretes. In this case the *in situ* concrete has a higher creep factor and a lower elastic modulus than the precast concrete. Both these factors may be allowed for by reducing the effective elastic modulus of the *in situ* concrete when determining the properties for the analysis of the structure in its final form.

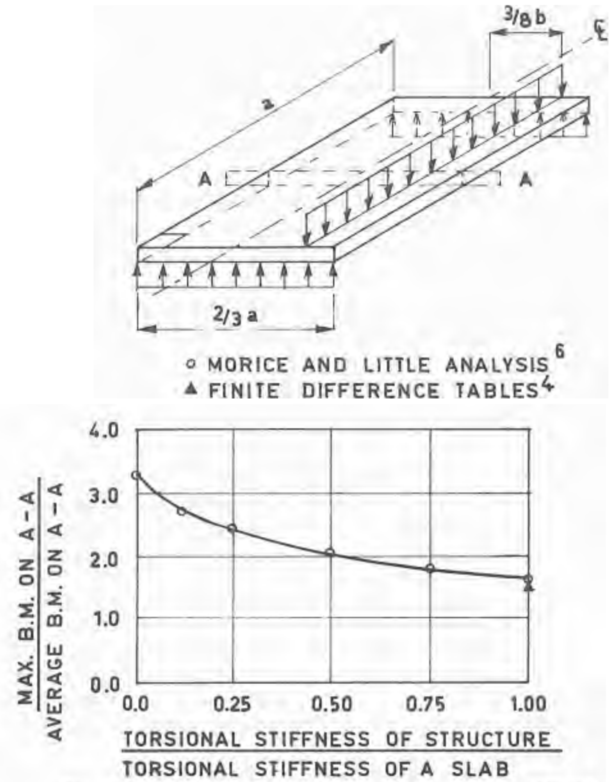

Fig. 5: The influence of torsional stiffness on moment distribution in a right bridge.

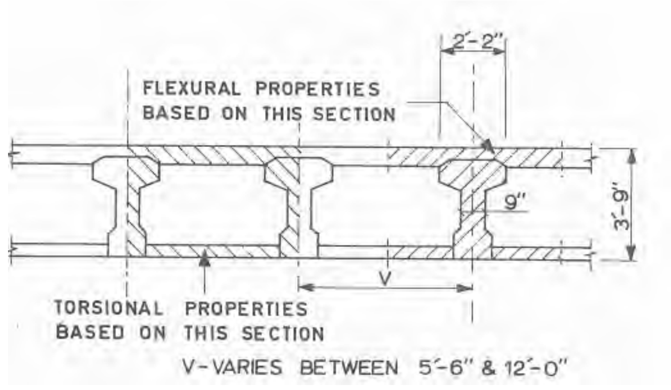

Fig. 6: Sections used to assess section properties for grillage analysis.

5. ANALYSIS OF THE KILMORE STREET BRIDGE

At the time when this structure was designed (mid 1967) there were no suitable computer programmes available in Christchurch, so that hand-calculation methods had to be used. For the purposes of design the dead loads were assumed to be distributed in their initial condition. Estimates showed that creep redistribution of prestress and dead loading would tend to improve the stresses in the beams. The distribution and magnitudes of the live load bending moments were estimated and subsequent analysis has shown these values to be on the conservative side.

Prior to the construction of the bridge a local computer bureau offered a grillage analysis package and this was used to check the assumed distribution of live load moments. At a later stage a second grillage analysis was made in the United Kingdom and the results from this were compared with the bending moments found from influence tables for skew slabs[4] (see Fig. 4).

The analysis of this type of structure, which in its final form lies somewhere between a slab and a grillage, presents very considerable difficulties and an analysis based on either model leads to error. It is hoped that new techniques based on finite element analysis will be developed which will deal more satisfactorily with this structural form. Some of the limitations of using the grillage analysis for this type of structure are discussed in the following paragraphs.

In the choice of the sectional properties to be be used in the grillage analysis the values of flexural stiffness present little difficulty. The structure can be assumed to be cut mid-way between the centres of the equivalent beams and the inertias may be calculated from the concrete lying between the cuts. If the section is made up of different concretes it may be necessary to assume each has a different modular ratio to allow for the differing elastic modulii and creep characteristics. The assessment of the torsional properties is a much more difficult task, and exact values cannot be found. Various methods given in the literature for assessing these constants lead to widely different values for the same structure [5,6,7], and the differences between these can have an appreciable influence on the results

7. CONSTRUCTION AND COSTS

The Madras Street bridge was opened to traffic in July 1968, and the Kilmore Street bridge in November 1969. Both structures appear to be performing quite satisfactorily.

Table III gives the cost break-down for both bridges. However, in comparing the cost of one against the other, allowance should be made for the greater length of the abutments and the longer skew span of the Kilmore Street structure, associated with a rise in building costs in the intervening period.

8. CONCLUSIONS

Both bridges were constructed using stage-stressed precast beams. A soffit slab containing post-tension cables was cast between the tension flanges of the beams, and the hollow box type of structure was completed with the addition of the top slab. For the Madras Street bridge, where the skew was 39°, it was found that a parallel arrangement of beams at 6 ft centres could be used. However, for the Kilmore Street bridge, where the skew was 51°, a similar scheme was found to be less economic when compared with an alternative arrangement using non-parallel beams.

- In the analysis of the Kilmore Street bridge it is shown that the initial dead-load bending moments acting on the beam might be found from statics, but that these changed with time, owing to creep. Computer grillage analyses were made to check the distribution of the live-load bending moments and to investigate the influence of creep on the structure. For the dead-load case the results of the grillage analysis are compared with the distribution obtained from influence tables for skew slabs. The principal moments obtained from the grillage were found to be about 15% greater than the equivalent values obtained from the slab analysis.

- The grillage analysis was found to be a very convenient tool for design; no specialist knowledge of computers was necessary.

- Some of the limitations of the use of the grillage analysis to represent this type of structure are discussed. In particular it is shown that a model based on a grillage cannot adequately represent the torsional and Poisson properties of the structure. These limitations must be borne in mind when interpreting the results.

- It is shown that creep movements in the concrete may cause a redistribution of stresses in the section and load paths in the structure. Such redistributions occur if the form of the structure is changed after part of the load has been applied. A method of assessing the effects of creep redistribution within a structure is outlined and illustrated with reference to the Kilmore Street bridge. The analysis shows that a substantial change in the magnitude of the bending moments can occur with creep.

TABLE III

COSTS

Madras Street Bridge-1967/68		
Gross deck area	5,280 ft2	
Width	66 ft	
Clear span	60 ft (normal), 75 ft (skew)	
Deck and superstructure	$5.93/ft2	$31,300 total
Abutments and foundations	$4.13/ft^2	$21,800 total
Establishment, site clearance, etc.	$2.76/ft2	$14,500 toatl
	$12.82/ft^2 of deck	$67,600
Kilmore Street Bridge-1969		
Gross deck area	7,200 ft2	
Width	66 ft	
Clear span	60 ft (normal), 95 ft (skew)	
Deck and superstructure	$6.64/ft2	$47,800 total
Abutments and foundations	$5.37/ft^2	$38,700 total
Establishment, site clearance, etc.	$3.20/ft2	$23,000 total
	$15.21	$109,500

9. ACKNOWLEDGMENT

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The electrical measurement of physical properties

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An introduction to the electrical measurement of physical properties with suggestions for transducers and instruments. Advances in solid-state technology have made practicable in portable equipment methods which were formerly limited to mains-operated instruments. The same technology which produced the digital voltmeter and data processor has made available the data logger, a device capable of scanning many transducers at high speed and printing out information as often as required or storing it on magnetic tape if the acquisition rate is too high for a printer.

Since 1967 L. O. Hunter has been technical director of a private testing laboratory and part-time lecturer in illumination at the University of Auckland School of Architecture. He was born in Auckland and between 1940 and 1945 served in the R.N.Z.A.F. as a flying instructor and subsequently as a flying boat captain. Between 1946-55 he was with the N.Z. Electricity Department as electrical draughtsman and then as electrical engineer, mainly on substation construction. He joined Fletcher Holdings design department in 1956 as an electrical engineer for cement works, timber and joinery mills. In 1960 he was building services engineer with a firm of architectural and engineering consultants and won an Angus Award for a services paper. He is a Trades Certification Board examination moderator.

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1. TRANSDUCER PRINCIPLES

A TYPICAL measurement chain consists of transducer (where a physical quantity is converted to an electrical analogue), a transmission link (line or radio channel), and an indicating or recording device, including the transducer power supply and signal conditioning devices.

Transducers which convert some physical quantity to a voltage make use of variation in one of the basic electrical quantities, resistance, capacitance, inductance (and reluctance), voltage and frequency. This latter quantity can be varied in tuned circuits by change of capacitance and/or inductance and in specially-cut quartz crystals by change in temperature. "Pick-up" and "sensor" are alternative names for instrument transducers.

The basic electrical variables employed are:

(i) *Resistance:* Used in strain gauges, temperature and humidity sensors, photo-resistors, microphones and other acoustic and vibration sensors and in ionising radiation detection.

(ii) *Capacitance:* Non-contacting micrometers, dielectric constant measurement, microphones, liquid-level gauges.

(iii) *Inductance:* Measurement of length changes as in load cells, permeability measurement, acoustic and vibration transducers. Variable inductance is achieved by changing reluctance as well as variable mutual or self-inductance.

(iv) *Voltage:* Length measurement by potentiometer action (a sliding tapping on a resistor), velocity (or flow measurement by electromagnetic induction, piezo-electric generation in acoustic and vibration transducers, photovoltaic generation, temperature measurement with thermocouples, magnetic flux measurement and pH measurement of liquids.

(v) *Frequency:* Temperature and humidity measurement with crystals, strain measurement with tensioned vibrating wires, pressure change in vibrating tubes, pulse rate from semiconductor sensors and velocity by Doppler frequency change.

2. TRANSDUCER SELECTION

Transducers chosen for a particular measurement task must be selected with due regard for their linearity, resolution, response speed, measurement range, stability, temperature range and coefficient and their effect on the object being measured.

Accuracy of measurement depends on the resolution and the calibration stability of the system, i.e. the transducer and associated instrument.

A newly acquired instrument and transducer should be carefully tested and results noted on a calibration record. Subsequent calibrations and their rate of drift should indicate to the user the degree of measurement accuracy to be expected.

For most purposes there is a choice of transducer, i.e. strain gauges or differential transformers as in load cells; thermocouples or resistance sensors for temperature. The cheapest system which will meet all requirements is the obvious choice.

3. INTERCONNECTIONS

Most transducers produce very low voltage outputs (a few micro-volts with strain gauges) and long leads are subjected to induced voltages from external fields or produce their own small voltages at junctions or temperature gradients.

High-impedance circuits should use shielded cables to eliminate electrostatic induction. Low-impedance connections can be made with twisted cable pairs balanced to ground to reduce electromagnetic induction.

Bridge-connected transducers (strain gauges) can be excited at a.c. and the frequency placed well above 50 Hz as the instrument can be arranged to reject this frequency and its lower harmonies as well as not respond to direct voltages.

Instruments connected to low impedance d.c. devices (thermocouples) which are responding to slowly varying quantities can be made to have a decreasing response with increasing frequency and thus be little affected by 50 Hz interference.

It is also important to insulate leads and transducers so that earth loops are avoided. Interconnections should have only one ground point, preferably at the instrument. Flexible cords for instruments may be 3-core when the metalwork may be expected to be grounded to the earthed conductor or 2-core when double insulation is used.

When long connections are used a ground point should always be established otherwise, in some circumstances, excessively high voltages can be electrostatically induced causing insulation failure, or accidental connection to live conductors will produce an electric shock hazard.

4. DISPLAY AND RECORDING

The visible output of the transducer instrument can be either an indication on a moving pointer meter, a digital display or cathode-ray oscilloscope screen. If a record is required, the moving pointer can be arranged to produce an inked record on paper in the form of a roll or circular chart. There are several systems used to reduce pen to paper friction; the chopper bar where the pointer is pushed against the self-inking paper about once per second, a light beam on photo-sensitive paper, a small gap bridged by ink or a servo-mechanism.

The data logger record is usually a typed record provided by a small printer. Large systems which formerly produced many records on a 10 in. wide chart can now have a printed record. A common application is the recording of temperatures at different points in a power station or industrial plant. The data logger scans the thermocouple outputs, converts the small d.c. analogue voltage to a digital figure and prints out the value at some predetermined rate.

There is no fundamental limit to the number of points which can be scanned but 10 and 100 are standard and these can be scanned once a second. Information which changes too rapidly for a printer can be stored on magnetic tape and then printed out at a slower speed.

The continuous recording of rapidly changing phenomena is a quite different problem from that solved by the data logger which, even if monitoring one point only, is limited in recording speed by the mechanical printer.

A self-balancing potentiometer might record up to 10 Hz (with a small excursion amplitude), a pen oscilloscope to 100 Hz, a light beam oscilloscope to 1000 Hz, a tape recorder (video) to 10^7 Hz and an oscilloscope to 10" or 10" Hz with sampling techniques.

Very brief phenomena can be recorded on a storage oscilloscope screen and photographed at leisure.

5. POWER SOURCES

Many portable instruments designed for use with specific types of transducers have solid-state electronics and sufficiently small power requirements to operate from batteries. The requirements of the transducer, often at audio frequencies, will be provided by the associated instrument. All battery-operated instruments should have a voltage check and, within the allowable voltage range, be stabilised so as to remain within claimed measurement accuracy unless they are bridge types, where battery voltage variation does not produce a change in reading.

Some very low current devices such as photo, meters use 1.3 V mercury cells which hold a reasonably constant voltage and then die suddenly. The common l.5 V carbon/zinc cell has a new voltage of 1.6 V and should be discarded at about l.35 V.

Rechargable nickel/cadmium cells are provided in some instruments while larger portable instruments can be operated from 2 V lead/acid secondary batteries which must be placed on charge before falling below 1.8 V per cell.

Instruments designed for operation on 230 V a.c., over the usual range of – 15 to 10% as found in New Zealand can be used in the field, powered by portable engine/generators or battery operated inverters.

6. TRANSDUCER CALIBRATION

All measuring devices, regardless of age or quality, must be calibrated at least in the range where measurements are proposed. Most transducer and instrument combinations can be checked for accuracy by some means which is simple and basically accurate. Examples of calibration procedures are:

(1) *Temperature:* Spot checks using boiling and freezing points, with allowance made for altitude in the case of the former. Continuous calibration with a stirred and heated water or oil bath with a master mercury-in-glass thermometer for temperature measurement. Always be suspicious of thermocouples, they can be contaminated, corroded, oxidised or partially open-circuited.

(2) *Humidity:* Spot checks over saturated salt solutions or wet and dry bulb psychrometer for continuous measurement of humidity. B.S. 3718 lists a range of salt solutions. Sufficient exposed liquid surface is essential with adequate air movement.

