1963

VICTORIA

REPORT

OF

ROYAL COMMISSION

INTO THE

FAILURE OF KINGS BRIDGE

PRESENTED TO BOTH HOUSES OF PARLIAMENT BY HIS EXCELLENCY'S COMMAND

By Authority:
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No. 1.—6352/63

ROYAL COMMISSION OF ENQUIRY INTO THE FAILURE OF KINGS BRIDGE

Report of the Royal Commission to His Excellency, Major-General Sir Rohan Delacombe, Knight of the British Empire, Companion of the Bath, Companion of the Distinguished Service Order, Governor of the State of Victoria and its dependencies in the Commonwealth of Australia.

YOUR EXCELLENCY,

In pursuance and execution of Letters Patent dated the Twenty-eighth day of August, 1962 under the Seal of the State of Victoria, whereby His Excellency, the Honourable Lieutenant-General Sir Edmund Francis Herring, K.C.M.G., K.B.E., D.S.O., M.C., E.D., Lieutenant-Governor of the State of Victoria issued to us :—

His Honour Edward Hamilton Esler Barber, a Judge of the County Court ;

Dr. James Adam Louis Matheson, M.B.E., M.Sc., Ph.D., M.I.E.Aust., Vice-Chancellor of the Monash University ;

and

Professor John Neill Greenwood, D.Sc., M.Met.E., Dean of the Faculty of Applied Science of the University of Melbourne ;

a Commission authorizing and appointing us to enquire into and report to Your Excellency upon the following matters, namely, the cause or causes of the failure of the bridge known as Kings Bridge, constructed pursuant to the *King-street Bridge Act* 1957 and in addition and without derogating from the generality of the foregoing, the following matters :—

(I.) The terms, conditions, specifications and drawings in accordance with which tenders for design and construction of the bridge were invited by the Country Roads Board, and whether the same were adequate and reasonable for the purpose ;

(II.) The tenders received, the action taken to investigate the same, the circumstances surrounding the acceptance of the tender submitted by Utah Australia Limited, and whether the acceptance thereof was reasonable and proper and justified in the circumstances ;

(III.) The design submitted and adopted for the bridge, and whether the same was adequate and suitable or was in any and what respects defective or inappropriate or deficient ;

(IV.) The materials and processes and workmanship used in the construction and erection of the bridge, the standard and suitability thereof for the purposes for which they were used, whether they were in accordance with the contract specifications and whether they were in any and what respects defective or inadequate ;

(V.) The nature, extent and standard of supervision exercised over the construction and erection of the bridge, and whether the same was reasonable and adequate or was in any and what respects inadequate or defective ;

(VI.) Whether any and what negligent, culpable or improper act or omission directly or indirectly caused or contributed to the failure of the bridge, and if so the party or parties responsible therefor ;

(VII.) Whether the construction and erection of the bridge in accordance with the tender submitted by Utah Australia Limited was reasonable having regard to the known state of engineering and scientific knowledge and experience subsisting at the time the tender was accepted ;

and His Excellency directed and appointed that His Honour, Judge Edward Hamilton Esler Barber should be Chairman of the said Commission.

We, the undersigned, Chairman and Members of the Commission having duly enquired into the several matters aforesaid, now have the honour to report to Your Excellency as follows :—

1. PUBLICATION OF NOTIFICATION OF SITTINGS OF THE COMMISSION.

Pursuant to our direction Notices of the date of Sitting of the Commission were published in the " Herald " newspaper on the 1st day of September and in the " Age " and " Sun " newspapers on the 2nd day of September, 1962.

2. SITTINGS OF THE COMMISSION.

The Commission held a Preliminary Hearing on the 6th September, 1962 and thereafter between the 1st October, 1962 and the 28th March, 1963 sittings upon 71 days. The Commission heard evidence *viva voce* of forty-five witnesses, a list of whom is contained in Appendix 1 of this Report. Two hundred and thirty-seven exhibits were received in evidence (Appendix 2).

The persons who gave evidence before the Commission did so on Oath and were subject to examination and cross-examination by Counsel. The evidence given was reported verbatim and embodied in a transcript of evidence which is respectfully presented with this Report.

3. REPRESENTATION OF PARTIES BY COUNSEL.

Mr. S. T. FROST, Q.C. with Mr. G. JUST appeared to assist the Commission.

The following Counsel were granted leave to appear :—

Mr. J. STARKE, Q.C., with Mr. S. E. K. HULME and Mr. S. P. CHARLES, instructed by Malleson Stewart and Co., on behalf of Utah Australia Ltd.