(3) *Length:* Spot checks with gauge blocks. These can be used to check a precision dial micrometer and then lengths from 0.0001 in. up to several inches can be checked. Few transducers exceed 10 in. in measurement length. A working dial micrometer should always be regarded with suspicion because of the possibility of the rack teeth having been damaged and a local error introduced. For measurement to ±0.001 in., a precision vernier calipers checked with gauge blocks is useful and 10 in. or more can be obtained.

(4) *Force transducers:* Are checked with certified weights. Weights are usually accurate but are not so when corroded or dirt laden.

(5) *Pressure transducers:* Are checked with dead weight testers. These are oil-filled systems which include a cylinder with plunger of precision diameter (usually providing an area which is a simple fraction of a square inch) with a weight platform. Friction is eliminated by spinning the plunger while in use.

(6) *Velocity transducers:* Are calibrated by reference to secondary standards of time and distance. These quantities can be measured to any necessary standard of accuracy.

(7) *Radiant energy:* A radiometer disc can be calibrated by introducing a known electrical energy, i.e. a small heating element is bonded to the back of the disc and a sensitive wattmeter used to measure the energy input. Alternatively, the disc can be arranged to "see" a surface of known area and temperature.

A photometer is calibrated with a standard lamp and an optical bench. A useful rough check is to use a bare 100 W lamp away from reflecting surfaces; at 1 ft distance, the illumination level is approximately 100 lumens / ft^2 at rated voltage.

The calibration of transducers for other forms of radiation is regarded as coming within the scope of physics rather than engineering.

(8) *Sound pressure:* Meters are calibrated with portable calibrators provided by the manufacturer. Where litigation is likely sound pressure level readings should not be taken without calibrating before and after measurement.

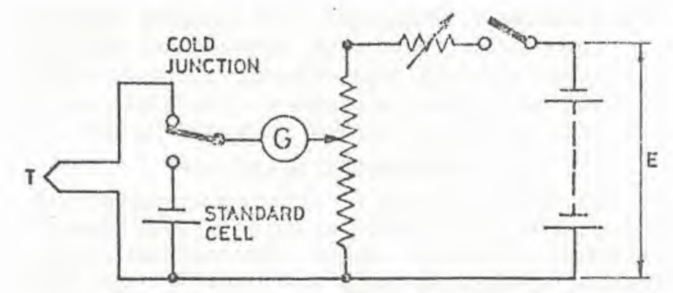

Fig. 1: Thermocouple voltage potentiometer.

(9) *pH meters:* Are calibrated with standard buffer solutions of known pH value.

7. TEMPERATURE

The sensors available include resistors of positive and negative temperature coefficient materials, thermocouples, and for ultra-precision measurement, quartz crystals which are cut for a linear temperature/frequency characteristic.

7.1. Thermocouples

There are approximately six junctions of dissimilar metals in common use which produce a small potential difference between hot and cold junctions. These are: platinum/platinum 10% rhodium, platinum/platinum 13% rhodium, chromel/alumel, iron/constantan, copper/constantan and chromel/constantan. Copper/constantan has a useful range from —190 to 390° C with an output of approximately 1 millivolt/25° C.

The output of all thermocouples is non-linear and tables of millivolts/temperature must be used for the correct reference junction temperature. This is usually tabulated for 0° C but can be calculated for other junction temperatures. For example, if a transistor controlled reference junction of 50° C is used, 2.035 mV (copper/constantan), is subtracted from 0° C tabulated values to obtain the millvolts corresponding to temperatures above 50° C.

Iron/constantan has the highest output (I mV/20° C) 15ut is less reliable than copper/constantan as the voltage/temperature coefficient changes with slight contamination of the iron.

Chromel/alumel is useful for higher temperatures and has a useable range from — 190° to 1370° C although prolonged use at high temperatures results in a short life—particularly with small gauges of wire. The output is 1 mV/25° C at 25° C and falls to 1 mV/29° C above 1300° C.

Platinum/platinum +10% rhodium allows an extension of range to 1760° C at an initial sensitivity of 1 mV/147° C.

Thermocouples and extension wire can be obtained in gauges between 14 and 26 s.w.g. The extension wire is not as pure and thus not as accurate but can be used as the thermocouple by soldering, brazing or welding, according to the maximum temperature.

The measuring instrument is desirably a high-impedance voltmeter or potentiometer which draws negligible current, otherwise the instrument must be calibrated for the actual length of wire used. The most popular recording instrument is the self-balancing potentiometer (Fig. 1) and instruments designed for thermocouple use are available with internal cold-junction compensation.

7.2. Resistance thermometers

Resistance thermometers are useful for precise measurements using pure strain-free platinum and for cryogenic temperatures using carbon and semi-conductors. Where a sensitive indication over a small temperature range is required thermistors can be made with large and very non-linear positive or negative temperature/resistance coefficients with the rapid change of resistance point positioned over a wide temperature range by adjustment of composition during manufacture.

The electrical measurement technique is essentially that of resistance measurement; half or full section resistance bridges may be placed in the sensor with combinations of high and low and/or positive and negative resistance/temperature coefficient materials.

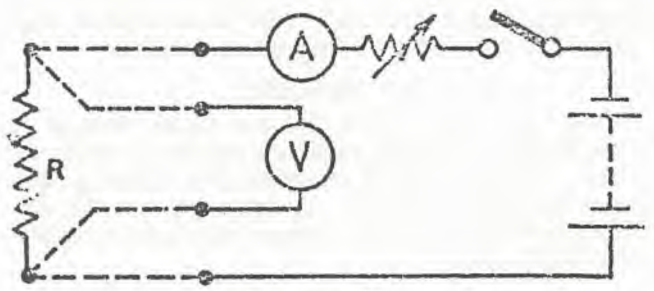

Fig. 2: Resistance thermometer.

The voltage produced can be converted to temperature using the calibration information when the bridge is supplied at constant current with separate conductors (Fig. 2) and thus eliminate the effect of lead resistance.

8. HUMIDITY

A humidity sensor may use the change in length with humidity of a hair or area of a membrane, the change in electrical resistance of a cell, a change in dielectric constant or the comparison between voltages produced by wet and dry temperature sensors. This latter system is inconvenient in that provision must be made for drawing air over the wet sensor and also maintaining water in the sensor wick. A quartz crystal with hygroscopic plastic coating can vary in resonant frequency with humidity. For control purposes, there is a selection of proprietary humidity transducer systems which may have their set points adjusted so that switching will take place reliably at an approximately constant relative humidity (provided the sensor is maintained in good condition).

For measurment purposes down to low relative humidities, a wet and dry temperature sensor (psychrometer) is a convenient method and reliable, provided temperature is measured accurately and the wet sensor is kept damp with a good airflow over it.

9. LENGTH

Many variables can be reduced to a change in length, e.g. force, pressure, acceleration. The electrical variables available in sensors are resistance, both strain and slider variable; inductance and mutual inductance; capacitance and voltage.

9.1. Strain gauges

Very small changes in length (micro-inches/inch) can be measured using strain gauges. Resistance change occurs because of strain modified resistivity as well as length and cross-sectional area change. Gauges are available in wire, foil and semiconductor form in a large variety of shapes and temperature ratings for cementing to surfaces subject to strain. Unbonded gauges are used in some transducers. These are wound on pins and air insulated.

The electrical measurement of strain is that of very small resistance changes while attempting to eliminate relatively large and often uncontrollable temperature induced resistance changes. Usually two similar gauges, one strained, one unstrained, are placed in the same temperature environment. These form two arms of a bridge with the measuring instrument completing the circuit (Fig. 3). Temperature-compensated gauges are available for use on a limited range of metals. If these are used a single gauge mounting is sufficient. In order to eliminate thermal voltages, the usual practice is to excite strain gauge bridges with a.c., and rectify synchronously to enable strain polarity to be observed.

Wire-wound strain gauges, with a gauge factor of about 2.0 (ratio of resistance change to strain) are available with coefficients of resistance to match the expansion of steel or aluminium. Foil gauges can be obtained in a greater variety of shapes and sizes, in materials to match steel, stainless steel and aluminium, and also with gauge factors around 2. Semiconductor gauges have high sensitivities (gauge factors +50 to +250 or — 100) but high resistance/temperature coefficients.

Gauges are available on paper, polyester, bakelite or asbestos for use with nitro-cellulose, phenolic, polyester, silicone, synthetic rubber, expoxy resin or cyano-acrylate cements according to the base material to which the gauge is to be cemented and the maximum working temperature.

For example paper gauges with nitro-cellulose cement can be used up to 70° C, polyester with polyester cement up to 170° C or bakelite with phenolic resin to 180° C. An epoxy based foil gauge with

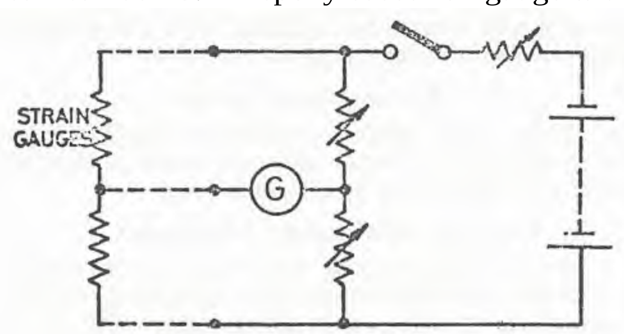

Fig. 3: Resistance bridge.

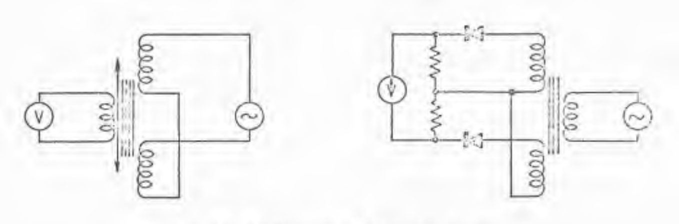

Fig. 4: Differential transformers.
(output polarised) (output unpolarised)

epoxy resin cement, cured at room temperature is usable up to 80° C. This is a general purpose combination for use on metal.

The techniques for cementing on a gauge are those of any good glued joint; clean, roughened, grease free surfaces, the gauge clamped, correctly oriented, and with a thin glue line, until cured.

All conductors and joints must be dry, electrically insulated and moisture-proofed with wax or epoxy. Prior to taking readings, check insulation resistance to ground with an ohmmeter and repeat this check for subsequent strain readings. Always note ambient temperature and, where possible, read strain at the same temperature as when checking unstrained gauge resistances.

The manufacturer supplies gauges with the resistance accurate to about ±1% or slightly better. Provided that unstrained readings can be taken after cementing in place the bridge can be balanced and the adjustment noted. The gauge factor must usually be accepted but it is likely to be within ±1% which is usually enough for engineering purposes.

9.2. Inductance gauges

Small distances in the range 0-100 microns (such as aluminium anodising thickness) may be measured using small air-cored coils excited at radio frequencies. Coil inductance is reduced by proximity to a non-magnetic conducting surface. The change in inductance can be measured by bridge techniques as an out-of-balance voltage on a calibrated meter.

9.3. Mutual inductance gauges

Very sensitive displacement transducers are made using a moving coil or magnetic core between the two fixed coils with opposing fluxes forming a differential transformer (Fig. 4). In the central balanced position, the coil has zero voltage induced. Force transducers covering wide ranges are available using this principle which rivals the strain gauge for sensitivity.

9.4. Capacitance gauges

These have limited engineering application in the measurement of small distances although they are useful in certain liquid level applications.

9.5. Variable voltage transducers

A wire-wound resistor with linear or rotary slider is a simple and practical method of convering displacements of up to 10 in. or more to voltage (Fig. 5). A constant-voltage power supply is required with a suitably calibrated voltage-operated indicator or chart recorder. Resolution depends on the pitch and linearity of the winding; figures of 0.001 in. and ±0.5% are typical. The slider does not need to be against the zero stop for the recorder to read zero, a suppressed zero is obtainable from the zero adjusting potentiometer in the recorder.

10. FORCE

Force transducers operate by using an elastic element to convert to strain and then transducing as previously described. The common methods of measuring length are used with strain gauges and differential transformers being the most usual.

11. PRESSURE

As for force, except that the elastic element is a diaphragm, a bellows operating against a spring or a bent, twisted or coiled Bourdon tube. Strain gauges can be cemented directly to a diaphragm and the output calibrated for pressure without additional mechanical complication.

12. LIQUID LEVEL

There are a number of alternatives to a simple float operated rheostat or potentiometer. These are:

(a) Variable capacitance with insulating liquids.
(b) Variable conductivity with electrolytes.
(c) Change in thermal conductivity in a string of thermistors.
(d) Measurement of pressure at the bottom of the tank.
(e) Sonic distance measurement.
(f) Radio frequency cavity volume measurement.

13. VELOCITY

Linear velocity may be converted to voltage by electromagnetic induction (Fig. 6). Angular velocity can be converted to frequency using the same principle or to pulse repetition rate with a magnet passing an inductor.

A useful flow transducer uses the principle of electromagnetic induction with a conducting fluid

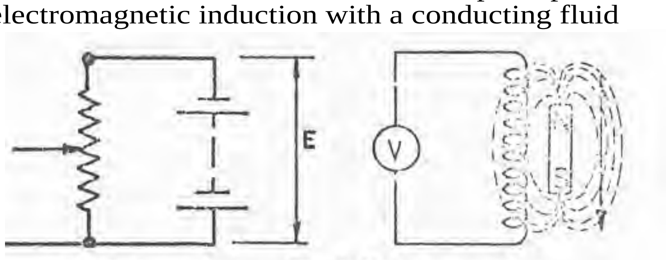

Fig. 5: Potentiometer. Fig. 6: Velocity transducer.

moving in a magnetic field and inducing a voltage proportional to velocity. Another type uses a propellor with a pulsed output.

Relative velocity can be measured by the frequency shift of reflected acoustic or electromagnetic energy or, for small objects such as bullets, by the measurement of time over a known distance using photo-electric devices or electrical coils to produce a voltage pulse at each passing point.

14. RADIANT ENERGY

Radio: Radio energy can be measured by the voltage induced in a di-pole conductor of 1 m length.

Heat: Heat energy can be measured by temperature change in resistors (thermistors) or thermocouples in a radiometer. This is a shielded black disc which undergoes a temperature rise related to the energy output of a heat emitting area in the field of view.

Light: Portable photometers use selenium/iron photo-voltaic cells and microammeters or photoresistors with primary cells (Fig. 7).

Spectral response should be corrected to approximate to the eye and, for illumination level photometers, the cell should respond to illumination received over a hemisphere.

Ultra-violet energy: Photo-emissive tubes with caesium or rubidium telluride photo-cathodes and lime glass or silica windows are used for measurement of ultra-violet energy.