Mr. B. L. MURRAY, Q.C., with Mr. X. CONNOR, Q.C., and Mr. J. WINNEKE, instructed by the Crown Solicitor, on behalf of the Country Roads Board.

Mr. O. J. GILLARD, Q.C. and Mr. D. DAWSON, instructed by Weigall and Crowther, on behalf of King-street Bridge Design Ltd. (Mr. Gillard withdrew on the third day of October, 1962).

Mr. N. E. BURBANK, Q.C. with Mr. H. BALL and Mr. G. S. H. BUCKNER, instructed by Russell Kennedy and Cook, on behalf of Johns and Waygood Ltd.

Mr. N. E. BURBANK, Q.C. with Mr. H. BALL, instructed by Russell, Kennedy and Cook, on behalf of Murex Australasia Ltd.

Mr. J. McI. YOUNG, Q.C. with Mr. N. M. STEPHEN, instructed by D. J. Nairn on behalf of Broken Hill Proprietary Company Ltd. and Australian Iron and Steel Pty. Ltd.

Mr. L. VOUMARD, Q.C. with Mr. W. FAZIO, instructed by the Commonwealth Crown Solicitor for the Commonwealth of Australia.

CONTENTS

There are frequent references throughout the Report to clauses of the C.R.B. specification and to sections of the Report itself. To differentiate between the two sets of references the following scheme has been followed :—

References to the C.R.B. specification are in the form :—

Clause x–y–z. e.g. 2–3–16.

References to the Report are in the form :—

Section x.y.z. e.g 2.3.5.

In the course of this Report it has been necessary to mention with great frequency a number of corporate bodies and individuals. In order to save space and avoid tedious repetition we have referred to such corporations using their full name where first mentioned, and thereafter have used initials or short form of the name.

The abbreviations used for corporate bodies are as follows:—

Australian Iron and Steel Pty. Ltd.	A.I.S.
Broken Hill Proprietary Company Ltd.	B.H.P.
Country Roads Board	C.R.B.
Defence Standards Laboratories	D.S.L.
Engineering Testing and Research Services Pty. Ltd. ..	E.T.R.S.
Johns and Waygood Ltd.	J. & W.
King Street Bridge Design Ltd.	K.S.B.D.
Murex (A/asia) Pty. Ltd.	Murex
Utah Australia Ltd.	Utah

The following is a list of individuals who are frequently mentioned throughout this Report. As this list sets out the initials and other relevant information, we have in general used surnames only in the body of the Report.

Bonwick, J. E.	B.C.E., Office Engineer for Utah on the Project 1957–1958; Project Engineer for Utah on Kings Bridge Project 1958–1960.
Butler, L. T.	B.A. (Eng.) (Oxon.), B.Sc. (Eng.), C.E., M.I.E. Aust., C.R.B. Supervising Engineer for King-street Bridge throughout the Project.
Campbell, R.	Welding inspector employed by J. & W., concerned with supervision and inspection of welding on Project.
Clarke, N. V.	Resident Inspector for C.R.B. at J. & W.'s shop on Project from early 1959.
Darwin, D. V.	M.M., M.C.E., M.I.E. Aust., Chairman of C.R.B., 1949–1962.
Eastick, R. F.	M.B.E., C.E., A.M.I.E. Aust., Senior Constructional Engineer (Bridges) of C.R.B. throughout the Project.
Farrar, W. C.	B.C.E., A.M.I.E. Aust., an Executive Director and the Manager of the Structural Department of J. & W. throughout the Project.
Ferris, I. J.	B.Met.E., A.R.A.C.I. Principal Scientific Officer, Defence Standards Laboratories
Fink, G. W.	Project Manager for Utah on the Project from August, 1957, until February, 1961.
Francis, Professor A. J. ..	M.Sc. (Birm.), Ph.D. (Birm.), M.I.C.E., M.I.Struct.E., Head of the Department of Civil Engineering, University of Melbourne.
Hardcastle, B. T. A. ..	B.C.E., A.M.I.E. Aust., Managing Director of K.S.B.D. since 1957.
Hudson, R. F.	B.Met.E., Manager and Metallurgist for E.T.R.S. since 1957.
Hyland, Sir Herbert ..	Member of Legislative Assembly, Victoria and the Leader State Parliamentary Country Party.
Jackson, F.	A.M.I.E. Aust., C.R.B. Assistant Engineer on the Project.
Longo, L. N.	A Utah executive in charge of the preparation of tenders for the Project.
Masterton, C. A.	M.C.E., A.M.I.E. Aust. During the Project was in turn C.R.B. Assistant Engineer for Bridges and Engineer for Bridges.
Mathieson, J.	M.C.E., M.I.E. Aust., C.R.B. Chief Engineer throughout the Project.
Miller, V.	A Utah civil engineer engaged on design aspects of the Project.
Ralston, O. B.	Service Officer of B.H.P. attached to its Melbourne office throughout the Project.
Reedy, L.	Chief Engineer of Caterpillar of Australasia Pty. Ltd. 1959–1963.
Roderick, Professor J. W. ..	M.A., Ph.D., M.I.C.E., M.I.E. Aust., F.A.A., Head of the Department of Civil Engineering, University of Sydney.
Scarlett, K. F. A.	An executive of J. & W. engaged on the Project, 1958–1960.
Stocker, W. R.	B.Sc. (Engineering) (Glasgow), M.I.M.E. London, Assistant to the Managing Director of J. & W. since 1949.
Thompson, J. W.	Executive Officer, Administration, at Port Kembla Steelworks of A.I.S.
Ward, F. A.	Now Assistant Australasian Sales Manager of Murex. Was in turn the Senior Technical Representative and the Assistant Branch Manager of Murex during the Project.
Wilson, C. A.	B.C.E., A.M.I.E. Aust., Senior Design Engineer in the Bridge Division of C.R.B. since 1950.