X-rays: These can be detected by fluorescence or photographic emulsions and measured with ionisation chambers. The latter are gas-filled metal containers with insulated wires on the axis. With a potential difference between wire and cylinder, the current flowing is proportional to the electrons emitted by the x-ray photons. A similar chamber but with a higher potential difference is the geiger tube used to detect gamma ray photons. Electron multiplication is allowed to occur in these tubes.

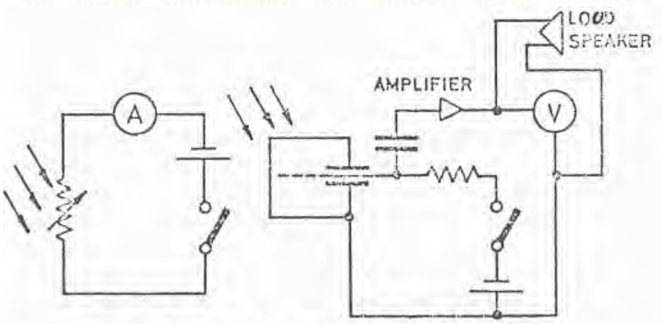

Fig. 7: Radiation detector. Fig. 8: Geiger counter.

Semiconductor crystals are available for use as x-ray sensors.

Gamma rays: In small numbers these are detected with geiger tubes (Fig. 8) while higher levels of flux may be detected and measured with scintillation devices. These use crystal fluorescence as the energy convertor and photo-multiplier tubes to amplify the light output.

15. SOUND

Sound pressure is converted to voltage in microphones, amplified, rectified and displayed on a microammeter. The decibel scale used with sound level meters is a logarithmic pressure ratio based on a zero decibel level arbitrarily made equal to 0.0002 dyne/cm² (microbars), because this is about the lower limit of human hearing.

The most linear microphone available is probably the condenser (capacitor) variety. This requires a polarising voltage and a very high input impedance amplifier. Other microphones use piezo-electric crystals, magnetic induction, or variable resistance using graphite capsules as in telephones. Amplifiers use A, B, C, D and linear scales which are modifications to the frequency response. The "A" scale most nearly matches the ear at low levels and is used as a single figure in assessing noise level.

16. MAGNETIC FLUX

Hall effect semiconductor transducers are used to produce voltage proportional to flux. The sensor is in the form of a thin semiconductor strip with a controlled longitudinal current. The voltage appearing between the sides is proportional to magnetic flux.

17. CONCLUSION

Almost anything can be measured electrically at a distance; the most unsatisfactory quantity being moisture content, particularly at low levels. All reputable manufacturers of quality systems produce instructions on the use of their instruments and transducers and some produce very detailed directions for technicians. Engineers will find bonded-foil strain gauges one .of the most versatile and useful transducers.

18. REFERENCES

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ARTHUR, K. (1970): *Transducer Measurements.* Tektronix Inc. (A qualitative and useful introduction with the accent on oscilloscope measurements. Contains step-by-step instructions for mounting strain gauges.)

Westport harbour entrance

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This paper summarises investigations into the causes of entrance shoaling at Westport and the efficacy of dredging in relation to alternative means of improvement. Recent extensions of the main breakwaters combined with narrowing of the entrance are described, including the effect of these works on the entrance stability.

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H. Fyson is chief construction engineer (civil), Ministry of Works.
Biographical details of both appeared in the July 1971 issue, pages 196 and 198 respectively.

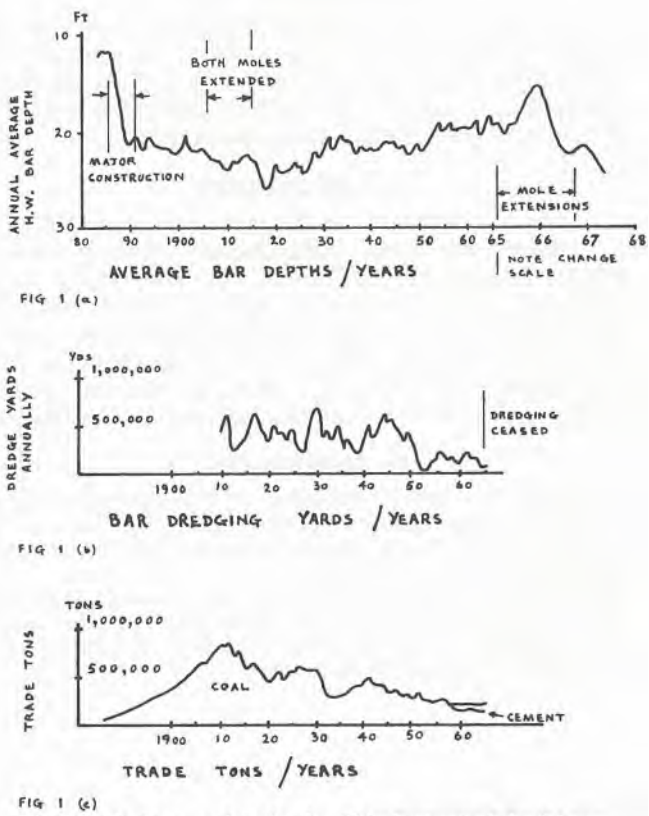

Fig. 1: (a) Average bar depths; (b) Amount of dredging; (c) Annual trade.

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1. HISTORY

FOLLOWING recommendations by Sir John Coode in 1880, the entrance to Westport harbour was trained by two main breakwaters supplemented by river training-walls over the 1886-1900 period' giving spectacular improvements in entrance depths. Various extensions of both breakwaters were undertaken between 1906 and 1916 finishing with the west mole about 200 ft in advance of the east mole. As a further means of combating decay in depths, dredging was commenced in 1904 and was not discontinued until 1966 when further breakwater extensions were in hand. From 1910 the coal trade was declining making it difficult to support capital works for greater entrance stability (Fig. l). History up to 1946 is well covered by F. W. Furkert[2].

2. THE COASTAL DRIFT

The heaviest seas come from the west and the southwest and the winds which generate these seas have wide open expanses of water over which to blow. Long waves of great eroding and transporting power are common on this coast. Over a two-year period of recordings, only 5% of waves of periods less than 7 s (corresponding to a wave length of 300 ft) were observed, and wave periods of 13 to 14 s (corresponding to a wave length of l,000 ft) were not uncommon. There is a nearly continuous movement of coastal detritus from the south to the north, and the residual material that is not so finely ground as to be drawn down into deeper water, finally spills into western approaches to Cook Strait along the line of Farewell Spit. The Steeples reef west of Westport reduces wave heights and all seas approach the entrance from practically the same direction—northwest (Fig. 2).

The prevailing humid onshore winds precipitate a heavy rainfall on the flanks of the high ranges only 15 to 25 miles from the coast. The coastal rainfall at Westport and Greymouth is 70 to 80 in. per year and only a few miles inland from these ports rises to 120 in./year. There are 19 peaks in the Southern Alps rising above 10,000 ft and the annual snowfall and rainfall at these higher altitudes is unknown. The elevation of the mountains, the steep seaward slopes, frost action and heavy rainfall result in an enormous quantity of detrital material being carried to the sea, where again the attrition of the heavy seas reduces the boulders and heavy gravels discharged on the beaches into sand, often before the next river mouth to the north is reached.

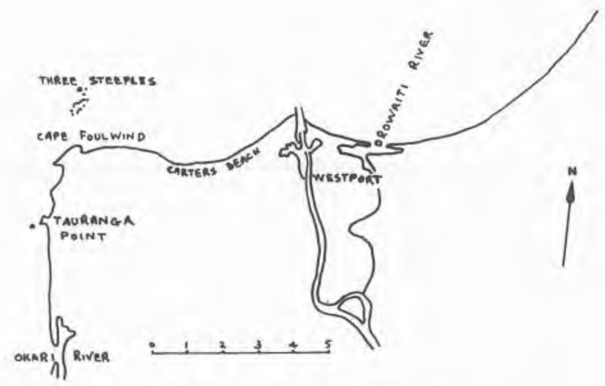

Fig. 2: Westport roadstead.

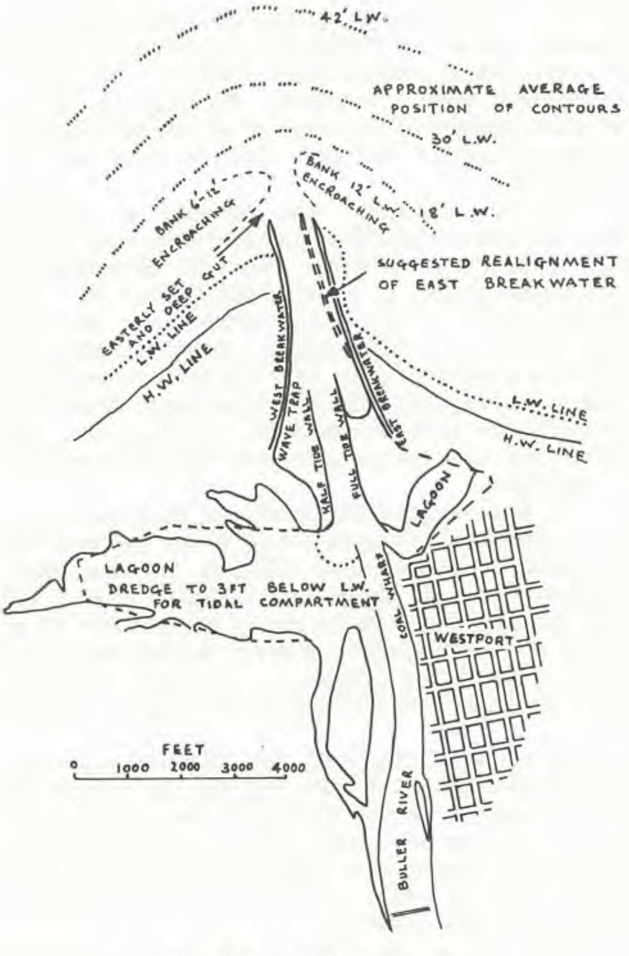

Fig. 3: General harbour layout.

3. BASIS OF INVESTIGATIONS

In 1946 Rendel, Palmer and Tritton, represented by Buckton and Clark, recommended improvement in tidal compartments, investigation and subsequent dis-:ontinuance of dredging unless its value was proven, external narrowing of the entrance moles and experimental dragline dredging. This report was examined by Newnham, Furkert and Wood who supported improvement of tidal compartments with narrowing of the entrance inside the present moles if this first work proved insufficient. They considered that dragline work at the entrance was not practicable and that intensive dredging tests were not possible, but that dredging should continue.

Proposals were prepared for tidal compartments and for a new dredge, but it was recommended to the Minister of Marine that the investigations necessary to supply reliable data and thus resolve the many points which were in doubt should first be undertaken. Investigations on an intensive daily basis were undertaken during 1951-53 (Fig. 3).

4. BAR DREDGING

Existing dredging and other records were analysed and disclosed:

(i) Years of intensive dredging did not correspond with years of improved depths—intensified dredging had often been undertaken because of poor depths which nevertheless remained poor.

(ii) Years of good bar depths occurred up to 1916 whenever mole extensions were proceeding. Intensive dredging between 1929 and 1931 did not check the closing phase as the storage for drift material west of the entrance filled up. Since then depths have fluctuated but not deteriorated further. (Note that since 1950 daily echo soundings in place of fair weather lead-line soundings have reduced the calculated annual average by about 1 ft, representing a more accurate average but no physical change.)

(iii) On average the dredge worked on the bar 135 days per year (566 working hours) and only 50 days/year on bars of restrictive depth (208 working hours). Reasonable utilisation of bar-dredging plant was not obtained because of the weather and the inability of the dredge to operate on shallow bars except around high water (Fig. 4).

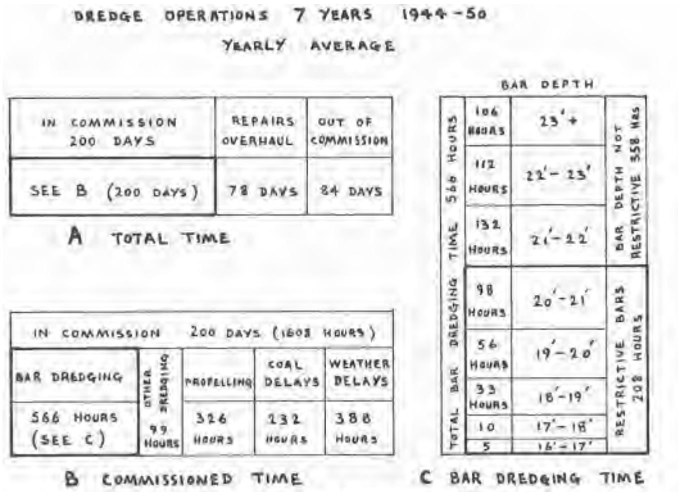

Fig. 4: Analysis of dredging operation.

The 21 years of special study disclosed:

(i) Sixty dredging tests on bar shoal areas showed improvement in half the tests only, and in some of these the areas immediately to the east were shoaled by settlement of sand disturbed. In total, in one-third there was no change and in one-third depths decreased. Improvements after removal of 1,000 to 3,000 yd rarely exceeded 6 in. and on cessation of dredging the bar shape generally re-formed within two to four tides.

(ii) Set, river and tidal flow were the major factors in determining the bar formation. Easterly sets are due to winds mainly from south-west to west, piling up the coastal waters against the breakwater (Fig. 5). These sets were observed at the surface on 103 days of the year ended March 1952 and westerly sets on only 21 days. It is probable that easterly sets occurred at depth on a much greater number of occasions. Under easterly sets the bar assumed a bisected form with a deep gut extending from just off the end of the west mole in a north-easterly direction across the channel. This gut was caused by the combination of river and tidal flow. The scour of this set was such that the bottom of the gut was frequently 30 ft below low water and 10 to 15 ft below the average bar depth. Between these two channels a strong sandbank formation was set up with depths of 6 ft to 8 ft on the Half West line tapering downwards into mid channel or further eastward.

No practicable dredging programme would remove this sandspit when formed by set currents sufficient to scour the cross channel so far below the mean bar formation, and the bar would be rapidly replaced from the vast storage area to the west if dredged away. Several river freshes of the order of 30,000 to 40,000 cusec were necessary to overcome and fill the gut which persisted when surface sets were not recorded and was found to exist over two-thirds of the year.

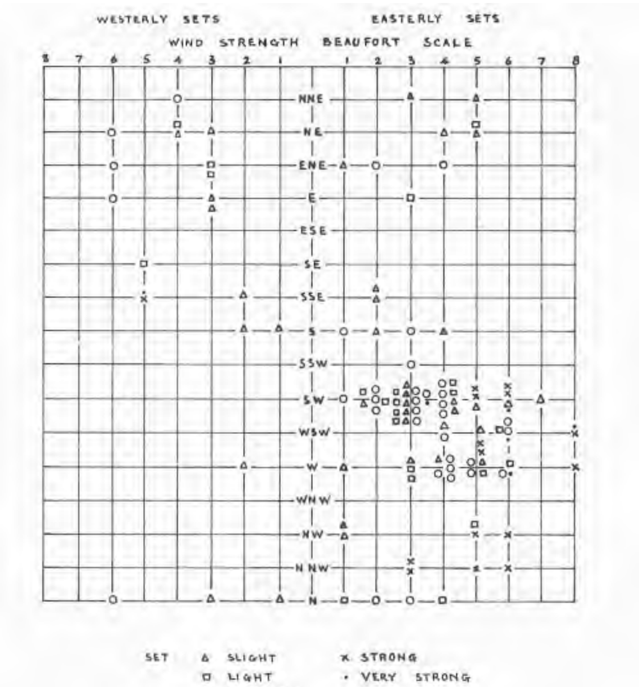

Fig. 5: Set and local wind relationship.

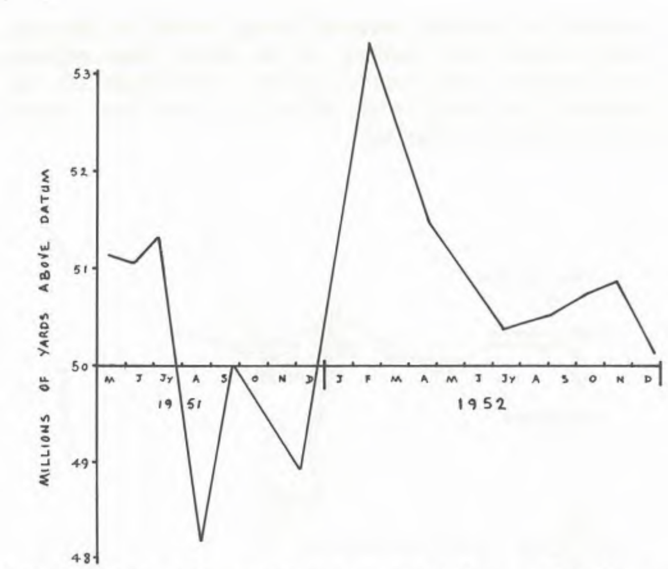

Fig. 6: Erosion and accretions. over 1 mile² area off harbour entrance.

(iii) River freshes of the order of 30,000 to 40,000 cusec were usually required to obliterate the set gut and form a smooth "paddock"-type bar. The restrictions to navigation then tended to be more within the capacity of the dredge. In the course of a few days river freshes causing this formation have removed 20,000 to 40,000 yd of material from the bar area giving improvements which are of greater consequence than a fortnight's dredging. In one or two days changes of the order of 40,000 yd^3 occurred in the volume of the bar when set-type bars re-established.

(iv) The roadstead was sounded at monthly intervals whenever possible at or near high-water spring tides to eliminate variables. After full allowance for error, aggradations of up to 4,000,000 yd in a two months' period occurred and other lesser but very large erosions and aggradations occurred continually. From the conditions these were presumed to be northward movements generally in gushes which could have occurred more than once between recording dates. The record (Fig. 6) takes no account of material remaining in suspension.

(v) Recording of the volume of sand moving on the sea floor and in suspension on to the bar area was hampered by the extreme difficulty in obtaining a continued series of samples of sediment over a full cross-section parallel to the line of the entrance at all depths, and corresponding velocity readings under any but the calmest sea conditions. The sand in the bar area is very fine, over 90% passing a 120 U.S. standard sieve.

On a day of practically flat calm the volume of sediment transported directly into the bar area in the layer between 3½ ft and Φ ft from the bottom only, was at the rate of 1,200 yd/day. No bed load or higher level samples were taken, but the volume transported below the 3½ ft level would be much greater than in the layer observed.

With seas of the order of 3 ft to 4 ft in height, the volume of sediment suspended at the surface alone was found to be in excess of 1,200 yd/day. In these circumstances, the volume of sediment moving at lower levels could be enormous, but it is doubtful if it could be checked accurately with means at present available under the conditions holding at the harbour entrance. In the latter stages some success was obtained using a trailing hose, pump and filter.

It was concluded that although a dredge for internal port maintenance wolud be required it would be useful only in rare circumstances on the bar.

5. TIDAL COMPARTMENTS

This scheme proposed removal of approximately 3,000,000 yd to dredge lagoon areas (250 acres) to 3 ft below low water.

The tidal volumes considered are given in Table I

which shows an increase in total flows of about 20% above the existing flows. Fresh water of normal river-flows lies on top of the sea-water tidal wedge. Tidal compartment flows would therefore act on the bar over the greater part of the year, and indeed at any time when the river in fresh is not directly eroding the bar. With a fresh of over 30,000 to 40,000 cusec in the river, the tidal wedge is probably forced out of the river. At this stage river flow directly erodes the bar.

A. J. Clark, a partner of Rendel, Palmer and Tritton, calculated the average improvement of bar depths at l.0 to 1.6 ft on theoretical considerations. Much of the restriction in bar depth is due to the spit of sand which extends from off the tip of the west mole across the entrance when set is prevalent and it seemed that the improvements on the shallow section of the bar could be greater than the average values referred to above, because of the restriction to flow offered at th's point. At times of low river-flow, the bar depth is maintained largely by tidal action and improvement in scouring action on the bar at such times is of primary importance.

6. NARROWING THE ENTRANCE

What has been said above regarding tidal compartments is also applicable to tidal flow from a narrowed entrance in that similar improvements of tidal scour may be obtained. If the entrance were narrowed by one-sixth of the mean tide sectional area, the mean tidal velocities would be increased by 20%.

Narrowing of the entrance also offers an opportunity for improvement by prolonging and increasing the velocity of river freshes so that all river flows will be similarly increased in average velocity by about 20%. At present a flood of 40,000 cusec is sufficient to commence considerable scouring action and if prolonged or followed by other freshes even of lesser dimensions will markedly increase the depth of water on the bar. With restriction of entrance width similar scouring action should be available at 33,000 cusec, ar conversely scour from a 40,000-cusec flow would pe increased by 20% and would have the same effect m the bar as at present until it fell to this lower figure of 33,000.

By the combination of improvement by the ncrease in velocity of both tidal and river-outlet flows t was expected that average depths throughout the year could be improved by not less than 2 ft, about aalf of which would be caused by tidal flow and the Pther half by river flows.

Narrowing could be most cheaply done by internal -ealignment of the eastern mole but seaward extensions pf both moles as finally adopted has the advantage of additional temporary increase in depth by retardation Pf the littoral drift.

In spite of the findings of the 1951-53 investiga-ions bar dredging was sufficiently entrenched for the dredge to remain in commission throughout its useful ife. It was represented to Government that with the .conomic difficulties facing the West Coast, and the mminent retirement of the dredge, extensions and narrowing of the breakwaters would give some permanent improvements at a cost comparable with iredge replacement and employ a work force in the Buller area. This was approved in 1963.

7. MODEL STUDY

A model of Westport habrour was made at the 2entral Laboratories, Gracefield, comprising an area just downstream from the Buller bridge to a point approximately 4,200 ft seaward from the breakwaters, and including the eastern and western tidal lagoons. The model had a fixed bed. No provision was made for tide or wave generation, and fresh water was used throughout. The sea level was varied by means of a hinged weir and it was possible to reproduce an msterly set. The horizontal scale was 1:516 and the vertical scale 1:120.

The purpose of the model was to determine the most satisfactory alignment for the breakwater extensions. Thus the model extensions were movable, and were heavy enough to be stable under the flow. It was assumed that a river flow of the order of 55,000 zusec was needed to produce an improvement at the bar, and it was found that this volume also produced velocities in the model which were readily measurable with the equipment available. In reality, bar improvement is noticable with somewhat lower river flows—say 30,000 cusec—but the results obtained are not thought to have been far wrong on this account.

Velocity distributions were measured at distances pf 800, 1,000 and 1,200 ft from the ends of the breakwaters. With the existing breakwaters (before extension) the maximum velocity was about 4 ft/s and was fairly uniform over a width of 300 to 400 ft. None of the alignments tested was exactly equivalent to that which has been built, but the one which approached it nearest was the one which gave the most symmetrical pattern of velocity distribution. As would be expected, the increase in velocity was roughly in the same proportion as the decrease in entrance width. The reason for studying the velocities for some distance beyond the ends of the breakwaters was that the bar forms in a variety of locations, sometimes close into the mouth, at others up to 2,000 ft out.

The alignment adopted ensured that river flows of scouring velocities could be expected to move with a straight, non-turbulent action, carrying the maximum effect out to sea over the bar areas.

8. DESIGN OF BREAKWATER EXTENSIONS

To give maximum concentration of flow the ends of the breakwaters were made coterminous and the entrance width was reduced by one-sixth. Narrowing was effected by extending both breakwaters, the extensions converging inwards. The western breakwater was extended 300 ft requiring an extension of 500 ft for the eastern breakwater (Fig. 7).

The existing breakwaters had successfully withstood storms and earthquakes for many years, the granite blocks being stable in position and hardly weathered at all. Average batters of the rock mounds, transverse to the river, are between 1 in 1+ and 1 in 1½. To determine the shape of the new work it was possible to use the shape and rock size which had hitherto been successful. There are a number of empirical formulae which can be used to determine the shape and rock size of rubble-mound breakwaters, that of Irabarren Cavanilles being perhaps the best known, and this was examined as a check. This formula gives a value for the size of rock on the outer layers, in pounds weight, W, as:

$$W = k(2h)^3 \gamma \gamma 1^3/(\cos\alpha - \sin\alpha)^3 (\gamma$$

where k is a constant, 0.015 for natural stone
0.019 for artificial blocks
y is the density of the blocks
γ_1 is the density of water
a is the angle with the horizontal of the sloping outer face of the breakwater
2h is the wave height, crest to trough

Looking first of all at concrete as a possible material:

k =0.019 y=150 (say) γ = 64

Taking a slope of 1 in 1.5, cosa = 1.5/(3.25)i
sina = 1/ (3.25)1

2h was found by observation to be 10 ft.

From these figures, W comes to about 25 tons.

Granite, as available locally, has a density of 173 lb/ft³ and this increase in block density obviously has a most marked effect, as the block weight is inversely proportional to the cube of the weight in water; for the same wave height, rock slope, and using the other constant k of 0.015, the size of granite block is only 11 tons as compared with 25 for concrete.

It was decided to round off the minimum block weight for granite to 10 tons in this case. Observation of the existing breakwaters showed that this size of block at this slope had not been displaced. Concrete blockwork of any type would have been more costly. The ratio of greatest to least dimension was specified as 3 for 10-ton blocks, and 4 for blocks of 15 tons and over.

The outer protection had a minimum thickness of 10 ft to give not less than two layers of large rock. Beneath this, the size of the core material was reduced to allow the use of quarry wastes. The end slope was

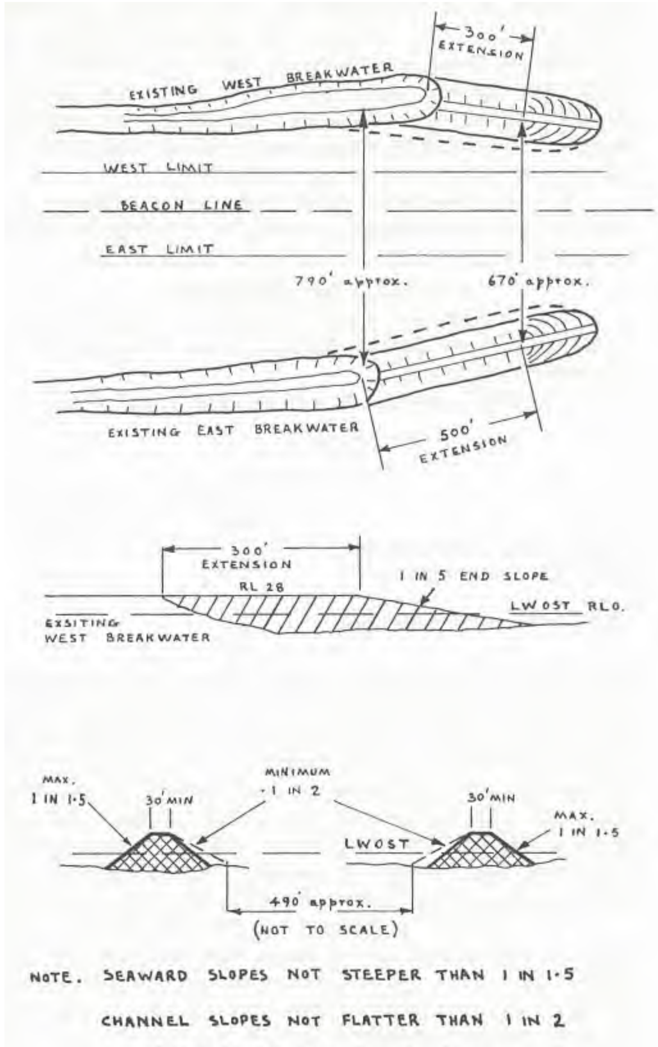

Fig. 7:Details of mole extensions.

made 1 in 5 in an attempt to reduce the intensity of the set current and the corresponding intensity of bar formation.

9. CONSTRUCTION

Records showed that the important factor in such work at this site was the speed of placing rock in 3rder to avoid the formation of a scour hole at the tip A the new work. When this happened, far more rock was placed than was really necessary. When the sea Ded was at a high level and a good length of rock base placed before scour occurred then good advance and :heap construction cost could be maintained.

Works of this sort are usually carried out by the larbour authority's own staff, or by contract based on he cost of materials used. But there seemed to be an opportunity here for a well-executed contract to be run m an end-product basis, allowing the contractor to benefit from good methods and the harbour authority to 3enefit from knowing the cost before the work started. k simple contract in which the schedule consisted of only three items—establishment on the job, construction of breakwaters (per foot run), and clearing ip on completion was adopted. The contract allowed

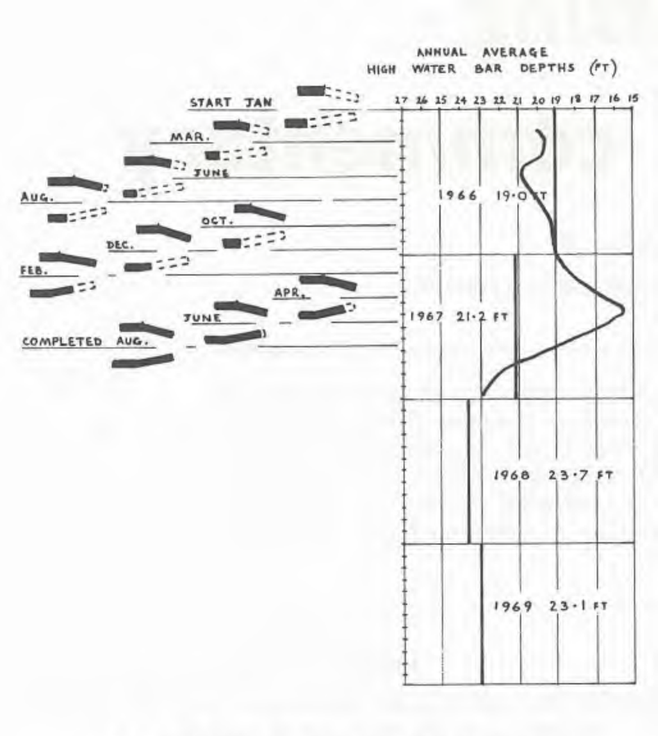

Fig. 8: Bar depth changes during construction.

payment for the work done only to the extent of 75% of the measured amount on first completion, the remaining 25% being given after six months under sea conditions. The rate for clearing up on completion was stipulated to be a certain proportion of the first item as a minimum, and with a stated lower limit as it was expected that damage to local roads would be incurred.