PART 1.—INTRODUCTION.

1.1. Introductory Narrative.

1.1.1. Brief history of the project.

In November, 1954, the Parliamentary Public Works Committee recommended the construction of a fixed bridge to cross the Yarra River at King-street and referred the proposal to the Country Roads Board for investigation and report.

In 1955, the C.R.B. recommended that the crossing should form part of a route with freeway conditions for traffic bound for the centre of the City of Melbourne and originating south of the Hanna-street–City-road intersection in South Melbourne. In April, 1956, the Public Works Committee adopted the Board's proposal and made a further recommendation for a structure generally on the lines of that ultimately erected. The C.R.B. was recommended as the constructing authority with power to call tenders on a world-wide basis.

During 1955 and 1956 the C.R.B. surveyed the site, investigated the foundation conditions and prepared an outline scheme for the project which was to comprise two low-level crossings of the Yarra River; a high-level crossing continuing as an elevated roadway for approximately one-third of a mile to the south; and an overpass to carry Flinders-street traffic over King-street close to the northern ends of the river crossings.

The C.R.B. decided to invite tenders for the design and the construction of the structures and, to this end, prepared an outline drawing showing the grades, clearances and other limiting conditions within which the contractor was required to work. A specification was prepared which laid down in some detail the design requirements (loading, permissible working stresses, &c.) and also the standards required in materials and workmanship. In particular the contractor was permitted to offer a superstructure in reinforced concrete, pre-stressed concrete, mild steel, high-tensile steel or light alloy.

In September, 1956, tenders closing on the 29th January, 1957 were invited in Australia, United Kingdom and U.S.A. Tenders were received from seven tenderers who between them submitted fourteen different designs. After a full investigation of these tenders, the C.R.B. decided to recommend acceptance of the tender from Utah Australia Ltd. for the sum of £2,374,360 16s., and on the 27th May, 1957, formal advice to this effect was sent to the Minister—and was accepted by Cabinet. On 13th August, 1957, the contract between Utah and the C.R.B. was executed.

Work on the bridge commenced on the 19th September, 1957. On the 18th December, 1957, the King-street Bridge Act was passed—appointing the C.R.B. as constructing authority, conferring the necessary powers on the Board to construct the bridge for the Government of Victoria, and validating such steps as had already been taken by the Board.

The work of construction was completed in various stages, the high-level bridge and elevated roadway being opened to traffic on 12th April, 1961.

Pursuant to the King-street Bridge Act, certificates of completion were published in the *Government Gazette* as to each stage, the final certificate being published on the 18th October, 1961. Upon publication, each part of the structure became a metropolitan bridge or highway, and the Melbourne and Metropolitan Board of Works became the maintenance authority for the bridge and roadway.

1.1.2. The parties.

The design which formed the basis of Utah's tender had been prepared by King-street Bridge Design Limited, a company formed by a number of engineers specifically for the purpose of creating a design for this bridge. Utah Australia Limited having signed the contract with the C.R.B. for the construction of the bridge, engaged a sub-contractor—Johns & Waygood Limited for the fabrication, erection and painting of the steel work for the superstructure. J. & W. in turn ordered the necessary steel from Broken Hill Proprietary Company Limited. The electrodes used for welding were supplied by Murex (Australasia) Proprietary Limited.