The shape of the breakwater in cross section was defined by the maximum steepness of the sloping face permitted. On the river side there was also a minimum slope allowed in order to ensure that a long gentle slope was not left which might be a danger to navigation. The core material was allowed to be of any size provided that boulders were not able to be carried out into the navigation channel. The top of the breakwater was to be choked with finer material to provide a surface able to be negotiated by light vehicles.

With these provisions, the actual method of construction was entirely up to the contractor. He was required to state the method he proposed to use and to conform to the loading restrictions on the Buller River bridge. The method chosen by the contractor was to cart the rock by special vehicle, and to place it by heavy crawler-mounted crane.

10. INFORMATION PROVIDED FOR TENDERERS

As much information as possible was collected in t folder to be issued with the formal tender documents 9ut not forming part of the documents. Included in his was a geological report on the granite deposits; a eries of charts showing river flow, wind, set of sea, tnd wave height for one complete year; and a set of tll the reports on the works undertaken between 1929 and 1931, month by month, giving a somewhat ;loomy picture if anything of the difficulties which night have to be faced. It was left to tenderers to ize up all the factors affecting the work.

11. PROGRESS OF WORK AND EFFECT ON BAR

The contract was let on 26 May 1965, with a hree-year period for completion. The breakwaters hemselves were complete by September 1967. Reasontbly good weather had been experienced, but the good Progress was mainly due to the good organisation and nethods used.

The graph of bar depths shows clearly that no mprovement was effected until the breakwaters became coterminous. Indeed, extension of the already advanced western breakwater caused a serious decrease n depths for a short period, the exit flows leaking away o the eastward. It is early yet to say just what pernanent improvement will result because for a time here will be a build-up of sand on the western side. 61 close watch is being kept on beaches near Westport,)ut there has been no sign of any denudation eastvards—oddly enough there was slight erosion to the vest, but nothing significant (Fig. 8).

The extensions to the Westport breakwaters were carried out by the Ministry of Works for the Marine 3epartment. The contractor was Wilkins and Davies _onstruction Co. Ltd.

12. ACKNOWLEDGMENTS

Acknowledgment is made of permission given by he former Commissioner of Works, F. R. Askin, to)resent this paper and to include extracts from de)artmental reports including that of the then engineern-chief, C. W. O. Turner, in 1953, recording the investigation work.

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Partial prestressing - a commentary

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The philosophy of design for prestressed concrete structures is generally that of eliminating flexural tensile cracking. This concept is tempting from a theoretical viewpoint, but cracking owing to secondary effects (e.g. in composite beams, at beam junctions and around anchorages) is in reality unavoidable. In partial prestressing, flexural cracking is permitted but is controlled by the use of auxiliary reinforcement. It is, in essence, a combination of reinforced and prestressed concrete.

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Between 1967 and 1968 he was employed by the British consultants Robert Mathew, Johnson-Marshall as a resident engineer on the structurally-complex new embassy for the Czechoslovakian Government. In 1969 he rejoined Beca, Carter, Hollings and Ferner. Major works conceived and designed recently are the Pakuranga precast cantilever constructed bridge for the Auckland Regional Authority and the Terrace gully motorway for the Ministry of Works (Wellington).

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1. INTRODUCTION

PARTIAL prestressing eliminates some of the disadvantages of full prestressing, namely:

(1) A section with a high neutral axis (such as a T beam) will be penalised in comparison with the rectangular section, for it will be subject to tensile stresses at a much lower percentage of the full live load. The designer is led to form expensive shapes (box girders) by this "no tension" requirement.

(2) Many code design loadings are presented as those that may occur infrequently (e.g. desi,gn truck loadings on wide bridges). Because the full prestress concept overlooks the ability of prestressed concrete beams to be little affected by occasional overloading, it is unnecessarily conservative to provide a high level of prestress for rare loading conditions if the ultimate load criteria have already been satisfied.

(3) Post-tensioned prestressed concrete will generally require substantial amounts of mild steel reinforcement and may have an excessive load factor. Should flexural cracking be permitted under working loads there will be a reduction in the prestress force with these benefits:

   (i) The reduction of undesirable vertical and differential vertical creep deflections.
   (ii) Avoidance of heavy concentrations of anchorages sometimes at undesirable locations.
   (iii) A reduction in the energy absorption characteristics produced by heavy prestressing which may produce a brittle member. (This may be of particular importance in the resistance of earthquake-induced loads.
   (iv) A reduction in longitudinal creep deflections which will simplify bearing or column design.
   (v) Of lesser imortance is the possible reduction in the time for curing in view of the lower creep deflections and prestresses.

(4) Partial prestressing has a further advantage which may be of considerable importance to designers of continuous bridge structures. In such structures any differential temperatures across the depth of the section may cause substantial tensile stresses. If the section is allowed to crack, the imposed moment will be reduced as the beam deflects towards the simply supported deflected shape (refer to Appendix III).

2. HISTORICAL

The advantages of partial prestressing were first realised by Emperger in 1939 and Abeles in 1940[2]. Freyssinet opposed their theories as late as 1949', when he stated that systems intermediate between prestressed and reinforced concrete offered no interest whatsoever. His main objection was that of unacceptable deflections and his reason was that in many tests he had made on prestressed beams he had observed a rapid increase in deflection following flexural cracking. He considered this to be a valuable property as it gave a "free warning" that the design load had been exceeded. As reported by Abeles[2], Freyssinet subsequently (1952) saw value in allowing large tensions (750 lb/in^2) in members subject to occasional heavy loadings.

Because of Freyssinet's original opposition, partial prestressing was little used until the 1950s, although, owing to the influence of Abeles arid Finsterwalder, small tensile stresses were allowed in some design codes. The first international discussion on partially prestressed concrete took place at the 1962 F.I.P. conference, where it was made apparent that thinking was developing to adopt the concept of partial prestressing, especially in the U.S.S.R. Since then, experimental studies have been initiated in different countries. Thurlimann[s], in his preliminary paper to the 8th Congress of the International Association of Bridge and Structural Engineering in 1968, where partial prestressing was a chosen topic, made particular reference to the work of Baus and Depauw[4] as it represents an international survey of the present status and contains pertinent references. The tests of Brenneisen[s] are also worthy of mention.

In the intervening years partial prestressing has been incorporated into several codes[6] and design recommendations1. Recently the British Standards Institution issued a draft code for comment[s] which includes partial prestress concept.

Some major applications have been a 44 million gallon reservoir[10]; docks and warehouses"; and a 175 ft span bridge'. Relatively minor applications (rail bridges, warehouses, poles and railway sleepers) are described by Abeles[2].

3. DESIGN

The following classifications of prestressed concrete are recommended[8, 9]:

- *Class I. Full prestress:* The complete absence of cracks must be assured under all loading conditions (thus no tensile stress allowed).
- *Class 2. Limited prestress:* Tensile stresses are permitted without cracking (no tension allowed under dead load and part of the live load).
- *Class 3. Partial prestress:* The opening of cracks and the straining of reinforcement are allowed within certain limits.

The classes 1 and 2 may be designed on the basis of an uncracked section, and as such are familiar to designers and will not be considered further. It should be pointed out, however, that class 2 is in reality a partial prestressing case; for should the section be subject to an overload it will crack and thus possess no tensile strength for subsequent load applications.

It is important to understand that these classes are not classifications of quality% their sole object is to cover the possible fields of use and to establish for each one the safety with regard to cracking, bearing in mind the structural circumstances.

The criteria by which the choice is made depend on:

(i) The proposed life of the structure.
(ii) The working conditions.
(iii) The type and duration of loading.
(iv) The aggressiveness of the surrounding atmosphere.
(v) The sensitivity to corrosion and the degree of protection of the steel.

3.1. Limit states of design

As for reinforced concrete, these are:

(i) Excessive deflections.
(ii) Intolerable crack widths.
(iii) Strength (this may be produced by instability of form, concrete failure, tensile or fatigue reinforcement failure).

As a safety margin must be applied to the limit states it is necessary to formulate a design approach that can reliably ascertain the behaviour of the class 3 section.

3.2. Flexural analysis

The safety against flexural failure may be reliably computed from present knowledge.

The deflection and cracking behaviour will, however, vary considerably as it is governed by the size, cover, percentage and bond efficiency of the steel reinforcement, and the size, shape and concrete strength (which will also affect the bond) of the section. As post-tensioned tendons will have poorer bond characteristics than pretensioned tendons or untensioned reinforcement, the effect of this should be considered in deflection and crack width calculations1[2].

Two basic methods of flexural analysis have been evolved:

(i) *On the basis of a homogeneous section:*
Abeles[12] has proposed a design method whereby a hypothetical tensile stress is allowed depending on the environment (i.e. the allowable crack width) and the amount and type of reinforcing (tensioned or untensioned). These allowable tensions have been determined from a series of tests as those that have generally not produced cracks of unacceptable widths. This approach has been adopteds where a simple and speedy design procedure is presented.
This procedure also takes into account the influence of shape. Thus for cross sections such as T-beams, which are subject to sudden excessive increase in deflection on cracking, a lower hypothetical tensile stress will be permitted. This is because, with increasing percentages of reinforce-

ment positioned in the tensile zone and near to the tensile faces, the hypothetical tensile stress also increases.

The allowable British Standard[s] hypothetical tensions for 5,500 lb/in[2] concrete range from 580 lb/in² up to 1,370 lb/in².

A similar approach' allows maximum tensile strains of 2.5 times the maximum elastic strain.

(ii) *On the basis of a cracked section:*

The partially prestressed section may be considered along the classical lines of reinforced concrete.

Generally, the simplest procedure is to take a position of the neutral axis and the maximum concrete compressive stress and to vary these until the internal stresses balance the external loads that have been applied.

Birkenmaier'[4], in his commentary on the Swiss code, states that an increase in stress of up to 1,500 kg/cm² (21,000 lb/in') is allowed both in the untensioned reinforcement and the prestressing tendon. In this case, experimental work has shown that the crack widths will stay small (0.10 to 0.15 mm or 0.004 to 0.006 in.) and will close on removal of the applied load. In this manner the risk of corrosion is eliminated as well as the possibility of fatigue steel failure (good quality prestressing steel will resist dynamic variations in stress of the order of 2,500 kg/cm² (35,000 lb/in2)).

A formula proposed by Base and others" for reinforced concrete is crack width = 3.3 ce, where c is the cover to the reinforcement and e is the strain. Stevens"' reports good agreement with this formula for prestressed concrete.

Thus, for a stress of 21,000 lb/in² the reinforcing will strain 0.007 or a maximum crack width for 2 in. cover—of 3.3 X 2 X 0.7 X 10⁻ =0.0046 in.

The designer should, however, examine carefully each case, for poorly-bonded post-tensioned tendons will throw more load on to the mild steel reinforcing. Furthermore this approach may be strongly affected by the actual amount of prestress required. The designer should thus check the steel stresses for a design case assuming 10% less prestress has been applied than specified.

It is important to remember that these design methods are purely to ensure that there will be no unacceptable crack widths or excessive deflections on application of the design load. The choice of the hypothetical tensile stress is for this purpose alone, i.e. it is the minimum prestress needed to ensure that the maximum crack width does' not exceed a specified value. The crucial calculation is to ensure that the factor of safety against failure is sufficient.

In other words, the actual level of prestress may be dictated by factors other than crack width and deflection.

3.3. Fatigue

The fatigue resistance of prestressed concrete in flexure is satisfactory as long as the decompression load is not exceeded. Should this load be exceeded the concrete will be incapable of mobilising any tensile resistance, for concrete shows no tensile endurance limit. In consequence it is necessary to provide well-bonded prestressed or unprestressed reinforcement close to the tensile face so that cracks will stabilise and excessive deflections will not result, and to remember that the bond characteristics of post-tensioned tendons are poor".

Abeles and others'[s] found that the crack resistance of beams tested over a fatigue loading range of between 20% and 70% of the static failure load was remarkably good and gives confidence in the ability of partially prestressed concrete members to withstand an appreciable number of repetitive overloads.

In another study, Abeles and others'[s] found that millions of repetitions of loading over a range up to 650 to l,000 lb/in[2] (hypothetical) concrete stress at the tensile face of the member will not cause failure. In consequence, they say that should the maximum crack width at the first static loading be limited to 0.004 in. then subsequent fatigue failure at the same load may be avoided, if at the same time the range of the steel stress remains within the limits of the applicable Goodman diagram, remembering that untensioned reinforcement will also be subject to compression in the unloaded condition. Stevens[16] finds similarly that, provided stress in the added steel does not exceed 33,000 lb/in', or 0.55 times the yield stress, the limit state of collapse cracking and fatigue is likely to be satisfied.

The ultimate static load has been shown"[, 18] as being unaffected by millions of cycles, during which hair cracks open and close, as long as the steel is stressed below the endurance limit.

A comprehensive theory for the prediction of the fatigue flexural life, with the failure criterion being steel failure, is given by Price and Edwards24.

In summary, it may be stated that failure of materials in fatigue is unlikely to govern flexural design when code stresses and design procedures are applied, although particular attention should be given to the deflections which will occur under these load repetitions.

The designer may thus consider it prudent to design for no tension for what may truly be considered to be fatigue loading. This may, of course, be considerably lower than the design loading. Leonhardt²s suggests that for bridges carrying vehicular loads of between 60 and 100 tons the appropriate degree of full prestressing would be 50% to 60% of the maximum live load moment for bridges of 100 ft span, and perhaps 40% for a 320-ft span.

A limited amount of testing has been carried out on the fatigue performance of prestressed concrete in shear. Guyon21 discusses a report from Klimes at the 1966 F.I.P. congress, in which it is found that the concrete shear strength was reduced under alternating loading to 36% of its static strength. At the present state of knowledge it may be said that the risk of fatigue shear failure is much reduced if shear stresses are kept below 36% of the static values. The possibility of failure can be eliminated only by the application of orthogonal prestressing to eliminate principal tensions.