1.1.3. Description of the bridge structure.

As authorized by the King-street Bridge Act, and as finally erected, the whole Kings Bridge structure comprises the Flinders-street overpass, the low-level bridges and the high-level bridge and roadway mentioned above. The latter crosses the Port Melbourne Railway and the St. Kilda Railway, Whiteman-street, Queen's Bridge-street and City-road and runs into Hanna-street near Grant-street, South Melbourne.

The matters concerned in this Enquiry, however, relate to the high-level bridge roadway only, and the expression " the bridge " as used hereafter may be taken to refer only to this portion of the above structure.

The bridge consists of two parallel structures forming eastern and western carriage-ways. Each structure is constructed with four lines of main girders, supported by piers, and with a reinforced concrete deck. In most spans the design was for four suspended girders carried by cantilever girders, extending from supporting piers. Every girder consisted of two flange plates, welded to a web plate and strengthened at intervals by vertical steel stiffeners. It was necessary for the designers to overcome the problem arising from the fact that the bending moments sustained by the suspended girders are at a maximum at mid-span. One possible solution would have been to have used tension (lower) flanges varying in thickness so that the thickest portion was in the centre of the girder's length. The alternative design, which was actually chosen, was to attach a cover plate on to the bottom of the lower flange of the suspended girder terminating some 16 feet from each end of the girder. The cover plates were tapered at each end from their maximum width to a square end detail 3 inches wide. The cover plates were completely welded to the flange by fillet welds running the entire length of each side of the cover plates, continuing along the side of the taper and terminating in a transverse weld across the 3-in. end of the cover plate.

All steel in the structure relevant to the present Enquiry had been fabricated by J. & W. from a high-tensile steel supplied by B.H.P. to B.S.968 : 1941. It is to be noted, however, that the C.R.B. specifications required high-tensile steel, if used, to satisfy some of the clauses of B.S.968 : 1941 and certain additional clauses notably one relating to the impact strength of the steel.

1.2. Collapse of the Bridge on the 10th July, 1962.

On the 10th July, 1962, shortly after 11 a.m. a low loader and trailer weighing unloaded about 17 tons—carrying a load of approximately 28 tons, drove on to the western carriage-way of the bridge from the South Melbourne side. When it reached a span, W.14, close to the southern end of the western-half section of the bridge, that span suddenly collapsed—sagging a distance of approximately 1 foot, being kept from further subsidence by the concrete deck and the vertical concrete wall slabs which enclosed the space under the bridge. The girders which collapsed were designated W.14–1, 2, 3 and 4. Subsequent examination disclosed that all four of the suspended girders at this point, which were approximately 100 feet in length, had fractures approximately 16 feet from the southern end. Three of them had fractures at a similar position 16 feet from the northern end.

The bridge was closed to traffic and very shortly after the 10th July, a Committee of Investigation was set up to enquire into and report upon the causes of failure and measures to be taken for the repair of the bridge. A copy of the portions of this Committee's report relevant to this Enquiry appears in Appendix 3.

1.3. Immediate Cause of Collapse.

Upon examination by the expert Committee, subsequently confirmed by our own observations, it was found that fractures had occurred at the south end of all four girders W.14–1, 2, 3 and 4 and at the north end of girders W.14–2, 3 and 4—there being no fracture at the north end of girder W.14–1. The fractures were all at points approximately 16 feet from the end of the girders and were all found to start from the welds at the ends of the cover plate. In every case fracture had initiated at the toe of the transverse weld at the end detail of the cover plate and extended completely through the lower flange up the web, and in some cases through the upper flange.

All fractures were typical of failure by "brittle fracture" originating from toe cracks in the parent metal of the flanges at the transverse fillet welds. As will appear from the later detailed examination of these fractures, they all exhibited signs which established beyond doubt that the ultimate dramatic collapse was but the last stage in a "cascade" pattern of fractures—which had taken many months to reach the point where final collapse occurred.

The evidence establishes that the ends of these cover plates were all manually welded as the last operation in the fabrication of the girder, after the flanges had been welded to the web and the cover plate to the lower flange.