3.4. Shear and Torsion

As partjally prestressed members will, in general, be subject to cracking under the maximum design shear loads, an elastic shear analysis is not valid. Further investigation is required. Nevertheless, it does appear[3] that the presence of prestressing will improve the shear and torsion properties of reinforced members. Partially prestressed members should, in general, be treated in a similar manner to reinforced concrete when considering shear and torsion and, presumably, their interaction9.

4. APPLICATION OF PARTIAL PRESTRESSING

A design philosophy which may result in a substantial reduction in prestress will be viewed with caution by supervising authorities. Some hesitancy is justifiable in the case of continuous structures; their design is much more prone to inaccuracy. For example, simple beam theory may underestimate the stresses at bearings, and creep assumptions may be important. Furthermore, the method of construction will have an appreciable influence and may lock unforeseen stresses into the structure.

Provided that continuous structures are analysed rigorously and supervised closely, there would appear to be no objection to the use of the partial prestressing for certain types of loading.

Most likely, partial prestressing will find its greatest application in simply supported structures and, it follows, in the precasting industry.

Whether or not the concept finds wide application, the ability of prestressed concrete to carry repetitive loading, even when such loads are well in excess of the cracking load, should be appreciated by designers and supervisors.

5. CONCLUSION

Partial prestressing is seen as an extension of reinforced concrete use which mobilises fully the strength of high yield steels. A prestress is applied to avoid the large deflections that would otherwise be necessary to stress the steel, yet the level of prestress is not so high that some of the disadvantages inherent in full prestressing are encountered. Pretensioned members may thus have lower transfer stresses thereby, decreasing the problems of creep deflections, and end block design, with no reduction in load-bearing qualities. In some cases lower capacity stressing beds may also be employed.

For post-tensioned members fewer cables will be required, with the much cheaper non-prestressed reinforcement ensuring the structural integrity.

Straightforward design methods for flexural analysis have been developed and largely substantiated by testing. Many structures have been constructed with partially prestressed concrete to date and it is apparent that the savings inherent in this design concept will lead to its wider application.

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APPENDIX I

Consider a 12 in. deep slab with nominal reinforcing in. HY60 at 12 in. centres:

Dead-load moment	210, Kin/ft
Live-load moment	+ 200, —98 Kin/ft
(a) Allowing no tension	
Prestress required	74 K/ft

(b) Allowing a maximum tensile stress of 350 lb/in^2 (ref. 8 allows 810) on the bottom fibre only

Prestress required 49 K/ft

For case (b)

Ultimate moment 765 Kin/ft

Ultimate moment required = 1.1 *DL* 2.5 *LL*

= 730 Kin/ft

Partially prestressed slab requires 34% less prestress.

APPENDIX II

Consider a 30 ft span, 18 in. deep double T beam:

Dead-load moment	320 Kin/ft
Live-load moment	220 Kin/ft

For adequate ultimate moment one needs two in. strands

(a) Allowing a tensile stress of 220 lb/in2

Prestress required 36 *K*

(b) Allowing a tensile stress of 550 lb/in2

Prestress required 18 *K*

The partially prestressed beam will have approximately half of the creep deflection.

APPENDIX III

An internal span of a multispan continuous bridge recently studied was found to have the following stresses induced under a temperature gradient of 25°F. The span of the bridge is 246 ft.

(a) Soffit tensile stresses at midspan

Owing to temperature gradient	620
Owing to other imposed effects	890
Total	1,150 lb/in2

b) Prestress required

To eliminate all tension	4,700 *K*
To leave 370 lb/in^2 tension	3,600 *K*

(c) Reinforcing

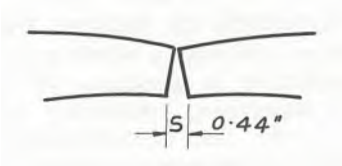
Limiting maximum reinforcing stress to 10,0001b/in2 we need 11 in^2 of reinforcing

The strain in the reinforcing is 10,000/(30x 10^6)=0.33 X 10-3

This straining will take place over the central 80 ft approx.

Total extension= 80 X 12 X 0.33 X 10^{-3} =0.32 in.

(d) Consider now the case of two free cantilevers under differential temperatures of 25°F so that this can be compared with the continuous case under study.



The extension *S=246* x 12 X 25 X 6 X 10^{-6} =0.44 in. where 6x 10^{-6} is the coefficient of expansion.

Thus (i) If the soffit can crack a total of 0.44 in there will be no moment induced stresses.

(ii) Cracking of 0.32 in. is produced by an applied moment equivalent to the moment acting on an uncracked section.

(iii) The applied moment decreases as the beam deflects.

In summary the partially prestress case requires 25% less prestress and will reduce significantly the temeprature gradient induced stresses.

PAPERS AND ARTICLES RECEIVED

THE following have been received by the New Zealand Institution of Engineers:

B. T. Anderson—Quay Street railway overpass.

T. Belshaw—Use of the Dutch deep-sounding penetrometer in New Zealand.

A. L. Titchener—How many engineering students, where should they be taught and what will it cost?

J. B. C. Taylor and D. C. Stevenson—The supply of and demand for professional and technician engineers.

R. McL. Dickie—The Thai-New Zealand road project.

A. T. Watkins—The design and supervision of an industrial structure.

P. J. Moss and A. J. Carr—The use of digital computers in civil engineering education and research.

G. R. Martin—Continuing education: the need for a positive approach.

G. G. Duffey, K. Moller and A. L. Titchener—An experimental programme for the study of the hydrodynamic and rheological properties of New Zealand wood pulp suspension.

J. J. Higgins and B. B. Keey—Simulation of a moisture-control system for wool dryers.

G. L. Bowen and G. K.Underhill—Manifold design.

R. G. Norman—Dinner address to New Zealand national conference on earthquake engineering.

B. W. Potter—Development of technician training beyond New Zealand Certificate level.

Metric units for use in waterworks practice in N.Z.

The Water Supply Committee of the N.Z. Institution of Engineers has considered the units recommended for use in waterworks practice in the United Kingdom, together with those units recommended for use in the civil engineering industries in the United Kingdom and South Africa. The committee is putting forward the following units to the Instiution's metrication sub-committee as its recommendation for metric units for use in waterworks practice in New Zealand. This list should be read in conjunction with NZS 6501 P : 1971 The International System (SI) Units and applies the units in that Standard to the particular requirements of the water industry.

Subject	*Recommended unit*	*Comments on usage*
Length	metre	
	kilometre	
	millimetre	
Area[1]	square metre	
	square kilometre	
	square millimetre	
	hectare	
Volume	cubic metre	For general use
	litre	For use up to one cubic metre
	cubic kilometre	For values in excess of 10^9 m3
Mass	kilogram	
	gram	
	milligram	
	tonne	For masses over one tonne. May be referred to as "metric ton" during change-over period.
Time[2]	second	
	day	
	year	
	minut	For use only in special circumstances (e.g. tank retention periods)
	hour	
Force	newton	
Velocity (linear)	metre per second	
	millimetre per second	
(angular)	radian per second	
	revolution per minute	
Flow (volumetric)[3]	cubic metre per second	For flows in excess of 0.1 cumec
	litre per second	For flows between 0.11/s and 1,0001/s
	millilitre	For flows up to 1,000 ml/s
Pressure4	metres head	For water pressure
	newton per square metre	
	kilonewton per square metre	General units for other than water pressure
	meganewton per square metre	
Temperature	degree celsius	
Work, energy, heat	joule	
	kilojoule	
Power	watt	
	kilowatt	

The following recommendations are made for the application of the above units for the expression of quantities in water engineering and allied disciplines:

Subject	*Recommended unit or derived unit*
Precipitation, runoff, evaporation, infiltration	millimetre
River flow	cubic metre per second
Flow in pipes, conduits, open channels, etc.	cubic metre per second
	litre per second
	millilitre per second
River flow per unit area of catchment	litre per second per square kilometre
Discharges, abstractions, yields5	cubic metre per second
Use of water resources	cubic metre per day
	cubic metre per year
Per capita consumption of water°	litre per person per day
Density	kilogram per cubic metre
	gram per litre
Concentration	gram per cubic metre
	milligram per litre
Calorific value of gas	joule per litre
Gas flow rate	gram per second
Hydraulic load per unit area (as in filtration rates)	cubic metre per square metre per day
Hydraulic load per unit volume (as in lagoons, bio-filters)	cubic metre per cubic metre per day

Biochemical oxygen demand (b.o.d.) loading	kilogram per cubic metre per day
Air supply	cubic metre or litre of free air per second
Pipes and boreholes:	
Diameter	millimetre
Length and depth	metre
Metered supplies	cubic metre. The day used to express the volume flowing during an accounting period.
Aquifiers:	
Hydraulic permeability	cubic metre per day per square metre
Transmissibility	cubic metre per day per metre
Drawdown	
Radius of influence	metre
Depth	
Specific capacity	litres per second per metre
Detention in tanks	hour
Rainfall intensity	millimetre per hour

It is recognised that not all subjects in the wide field of waterworks practice have been covered. However, it is considered that the main uses and applications of units in waterworks engineering and allied disciplines have been dealt with. Where units for other subjects are required, particularly where these do not apply solely to waterworks practice, other recognised sources should be referred to, e.g. the sector committees of the Metric Advisory Board.

It should be noted that the units, uses and applications recommended above do not necessarily correspond to the units, uses and applications recommended for other disciplines. It is therefore recommended that only SI units, or SI derived units, be used in inter-disciplinary communications.

NOTES

[1] *Area.* It is recommended that the hectare be retained as a unit of area. Although not an SI unit and not recommended for water resources purposes in the United Kingdom, it will remain in use overseas in both the water engineering and civil engineering fields. It is a convenient unit to replace the acre.

= *Time.* The basic SI unit of time is the second. It is recommended that the day and the year be retained as important natural units. The minute and the hour recommended for use only in special circumstances.

Flow. The recommended ranges of application of units are considered to be the most practicable for all normal purposes. A proposal to restrict the use of the cubic metre per second for the expression of flows greater than one cubic metre per second is considered to be too restrictive for this SI derived unit.

It is recommended that the second should generally be used as the unit of time in the expression of flows, whether the flow be instantaneous or continuous over a long period of time. Exceptions to this would be in the expression of yields, etc.,[5] and in the expression of flows during an accounting period.

Pressure. Metres head should be used for the expression of water pressure. The newton per square metre and its recognised multiples are likely to become the units used for the expression of other pressures in general civil engineering practice.

The kilogramme force per square centimetre and the bar are likely to be retained internationally for the measurement of gas pressure.

[5] *Discharges, abstractions, yields.* Units such as the thousand (million) cubic metres per day (year) have not been included in the list of recommended units as it is considered that these are logical and natural derivations of the basic units recommended. It is considered that the most practical means of expressing these derivations will evolve naturally with use and should not be subjected to hard and fast rules at this stage.

considered to be a more natural and more easily envisaged expression than alternatives such as litres per day per head.

COURSES AVAILABLE FOR C.E.I. EXAMINATIONS

THE N.Z. Institution of Engineers receives many enquiries concerning the courses that are available in New Zealand for the subjects of the Council of Engineering Institutions' examination. The current situation is:

C.E.I. Part I

The Auckland Technical Institute provides courses for this examination, and correspondence courses are available from the British Institute of Engineering Technology. Other technical institutes are interested in gauging the demand for courses, but none—including the Technical Correspondence Institute—has yet expressed its intention of mounting courses.

C.E.I. Part 2

The only courses available are those recently introduced by the Auckland Technical Institute, covering particular subjects, principally in the civil engineering field. The Institution knows of no correspondence courses that are available for the subjects of the Part 2 examination.

Candidates for the examination must be sponsored by a constituent member of the Council of Engineering Institutions, or by the New Zealand Institution of Engineers. The conditions for N.Z.I.E. candidates for the examination are set out in the Examination Regulations of the Institution and details of the subject syllabuses are given in C.E.I. Statements Nos. 7 (Part 2) and 8 (Part 1). Copies of these documents, and of past examination papers, are available from the secretary of the Institution, P.O. Box 12-241, Wellington.

Enquiries concerning courses of preparation for the examation should be made directly to the teaching institutes concerned.

I.H.V.E. OFFICERS

At the recent annual general meeting of the New Zealand Branch of the Institution of Heating and Ventilating Engineers the following officers were elected for the 1971-72 year:

Chairman, P. R. Robbins, Christchurch; *vice-chairman,* D. W. L. Cooke, Auckland; *hon. secretary,* R. I.. Lefebvre, Christchurch; *hon. treasurer,* F. F. Tredgett, Auckland; *committee,* C. A. Thompson, Auckland, E. Kotlar, Wellington, H. Canard, Christchurch.

SUCCESSFUL PRODUCTIVITY SYMPOSIUM

The Process Control, Productivity and Profit Symposium, held in Wellington at Victoria University in May, was well attended and very successful.

One consequence is the Government's intention to consider the establishment of a Productivity Institute or Council, and the papers presented at the symposium may be taken as the foundation for further productivity studies. Copies of the symposium proceedings are available from the organising bodies, the Wellington Division of the N.Z. Institute of Management and the Automatic Control and Instrumentation Society.

New ferry terminal at Picton

OVER the last 16 months travellers through Picton have been aware of a number of engineering works in progress at the port, where the Marlborough Harbour Board are building a second terminal for the Cook Strait ferry service.

The new ferries of 6,400 gross tons now on order will be considerably larger than the *Aramoana* and *Aranui,* having four rail tracks instead of three and a greater range of draught. The third ferry will also load heavy road vehicles on to its upper deck.

New hydraulically-operated double-span linkspans are being provided at the new terminal and also at the existing terminal, which is being upgraded for the new vessels. The new terminal will also include a third span at the level of the ship's upper deck, connecting a fly-over across the approach road and railways yards.

The principal works to date have been reclamation on the foreshore for improved passenger facilities, and the filling of the old Waitohi lagoon for new marshalling yards for rail and road traffic. Increased areas are now also available for harbour board activities, which include a substantial log export trade.

In order to gain maximum advantage from the reclamation work, the board decided to divert the stream which drains the watershed behind Picton, through a diversion culvert 40 ft by 7 ft high by 1,000 ft long, into the harbour, clear of the ferry terminal area. This diversion, completed last December, was immediately followed by foreshore reclamation which provides access to the new berth. This work was tested by an exceptional rainstorm within a few days of completion and was found to be successful.

In recent months interest has shifted to construction of the linkspan foundations and the wharf for the new berth. All thee works will be supported on internally-driven steel-shell piles with reinforced-concrete leading sections. These piles have proved very suitable for works at Picton and on the present project are driven both from floating gear and on land.

The new berth is also to be equipped for the present ferries which require a small linkspan for side loading of motor can on to the top deck, and twin passenger gangways. New designs have been prepared for these, in both cases with hydraulic operation.

The linkspans and their hoist towers are being manufactured in Nelson, from where the girders and towers will be transported complete to the site.

The project includes a new supply of electricity and distribution system for the port area and a temporary supply to existing facilities while the change-over is effected.

Design work for the project began over 18 months ago and construction was planned for completion at the original arrifial date of the third ferry, 1 October 1971. This represented an extremely tight programme for design and construction, which has been relieved in recent months by successive delays in the delivery programme for the ship.

Engineering work for the whole project for the Marlborough Harbour Board and for reclamation and services for N.Z. Railways is being carried out by Ian Macallan & Co., Wellington, who also carried out a long-term planning investigation for the port. This investigation, providing for all foreseeable expansion of road-rail ferry and overseas trade, was made primarily to ensure that the location of the second ferry terminal would provide for optimum future development of the port area, including suitable sites for future ferry terminals and overseas berths.

Controlling Wanganui River pollution

A $6.5 million works programme to solve the acute pollution problem in the Wanganui River is now under consideration by the Wanganui City Council.

At present 76 outfalls discharge the city's raw wastes directly into the river, polluting both the river and nearby Castlecliff beach.

When the construction outlined in the proposed 10-year programme is completed it will be possible to meet the requirements of the existing receiving waters classifications.

The programme provides for the construction of interceptor sewers ranging in size from 13½ in. to 61 in. in diameter. In general, these will be located along each bank of the river and will connect with the 76 sewer outfalls. There will be three small pumping stations and two large ones incorporated in the system.

The two interceptors will collect all waste from the existing combined sewerage-stormwater system, allowing for up to 2.7 times the existing average dry-weather flow.

However, because stormwater is also taken through the system, controlled overflow into the river through 30 special control structures has been incorporated to cope with exceptionally high rainfall. Nevertheless the river will still comply with pollution classification (Class SC) limits.

The interceptors will carry the waste to a treatment plant located in sand-hills near Wanganui airport, where it will receive primary treatment.

The treated waste will be discharged into the sea from an outfall 6,000 ft offshore. The treatment plant will remove about 50% of the settleable wastes, together with all floating fat and grease, before it is discharged. There will also be some 25% reduction in the concentration of coliforms before discharge.

The recommended scheme of drainage improvements has been designed by Worley, Downey, Muir & Associates, who also carried out the investigation programme, including all of the laboratory testing and analyses.

Public Service Division news

Internal protective coatings *for hydro power station penstocks*

THE N.Z. Electricity Department has 25 hydro power stations at which a total length of over 21 miles of internally-coated conduit is installed. The total surface area of the conduit is over 2,250,000 ft^2 and the diameters range from 1 ft 6 in. to 23 ft. The water that passes through the penstocks originates from three main sources:

(i) Snow-fed mountain streams,

(ii) Bush-clad high country, and

(iii) The large Waikato Valley catchment of the central North Island.

The water quality varies from the almost pure waters of the southern rivers and lakes to the more aggressive waters of the Waikato Valley.

The Ministry of Works, as the construction agents, have been active in the development and application of coatings in new work. Throughout the years a number of paint systems have been applied, in line with current practices of the day. The oldest and most widely used system was a thick coating of hot-applied coal-tar enamel. This was sometimes applied over a red-lead undercoat. A number of penstocks were coated with cold-applied coal-tar enamel.

After World War II a system was developed using a vinyl butyral wash-coat, an undercoat of zinc chromate and topcoats of aluminium-pigmented phenolic varnish reinforced with sand. During this period some penstocks were also painted with zinc-rich paint and overcoated with aluminium-pigmented phenolic resin.

Further developments in the late 1950s resulted in a vinyl paint system, originally with a wash primer and latterly bonded direct to the steel. This system is now generally used on all new penstock work. At Manapouri, however, the Department's consultants specified a coal-tar epoxy paint system for underwater steelwork.

Nearly all the paint systems have been applied to well-prepared blasted surfaces, the earlier ones being sand-blasted and the later ones shotblasted to a white metal finish.

When applying the coal-tar enamels both the surface and paint were heated prior to painting. In the early stages of paint system development all the coats were brushed on to ensure adequate "wetting" of successive layers. Further development led to brushing of only the first coat, with additional coats spray-applied. Today's materials allow the spraying of the complete multicoat system.

The performance of most of the coatings has been very good, with many paint systems lasting for over 20 years. The life of a coating system, however, is difficult to gauge as there is normally a long time between the application of the paint and the final assessment of its performance. During this time improvements take place in the paint systems themselves, and new paints are developed. For evaluation purposes a number of experimental sections have been painted in some of the penstocks.

When the paint system affords little protection to the steel, recoating is necessary. To monitor the coating's performance and to determine when recoating is needed, regular inspections are required. The present policy is a thorough inspection of the penstocks every three or four years, with a less rigorous inspection every year. These are carried out during the summer overhaul period. Factors affecting the thoroughness of the inspection are: The length of time the penstock has been dewatered; dryness of the surface; the extent of surface deposit; and the steepness and diameter of the penstock.

A number of requirements have to be borne in mind when considering a paint system to be used for repainting. The most important are:

(i) The paint must have a long life under water-immersion conditions,

(ii) The time available for painting is limited, and

(iii) The conditions inside the penstock cannot be completely controlled.

Coal-tar enamels are now not used for repainting mainly because the material is difficult to handle and has irritating effects on the workman. Vinyls are not favoured because of the difficulty of achieving the strict conditions under which the paint must be applied. They also have the disadvantage of producing copious fumes and have a low film-build per coat (although high-build vinyls are now becoming available). However, vinyls have an advantage in that any damaged areas may be repaired quite easily.

Penstock at Maraetai II during erection, showing a vinyl interior.

The paint system at present considered by the N.Z. Electricity Department to be most suitable for repainting is a coal-tar epoxy. It has extremely good water resistance, can be applied in a thick film, is tough and adherent, and can be applied under less than ideal conditions. However, it should not be applied below a temperature of 10°C nor at high humidities. This paint has been used extensively in penstocks overseas. The paint used by the Department for recoating work is a polyamide-cured coal-tar epoxy. It is applied by airless spray to give a total minimum dry-film thickness of 0.016in. in either two or three coats. The surface is dry blasted to either a white metal or near white metal finish depending on the extent of rust pits in the steel. The finer grades of abrasives are preferred. The paint is applied directly to this surface without a primer. Because the paint is thixotropic it will hang up in thick coats and cover the sharp edges of pits and projections. The penstocks are well ventilated by means of blowers and, where necessary, the air is heated. Once a coating is fully cured, however, it is difficult to repair any damaged areas.

The work is done either with N.Z.E.D. staff and equipment or by contractor.

Depending on factors such as accessibility, steepness and diameter, the cost of repainting a penstock, including the surface preparation, is approximately \$1.00/ft^2.

A properly planned programme of repainting will involve an average area of 80,000 ft^2 per year. With modern techniques and materials it is expected that there will be a minimum of 20 years between repaints and considerably longer in the more favourable locations.

PERSONAL

B. W. Spooner

B. W. SPOONER, B.E., C.Eng., F.I.C.E., F.N.Z.I.E., chief civil engineer for the Ministry of Works, retired recently after 40 years with the Public Service.

In 1934 he joined the Public Works Department and worked on the Waikaremoana hydro-electric schemes and on the East Coast main-trunk railway. In 1947 he was appointed senior engineer in the power design office in Wellington working initially on the large civil works involved in the establishment of the substations for the 220 kV distribution system and later he was in charge of sections of the design of hydro schemes on the Waitaki and Waikato rivers.

In 1955 he was appointed chief designing engineer and during his term of office the designs of the Christchurch-Lyttleton tunnel, the Mt. Maunganui wharf and a number of notable bridges including the Kawarau, the Mouth of Haast and the Newmarket Viaduct were completed.

In 1960 he attended the conference of the International Association of Bridge and Structural Engineering in Stockholm and visited the United Kingdom and the United States. In 1964, as assistant chief civil engineer, Mr Spooner attended an international government meeting on seismology and earthquake engineering in Paris.

In 1966 he was appointed chief civil engineer for the Ministry of Works. During his term of office he was involved in the decision to purchase the Jarva tunnelling machine for the Kaimai tunnel, he was responsible for the contracts for 400 miles of pipeline and for the construction of the treatment plant for the Kapuni Gas project. The airport works at New Plymouth and Rotorua and the present extensions to airports at Nandi and Rarotonga were also his responsibility.

Mr Spooner served 15 years as Minister of Works' representative on the Water Pollution and Control Council and has been deputy chairman since 1955. He also represented the M.O.W. on the N.Z. Standards Council and on the Local Authority Affairs Committee of the Board of Health.

In 1968 Mr Spooner became president of the N.Z. Institution of Engineers after serving many years at branch level and on the Council. As president he attended the inaugural meeting of the International Conference of Engineering Organisations at Paris and the conference of the Australian Institution of Engineers in Brisbane.

A. G. STIRRAT, C.Eng., F.I.C.E., F.N.Z.I.E., M.I.Struct.E., chief designing engineer (civil) of the Ministry of Works, has been appointed chief civil engineer on the retirement of B. W. Spooner.

Since he joined the Ministry of Works in 1941 he has held many posts.

In 1969 he travelled overseas to study design problems associated with large prestressed concrete bridges and gathered material of value in the construction of the Newmarket Viaduct. When he was acting as civil engineering advisor to the New Zealand Post Office information he gathered in Japan was of great value in the construction of the Warkworth satellite earth station.

A. G. Stirrat has been active in the N.Z. Institution of Engineers. He was a member of the Institution Council for six years, retiring as an executive vice president in 1969. He is interested in engineering education and is moderator of the Institution's professional interview panel.

A. D. Martin

A. D. MARTIN, C.Eng., F.I.C.E., has been appointed city engineer designate, Wellington; he will assume the office of city engineer on the retirement of J. S. ROBERTS, C.Eng., F.I.C.E., F.N.Z.I.E., F.I.Struct.E., A.M.I.T.E., next year.

In 1936 Mr Martin joined the Wellington City Corporation and, except for a period of service with the 2nd N.Z.E.F., has been with the city engineer's department since then. In 1948 he was appointed district engineer, and was promoted to divisional engineer in 1963. He was appointed deputy city engineer in 1964.

A. J. WATT, B.E.(Civil), M.Sc.(Eng.) London), D.I.C., M.N.Z.I.E., has reurned to New Zealand and joined Beca Carter Hollings and Ferner in Auckland al
ground engineering division.

He began his career in 1949 as an assistant engineer at the Roxburgh hydro scheme. Between 1951 and 1957 he was with the Army, stationed in New Zeaand, Germany, England and Malaya. During this period he had two years at Imperial College, London.

In 1958 he joined the Cementation Company Ltd., England, first as manager of the soils mechanics section, then of the geotechnical department and then
gineering division. From 1968 to 1970 he was a director of Cementation Ground Engineering Ltd.

M. J. WALKER, B.E., M.N.Z.I.E., assistant engineer to the Ashburton Elecric Power and Gas Board since 1967, has been appointed chief engineer replacing G. S. MII.I.AR, B.E., M.N.Z.I.E., who has accepted a position n Australia.

J. H. B. FISHER, B.E., C.Eng., M.I.C.E., M.N.Z.I.E., M.I.Q., who was previously marketing manager, N.Z. Cement Holdings Ltd., has been appoined assistant general manager (Operaions).

P. ARMSTRONG, B.E., M.N.Z.I.E., has been appointed general manager of McConnell Dowell Constructors Ltd. He was formerly operations manager of McConnell Dowell Ltd.

The appointment comes as a result of extension and remodelling of the company. Construction activities have been consolidated into McConnell Dowell Constructors Ltd., and steelwork and mechanical activities have been grouped under System Engineering Ltd. Both companies are wholly owned subsidiaries of the McConnell Dowell group.

R. H. L. WEBB, B.E., M.N.Z.I.E., has recently been admitted to partnership by R. Garden and Partners and will be resident in the Invercargill office.

Mr Webb has been in the practice for about two years including a period as resident engineer on the construction of he Bluff smelter wharf. Prior to this he was employed by Downer and Company Ltd. in various capacities, including management of the Invercargill area, management of a joint venture constructing extensions for Wilsons Cement Co. Ltd. at Portland, and engineering positions on the construction of the Aviemore diversion tunnel and Momona airport.

BUILDING SERVICES GROUP

BUILDING-SERVICES EDUCATION IN NEW ZEALAND

THE following is the text of a report prepared by the Building Services Group education sub-committee, and forwarded to the Institution.

Introduction

The value of building-services work in New Zealand per year is approximately $50 million.

Building-services design requires a very wide background, basically mechcanical, but extending into electrical, civil and structural engineering and architecture. Development in the field is very rapid and there is a major shortage of fundamental research and of local specialists.

Technical training

Technical training is virtually nonexistent and hence it is the major req uirem en t. "Air-conditioning" and "heating" have not been a success as supplementary subject to the N.Z. Certificate in Engineering as most students wish to specialise before their final year rather than spend an extra year studying. Unless this can be resolved, another system of education must be found. "Sandwich" courses may be useful and the Central Institute of Technology could be in a position to provide these and are willing to do so. However, nothing will happen without a demand from students, employers and the engineering institutions.

Degree training

Auckland University provides a mechanical engineering course that could readily be adapted to building-services requirements, provided the final-year course includes:

(i) Heat engineering
(ii) Control engineering
(iii) Industrial engineering
(iv) A fourth subject to complete the four (optional units) not taught at present but which could include acoustics and noise control, fire protection, aesthetics, physiology (as related to environment), and biology (as related to cold storage).

The drawing and design course would have to include considerable time on building-services systems and components.

Canterbury University had a course which was suitable, except perhaps in the subjects of noise control, fire protection and acoustics, but, because of a shortage of lecturers, the course is no longer offered.

Any course must include appropriate practical work, both in class and during vacations.

Several members of the mechanical engineering faculties have interests in fields which are to some extent applicable to building services. They would welcome suggestions for projects of interest to the industry. It is strongly recommended that building-services engineers maintain active contact with the universities and look to them for the solution of problems that require reasonably long investigations. If the interest is there the universities appear willing to supply the expertise, measuring equipment and staff to investigate problems.

The major problem with courses is the shortage of suitable lecturers. Advertisements for lecturers in England have not been successful; there could be more success in advertising in Australia or America.

Recommendations

(i) That members take advantage of the offers from both Auckland and Canterbury Universities to do research work and to investigate problems experienced in the building services.
(ii) That every opportunity be taken to publicise the need for education in building services and that the suggested syllabus changes be implemented and attempts be made to obtain lecturers.
(iii) That a notice be prepared and circulated, giving details of education available and suggesting courses in the universities and technical colleges which might be appropriate to the needs of the building services.

AN ENGINEER'S BOOKSHELF

CRACK PROBLEMS IN THE CLASSICAL THEORY OF ELASTICTY by I. N. Sneddon and M. Lowengrub; 221 pp. (Wiley, New York, 1969, $U.S.14.95).