The complex process which led to the collapse of the W.14 girders is divisible into four distinct stages :—

1. In at least seven of the eight transverse welds at the end of the cover plates, toe cracks occurred immediately after welding the end detail—each crack starting in the heat affected zone of the second weld bead and continuing into the parent metal of the lower flanges. These cracks, therefore, were running transverse to the length of the flange, at a position of stress concentration associated with the sudden change of thickness.
2. Very soon after welding—and before painting—some of the toe cracks extended by brittle fracture through the lower flange and in some cases into the web plate. The positions so affected were W.14–1(S), W.14–2(N), W.14–2(S), W.14–3(S) and W.14–4(S).
3. At some time after the final coat of paint had been applied in January, 1961 two of the primary brittle fractures propagated up the web plates. In girder W.14–2 at the south end the fracture continued 44 inches up the web and completely severed the bottom flange at this point. This girder, therefore, carried no part of the dead load or traffic load.
4. By the time the low loader was driven across the span on 10th July, the cracks in girders 2 and 3 were so extensive as to render these girders incapable of bearing any of the load of traffic and consequently the passage of the heavy vehicle caused a final failure in all seven fracture sites through the entire web plate and bottom flange plates in each position and through the upper flange plates of W.14–1, 2 and 3 at the South end.

It is considered that stages 1 and 2 were the critical stages—and that once these had eventuated, stages 3 and 4 were inevitable. Although a good deal heavier than the average user of the bridge, the low loader was well within the load limits which the bridge was designed to take without danger.

When the low loader arrived two of the four girders were virtually useless, and the other two seriously weakened—sooner or later a load of sufficient magnitude to fracture the span must have been imposed. The collapse could well have occurred at a time of peak traffic and have resulted in a serious injury and loss of life.

When a bridge fails, one immediately thinks of the possibility of subsidence or other failure of the foundations. It should, however, be stated at once that no scintilla of evidence came before us which would raise any doubt as to the soundness of the foundations or that the difficult problems confronting the contractor as to the foundations were quite satisfactorily solved.

Perhaps the main relevance of the existence of difficult conditions for sound foundations, is that we are left with a suspicion that a pre-occupation with these problems led to some lack of concentration on other problems associated with the superstructure.

Although the immediate cause of the failure of the W.14 spans was fairly readily ascertainable the reasons for the girders having reached such a dangerous condition, and without discovery by inspection, were found to be highly complex and such as to necessitate an investigation going back to the genesis of the Kings Bridge project.

It was found necessary not only to examine in some detail, such matters as the relevant sections of the bridge design, the quality of materials and the standard of workmanship, supervision and inspection, but in order to fix the relative responsibilities of the various parties engaged in the construction of the bridge, enquiry had to be made into a number of issues arising out of the relationships of the personnel representing the constructing authority, the contractors and the several sub-contractors involved.

PART 2.—THE CONSTRUCTION.

2.1. Nature of the Contract.

Before inviting tenders on a world-wide basis, officers of the C.R.B. gave careful consideration to the most suitable type of contract. They investigated three possible ways of handling the undertaking, which were :—

(*a*) The Board's own staff might have prepared a detailed design and bill of quantities and called for tenders thereon.

(*b*) The Board might have employed a firm of consulting engineers to carry out the detailed design for the purpose of calling for tenders.

(*c*) Tenders might have been invited on a " design and construct " basis, with the Board supplying its data of investigation and limiting outline dimensions of the structures, together with specifications both for the design and for construction.

The reasons which influenced the Board in its decision to choose the third method are set out in detail in a paper by Darwin included in a collection of papers entitled " Kings Bridge ", published by the Institution of Engineers Australia, (Exhibit 22), and were further discussed in his evidence.

The first method was rejected because it was said to entail a year longer for preparing plans and specifications, the second because it similarly involved loss of time, and also that there were no local experienced bridge design staffs of the necessary numbers outside the Board's own staff.

This latter objection is a curious one, as the design ultimately accepted was produced by local designers, a number of whom had combined for the purpose.

The contract as finally executed between the Board and Utah was not in the sense in which that description is usually understood, a " design and construct " contract, although it was so described and referred to throughout the Enquiry. Usually the constructing authority sets out the nature and extent of the required structure in wide general terms, and leaves it to the contractor to create a detailed design. Doubtless, because of the desire to have competitive tenders for construction as well as for design the Board found it necessary to prepare quite detailed design and specifications. A " design and construct " contract as usually understood is very suitable in cases where it is desired to employ the special expertise of some particular contractor, as for example where the contractor has some patented method of construction unobtainable elsewhere. On the other hand, when used in conjunction with competitive tendering, the true " design and construct " form is inappropriate, and must be modified, as in this case, to the extent where its advantages may be lost.