An account of calculations in the mathematical theory of elasticity relating to "Griffith" cracks and their three-dimensional analogues. Griffith's work (1921) is classical and is the origin of the theory of brittle fracture. Interest in the theory has recently revived as a result of the experimental discovery that at high or low temperatures materials that display plastic properties in standard tensile tests fail by a "quasi-brittle" process in that failure is by crack propagation and that there is only a small plastic zone concentrated at the crack tip.

There is a brief discussion of the physical considerations and the basic theory, but the remainder of the book is mathematical in approach dealing first with two-dimensional crack problems followed by extensions to three-dimensional problems. A very extensive bibliography gives papers relevant to the mathematical theory rather than the physical theory.

This book may well be a worthwhile text for mathematicians and engineering theorists involved in such problems as brittle fracture but it is not a necessary addition to a pragmatic engineer's library.

—J.C.S.

SKELETAL STRUCTURES by C. M. Bommer and D. A. Symonds; 106 pp., illus. (Lockwood, London, 1968, £1.25p).

A clear explicit treatment is given of matrix methods of the elastic solution of plane frames. The general method of treatment had its inspiration from the D.I.C. lectures of J. C. de C. Henderson at Imperial College of Science and Technology, London University. It is at least the third text on frame analysis inspired from the same source and the other two texts are referenced by the authors. As one who was also impressd by Henderson's lectures at Imperial College, the reviewer considers that the book is a faithful representation of the Henderson-influence coefficient method.

Nearly the first third of the book is devoted to matrix algebra. The structural analysis which follows emphasises the flexibility method with a shorter section on the stiffness method. Since it is generally recognised, and certainly by the authors, that the stiffness method is simpler to program and therefore preferable to use with an electronic computer, this emphasis is a strange one.

The authors have designed the text to educate the designer in the ways of analysis which are usually adopted in computer programs. They believe that with an understanding of the analytical processes concerned, designers would become less reluctant to use computers. Simple problems have been used to illustrate he essence of the subject. even though the full dvantages of the influence coefficient matrix method are completely utilised only when the technique is applied to complex problems. Problems dealt with include the effects of mperature, lack of fit and settlement of upports.

The shortness of the text has inevitably led o the exclusion of some aspects of the probma. For instance, only a passing reference made to the possibility and consequences of nalysis using an ill-conditioned matrix. Paricularly with the flexiblity method, this may well govern the type of and positioning of re-l nses to make the indeterminate frame determinate. Other items have also of necessity been treated with slightly less rigour than necessary for students. However, this is a conise treatment on frame analysis which may well provide a designer with a quick means of ppreciating the analysis techniques now used n the computer.

—J.C.S.

ENGINEERNG OF DYNAMIC SYSTEMS by William R. Perkins and Jose B. Cruz Jr.; 568 pp., illus. (Wiley, New York, 1969, U.S.13.95).

The systems approach to engineering has almost as many meanings as advocates. This book could have been on process control, or omputer science, or management science, or

INDUSTRIES DIVISION

RESULTS are appearing month by month from the policy laid down by the management committee early this year for meetings to be convened in local centres. To run every meeting by remote control from the central committee would be unworkable, but management committee members have been elected from Auckland, Waikato, Wellington and Christchurch and they provide the link with Industrial Division sub-committees which are now active in all these areas.

The Division is determined to see its main activities develop at local level, where all members can participate several times a year, and not just run from one annual conference to the next. Where possible, meetings are held in co-operation with the N.Z.I.E. local branch committee, but some of these are convened separately. Amongst forthcoming attractions on the I.D. front are:

AUCKLAND

15 September, "An approach to an e.d.p. application" (G. Dewar, parts division manager, British Leyland Motor Corp. N.Z., and T. Rudman, e.d.p. manager, British I.eyland Motor Corp. N.Z.).

This promises to be an unusual presentation of an e.d.p. application. Both speakers have had wide experience of electronic data processing and instead of dealing with the application of specific hardware or software they describe how they commenced with a specific problem and worked back from there. The points made during this address interest anyone affected by e.d.p.

20 October, "Marketing—or why engineers have jobs" (D. G. Buckleton, marketing and management services manager, Alex Harvey Industries).

As with all sections of the community, engineers are employed only because there is a demand for what they offer. To many engineers this demand emanates from a largely unknown territory whose nature is frequently misunderstood. D. G. Buckleton is an engineer (B.E., B.Sc.) who has made his career in the study of this territory and the forces which operate within it. This talk promises to be both relevant to many and refreshingly different.

WAIKATO-BAY OF PLENTY

19 October, "Training—by accident or by design?" (H. J. Harvey, staff training manager, New Zealand Forest Products).

No engineer would undertake an engineering project without formal and careful planning, organisation and control, yet all too often he neglects to give the same attention to the training and development of his staff. In these circumstances the experience provided will vary greatly but at best will consist of a loose and informal system of job rotation. If desirable experience or knowledge is gained it is more by accident than by design.

By introducing planning, organisation and control into training the level of knowledge and experience to be obtained is predetermined and all efforts are directed towards reaching the objectives set. In general the programme will include induction in policies, procedures and practices of the organisation; documented programmes of experience to achieve defined objectives; and regular discussion and consulting sessions with experienced engineers. Although the concept of the programme may apply generally to a specific group, say recent engineering graduates, each training plan must meet the needs of the individual.

The time taken to train and develop staff in this way will be amply repaid by the more efficient and effective work force produced.

Further details of this meeting, which is to be held at Paeroa, will be given in the Waikato-Bay of Plenty branch newsletter.

WELLINGTON

22 September, "Case study on human relations—the administrator" (Professor Graeme Fogelberg, Department of Business Administration, Victoria University).

Following the success of the first Harvard Business School style of case study on 9 June, this second exercise is being arranged for the benefit of Wellington members. They have been advised of details through an I.D. regional newsletter to the Wellington area.

CHRISTCHURCH

A 3-day seminar was held at the beginning of this month at the University of Canterbury on "Qualitative management techniques for manufacturing and process industries".

Divisional membership is building up steadily now, and interest is growing in outlying areas such as Whangarei and Masterton.

Those eligible for I.D. membership, interested in such meetings but perhaps with no local branch, should apply to the honorary secretary (P. D. Preston, 31 Copeland Street, Lower Hutt, or care of N.Z.I.E., P.O. Box 12-241, Wellington).

It takes only a few to justify calling a meeting.

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many other popular areas, but it is a text on the now familiar aspects of control theory applied to electro-mechanical systems.

Among the distinctive characteristics of the systems approach, regardless of the particular field of application, are a study of the responses of macroscale sub-systems; mathematical modelling of systems, including nonlinear functions and discrete variables; and optimisation studies. These are each discussed in this book, which ends before introducing probabilistic or stochastic variables.

It is composed of three main parts: modelling and simulation system analysis, and control and optimisation. A "theme example" associated with the attitude control of a space craft integrates the topics studied. The treatment is mathematical and, as a prerequisite, a good grounding in the theory of linear differential equations with time invariant coefficients, matrix and vector algebra, and the calculus of finite differences seems essential. A fundamental understanding of the dynamics of partieles and rigid bodies and of electrical circuit theory is also essential. Additionally, elementary experience with the use of digital and analogue computers and an elementary knowledge of control theory is desirable. This probably makes it suitable as a text for the final year of a B.E. course or for an M.E. course.

—H.McC.

ESSENTIALS FOR THE TECHNICAL WRITER by Hardy Hoover; 216 pp. (Wiley, New York, 1970, $U.S.6.95).

Dr Hoover is publication supervisor of the Hydraulic Research and Manufacturing Company and an instructor of extension courses of the University of California, Berkeley. The publishers state that he is an outstanding technical writer and one of the most acclaimed teachers in the field, and that the book, developed from the author's lectures, shows aspiring technical writers how to write tight, concise and clear technical and scientfic copy.

This reviewer finds these claims to be accurate; he has read the book with great interest and profit. It is an excellent textbook logically laid out and comprehensive in scope. It could be used by a class or by individuals studying alone. Each chapter includes test exercises with the answers fully explained. The publishers' claim that the reader is "well drilled through actual exercises" is an accurate statement. The book may indeed become an outstanding publication; it can certainly be thoroughly recommended.

—A.W.

LINEAR ELASTIC ANALYSIS by David G. Elms; 224 pp., illus. (Batsford, London, 1970, £3.50p).

The author, a senior lecturer at the University of Canterbury, has set out to convey an understanding of basic principles essential in the analysis of elastic structures rather than specific techniques, and to give a "feel" for structural behaviour. Through his concise presentation, the well-selected examples with which lie demonstrates different approaches to the same situation and the numerous illustrations, he fulfils his aims most successfully. The diagrams, generously provided throughout the text, express performance vividly and thus serve the purpose of developing the highly desirable feel for structural behaviour.

After a brief review of fundamentals, the understanding of which is a prerequisite for the reader, the force methods and displacement methods of analysis, as applied to frames, are presented in some detail. Methods based on the principles of virtual work, virtual forces and strain energy are briefly covered and related to each other. The formulation of analysis in terms of stiffness and flexibility matrices is repeatedly shown.

Apart from its role as a university text in structural engineering, this book can be strongly recommended to practising engineers. It will assist them greatly in identifying and classifying in Dr Elms' synthesis the everyday techniques of the design office, and in becoming familiar with the reformulation of these, in forms more amenable to computer processing.

—T.P.H.

MISCELLANEOUS ADVERTISEMENTS

ELECTRICAL ENGINEER

Applications are invited for the position of Assistant Substation Engineer, Takapuna, Auckland. Duties will include the design, construction and maintenance of 33 kV/11 kV substations. Conditions of appointment may be obtained from the undersigned. Closing date for applications 1 October 1971.

R. W. Hubbard,
Chief Engineer,
WAITEMATA ELECTRIC POWER BOARD,
Private Bag, Takapuna North.

The Thames Valley Electric Power Board invites applications for the following position:

AREA DISTRIBUTION ENGINEER

The successful appointee will be required to control and administer the Board's Morrinsville-Te Aroha district. The work will include the construction and maintenance of subtransmission and distribution systems, and the control of associated staff, located at two main depots within the area.

Applicants should possess a N.Z. Certificate in Engineering (heavy current, electrical, option) and have registration as an Engineering Associate, preferably together with previous suitable experience.

The position is an interesting and challenging one, in charge of a progressive and expanding district, having a maximum demand of approx. 15 MW and an area of approximately 400 square miles. The appointee will be one of a team of similar area engineers controlling various parts of the Board's district, with overall administration from Te Aroha. Housing available.

Conditions of appointment and further details on application. Applications close in Friday, 24 September.

O. C. STEPHENS
Chief Engineer
P.O. Box 123
Te Aroha
THAMES VALLEY

BULLER ELECTRIC POWER BOARD

ASSISTANT ENGINEER

Applications, addressed to the Engineer and closing at noon on 11 October 1971, are invited for the position of Assistant Electrical Engineer. Applicants must have a B.E. Degree, Member or Graduate Member of the Institute of Electrical Engineers. Experience in electrical supply authority work would be an advantage.

W. H. BROWN,
Engineer.

STRUCTURAL ENGINEER

We require a graduate engineer with 3-4 years' experience; or a registered engineer with 2-3 years' experience, to work on several large building projects now at concept stage, either in our Wellington or Palmerston North offices.

Salary in accordance with ability and experience.

Applications, stating qualifications and experience, should be made to:

Mr D. G. Irvine,
GABITES, TOOMATH, BEARD, WILSON & PARTNERS,
Architects, Town Planning Consultants, Civil Engineers, I.andscape Architects,
P.O. Box 1586,
PALMERSTON NORTH.

HOROWHENUA COUNTY COUNCIL

STAFF ENGINEER OR GRADUATE ENGINEER

Applications are invited for the above position, which offers a wide variety of Dxperience in survey, design and construction of both rural and urban works. Duties would include roading, bridging, water supply, sewerage and structural design.

Position would suit a recently Registered Engineer or a Graduate Engineer.

Salary range
Registered Engineer $5,200-$5,700
Graduate Engineer $4,300-$5,000

Commencing salary is dependent upon experience and qualifications.

Housing available at reasonable rental.

Conditions of appointment available from the County Engineer, P.O. Box 258, Levin.

CIVIL ENGINEER

We require the services of an Engineer capable of administering post-tensioning contracts. Duties would involve tender ing, organisation of site work, preparation of prestressing details and supervision of the design office. Previous experience in prestressed concrete would be preferable, but not essential. Our work covers an interesting section of Civil Engineering involving bridging, reservoirs, buildings and foundations.

The successful applicant shall hold a University Degree or equivalent and shall be experienced in the design or construction of concrete structures.

This position offers a salary of up to $8,000 depending on experience, an annual bonus, staff superannuation scheme and other staff privileges.

Apply in confidence to:

The Managing Director,
BBR NEW ZEALAND LIMITED,
P.O. Box 473,
MASTERTON.

PARTNERSHIP NOTICE

GABITES, TOOMATH, BEARD, WILSON & PARTNERS,
Architects, Town Planning Consultants, Landscape Architects,
Wellington,
and
DONALD G. IRVINE,
Registered Civil and Structural Engineer,
Palmerston North,
announce that they will amalgamate their practice as from 1 August 1971 under the name of
GABITES, TOOMATH, BEARD, WILSON & PARTNERS,
Architects, Town Planning Consultants, Engineers, Landscape Architects,
and will continue in the meatime to operate from their present offices.

61 Abel Smith St., and 59 Aurora Terrace, P.O. Box 5136, Wellington.
134 Broadway, P.O. Box 1586, Palmerston North.

A. L. Gabites
W. Toomath
J. A. Beard
D. J. Wilson
D. J. Edmondson
W. H. Aling:on
G. D. Anderson

Donald G. Irvine, C.Eng., M.I.C.E., M.N.Z.I.E.

MASSEY UNIVERSITY

LECTURER IN PROCESS ENGINEERING

in the Department of Biotechnology

Biotechnology is process technology Applied to materials of biological origin. Fhe field is closely allied to chemical and biochemical engineering and is of particular significance in New Zealand.

The duties include undergraduate and graduate teaching in process engineering, assistance with practical courses and participation in research. Qualifications ;ought are: A degree in chemical engineering, biochemical engineering or :hemical technology; a higher degree and industrial and research experience would be an advantage.

Salary within the range $4,814-$6,034.

Conditions of appointment and details concerning the University may be obtained from the undersigned, with whom applications close on 12 November 1971.

A. J. WEIR,
Registrar.

SITUATION WANTED

ELECTRICAL/ MECHANICAL ENGINEER

Senior American professional engineer (B.A., B.S.E.E., Columbia University) with diversified background in many facets of design and production desires to relocate in New Zealand. Copy of resume may be obtained *New Zealand Engineering* or directly from:

Henry G. Benis,
Hinesburg,
Vermont 05461, U.S.A.