Darwin advanced four main arguments in support of the form of contract adopted—

1. That this system attracted the ideas of different designers.
2. That " with the contractor basically responsible for his own design there should be especially good harmony in supervision ".
3. That a " similar procedure had been very successfully followed in the invitation of tenders for Sydney Harbour Bridge ".
4. That if an overseas firm secured the contract some of the design work would be done overseas thus " affording relief to hard-pressed local offices ".

None of these reasons, when examined, is really compelling. The first is true only up to a point. As a result of the Board's plans and specifications being drawn in considerable detail, the field in which the tenderers were able to exercise their ideas of design was comparatively limited. To give one example, from many others, the specifications were so drawn as to limit the thickness of flanges in welded constructions to 1 inch, a factor which proved restricting to designers. The second argument has a theoretical attractiveness, but in practice the hoped-for harmony soon gave way to discord.

As to the third argument, while the builders of the Sydney Harbour Bridge achieved a notably satisfactory result, this may well have been rather in spite of than because of the form of contract adopted, which was the subject of serious criticism by eminent members of the engineering profession (See Minutes of Proceedings of the Institution of Civil Engineers, Volume 238, 1935).

The fourth argument again exhibits a pre-occupation with the time element and the supposed inadequacy of local design staff.

In practice the form of contract adopted proved to be most unsatisfactory.

The most serious disadvantage was undoubtedly the absence of a design engineer responsible not only for matters of design in the first place but for general supervision and control of the various aspects of the work in progress. Had a consulting engineer been employed directly by the C.R.B. in the first instance to design the bridge and act in a supervisory capacity he would have been responsible directly to the C.R.B. and would have exercised supervisory functions for it. This is in marked contrast to the position under the system adopted, as the designers were responsible only to the main contractor and not to the Board. Moreover, they could not be called upon to supervise the sub-contractors for the Board. The contract between Utah and K.S.B.D. specifically provided that the designers would be available for consultation but would not be required to undertake any job supervision. (Exhibit 39, Cl. 4 (*e*)). In the result there was a complete separation of the functions of design and supervision.

This situation is well illustrated by the evidence of Hardcastle, one of the designers, to the effect that had he become aware that J. & W. was encountering a great deal of " cracking " during welding, he would have reconsidered the design, with a view to eliminating any features which might have given rise to undesirable stresses. Under the system adopted, the designers had no right to such information and no corresponding duty to undertake any revision of design.

It is very difficult to determine how much of the breakdown in organization occurring during the construction is traceable to the form of the contract and how much to the attitude of the various parties to it.

There existed a very noticeable gap between the constructing authority and the sub-contractors. This gap might have been filled to some degree by the main contractor, Utah, had the responsible officers of that company seen fit to undertake some active participation in the work of supervision and inspection, or had they felt it desirable to have arranged conferences between the various parties including K.S.B.D. to resolve difficulties as necessity arose, from time to time. It is fair to say that all of the parties relied on their legal contractual rights and so meticulously avoided stepping outside their proper contractual sphere, that a grave lack of liaison and co-operation between them resulted. To a considerable extent, this may be blamed on the parties themselves, but we feel that the disastrous effect of it, would have been at least mitigated had the C.R.B. used a form of contract which provided for a consulting engineer responsible for over-all supervision and control.

It is true that witnesses from all parties when asked for their views on the suitability or otherwise of the form of contract adopted, were in accord that it was a good arrangement, which worked well. However, an over-all consideration of the evidence compels us to an opposite conclusion.

It is our considered opinion that the C.R.B., while doubtless acting with the best intentions, made what turned out to be a crucial error of judgment in deciding upon the form of contract, which shaped the pattern of contractual relationships between the parties and failed to provide the necessary over-all supervision. These factors contributed to the troubles and difficulties encountered during construction and may have had a direct bearing on the failure of the bridge.

2.2. The Tenders.

2.2.1. The tenders received.

On the due date for the lodging of tenders, the 29th of January, 1957 the C.R.B. received fourteen tenders and one offer (not a legal tender) from seven different companies.

It was quite apparent that the Board and the Government expected that from the wide area in which the invitations to tender were published, there would be available a very large number of tenderers, some of whom would be corporations of world reputation in bridge building. Doubtless, there was disappointment that no tenderer which could fairly be classed as of top-rank bridge-building reputation saw fit to seek the contract for this important structure. Nevertheless, the actual number of tenderers was reasonable, and amongst them were a number of companies of solid reputation and wide experience, including Utah, whose reputation in general construction work stood very high.

The specifications (Clause 1–1–10) stated that the Board's officers in assessing these various tenders, would take into consideration the following factors :—

(*a*) Total cost ;

(*b*) Time to complete contract ;

(*c*) Adequacy of design ;

(*d*) Appearance ;

(*e*) Ease and cost of maintenance ;

(*f*) The ability of tenderer to finance the contract.

Tenderers were required to submit particulars showing their competence, personnel to be engaged upon the project, and a time schedule for various stages of design and construction.

In a letter to the Premier dated 12th March, 1957 (Ex. 10) Sir Herbert Hyland suggested that " two good outside consultants should be employed to sit in with C.R.B. officers when they were considering the tenders ". This suggestion was rejected, on the Board's advice that such assistance was unnecessary, and the matter was pursued no further. At this date it is impossible to say what effect, if any, the assistance of such consultants would have had, or even if there were available locally at the time any suitable persons not already associated with one or other of the tenderers. The Board's reluctance to accept such outside assistance is understandable. All that can be said now is that the presence of such independent advisors might have changed the picture, particularly in relation to the use of high-tensile steel. In any event, this might well have afforded some protection to the C.R.B. against subsequent criticism. No positive finding is possible, and we do not feel disposed to make any comment adverse to the C.R.B. on this matter.

The system adopted for the selection of the successful tenderer was designed to ensure a proper examination of all tenders. Wilson described the process as follows :—

He and Masterton were handed—not the actual tenders—but the preliminary drawings and calculations and were unaware of the names of the tenderers or the amounts. The two officers then checked the drawings and calculations for any obvious contraventions of the specifications or major errors in the design. At a later stage, the amounts of the tenders were given to the examining officers and a full report made to the Chief Engineer and the Board. The tender received from Utah (£2,374,360) was the third lowest valid tender received. In fact, the actual amount paid to Utah including extras was £2,770,794 16s. Always leaving aside the question of the use of high-tensile steel we are satisfied that there was no factor of superiority in any of the higher tenders, by comparison with that of Utah, which should have required the C.R.B. to reject the latter in favour of any of the higher tenders, or justified it in doing so.

The lowest tender, which was submitted by John Holland and Co. Pty. Ltd. in the sum of £1,793,658 was subjected to a critical examination. In his memorandum of the 27th May, 1957 (Ex. 11) Darwin set out the reasons for rejection which were as follows :—

(*a*) The foundations were not designed in accordance with the specification. This matter is discussed in Section 2.5.2.

(*b*) The appearance was said to be unsatisfactory.

(*c*) The financial strength of the company was subject to some criticism, mainly because it sought certain concessions as to method and time of progress payments and reduction in the amount of retention money which were thought to indicate " apprehension on the part of the company itself as to its ability to successfully finance a project of this magnitude ".

(*d*) The tenderer's experience was mainly in the field of industrial building and the staff and the engineers proposed to be employed was considered inadequate. The amount of £45,000 allowed for design was low as compared with the amount allowed by other tenderers.

(*e*) The tender figure was thought to be too low and indicated that the company would be unable to carry out the project at this figure.

(*f*) Interference with traffic would have occurred over a longer period than was the case with the proposals of other tenderers.

(*g*) There was said to be considerable risk of general public dissatisfaction due to delay or even stoppages occasioned by deficiencies of this tenderer.

During the hearing a good deal of evidence was directed to show that the steel flanges in this design were of greater thickness than the specified 1 inch and that, therefore, the tender was also outside the specification in this regard. (See Section 2.5.2.) Curiously, this was not one of the grounds relied upon in 1957 for rejecting this tender.

The next tender, that of E. G. Clementson (Vic.) Pty. Ltd. (£2,118,037) was rejected for very similar reasons :—

(*a*) Financial structure of the company was not regarded as impressive.

(*b*) The disabilities regarding piles mentioned in relation to the John Holland tender applied equally to this tender.

(*c*) A nominated sub-contractor had attached special conditions as to payments which were in conflict with the specifications and unsatisfactory to the Board.

In this case also it was said that taking the piles down to the mud-stone would involve so much extra cost as to bring the total price above that of Utah.

2.2.2. The tender accepted.

The Utah tender by contrast was considered to be free from any of the objections raised to the lower tenders and to display a number of satisfactory features which may be summarized as follows :—

The design was adequate, particularly as to the foundations which consist of 5-ft. diameter " Benoto " cylinders carried well down into mud-stone at a level which is an average 33 feet below the level provided by John Holland.

The time schedules were satisfactory, providing a minimum interference with traffic.

The appearance was pleasing and the design of supporting piers gives good traffic clearance and allowed useable space under the structure.

The maintenance aspect was satisfactory.

The tender price was realistic and reasonable.

The previous history of the company in Victoria and elsewhere was impressive and its financial position sound. It was associated with a number of eminent Victorian consulting engineers. (In passing it is observed that the idea of some of the design work being done overseas with consequent " relief of the hard-pressed local offices " seems to have disappeared from sight).

There were two outstanding features of the Utah tender which required earnest consideration by the C.R.B. officers. Firstly, the use in the superstructure of high-tensile steel, a material that was new and untried for welded bridge construction in Australia. This matter is fully discussed in other sections of this Report, from which it will be found that we were compelled to the conclusion that the C.R.B. should not have accepted high-tensile steel for use in the bridge, unless and until it had become satisfied, by the clearest demonstration, of the competence and ability of the tenderer, and its steelmaker and sub-contractor, to supply and fabricate the steel to a satisfactory standard. From this it follows that the Utah tender, in which this material was proposed for the whole of the

superstructure, should not have been accepted without far more enquiry and investigation on this basis. It is quite plain that insufficient critical attention was given to this matter at the stage of considering tenders. None of those involved appear to have had any realization of the extent to which they were venturing into unknown territory in using this material.

Secondly, concerning the design of the sub-structure ; the C.R.B. officers were very properly concerned to ensure absolutely sound foundations, which the Utah design was thought to provide, and in the result, appears to have achieved. However, as we point out in Section 2.5.2, the foundations in the Utah tender involved a much greater cost than those of John Holland.

It is true that the foundations designed by John Holland were outside the specification, and it is evident that C.R.B. officers doubted their adequacy for the purpose and rightly regarded the safety of the foundations as of paramount importance. Nevertheless, we feel that this aspect of the John Holland tender perhaps merited a more thorough investigation than it received.

The whole of the evidence, including the contemporary documents, suggests to us that from the beginning the C.R.B. officers were very greatly impressed by the reputation of Utah, which they regarded as a highly-efficient organization likely to make a success of the bridge project. While this may have been a perfectly proper attitude, we are left with the impression that it perhaps induced some zeal to find reasons in favour of accepting the Utah tender.

The Board having made its choice of tender, the recommendation (Ex. 11) was sent to the Minister.

The letter of advice to the Minister made no reference to what we have referred to as one of the two outstanding features of the Utah tender, namely the fact that high-tensile steel was to be used for the first time in Australia in a welded bridge. The Board's then Engineer for Bridges, Masterton, had informed representatives of Utah at a meeting before the acceptance of the tender that he doubted their ability to satisfactorily weld steel of Australian origin, but was told that B.H.P. had given an assurance that the steel was weldable. At the time the specifications were prepared he had no doubt that high-tensile steel complying with the Board's specifications and supplied from the United Kingdom could be welded, provided satisfactory welding procedures were used. The Premier expected that the Board's report upon the tenders and its recommendations would deal with all aspects of the project in a most satisfactory manner. We feel that the Board should have advised the Minister that the Utah tender involved the use of high-tensile steel of Australian origin for the first time in a welded bridge and that its own Engineer for Bridges entertained doubts about the use of such steel.

At this point the Minister directed that the tender documents and other material should be examined by Mr. V. G. Swanson, at that time Chief Engineer, Ports and Harbours in the Public Works Department, so that he could " appraise " the tenders. Swanson reported to the Minister by a memorandum dated 20th June, 1957 (Ex. 12). It is a little difficult to understand precisely the object of this appraisal. Swanson's own evidence suggests that he was mainly concerned to ensure that the tender accepted was within the specifications. He did not advert in a critical way to such matters as the use of high-tensile steel. Indeed, he disclaimed any competence to advise on this matter. However, on the aspect of the business propriety of accepting the Utah tender, the Board is entitled to rely on the Swanson memorandum as constituting contemporaneous independent advice supporting its decision.

Two aspects of the Board's decision to accept the Utah tender must be kept separate.

We criticize the Board for its too ready acceptance of the proposal to use high-tensile steel in the bridge, and for its agreement to the award of the sub-contract for fabrication to J. & W. without ensuring its competence to weld this unfamiliar steel. In these matters of judgment we feel the Board was in error. On the other hand, setting these matters aside, we are satisfied that the decision to accept the Utah tender was honestly made.

Early in this Enquiry we became aware that rumours were current, the nature of which led us to expect that some evidence would be forthcoming suggesting or alleging impropriety in relation to the tenders received and to the Board's action in dealing with them. No witness prepared to give such evidence came forward, and none was discovered by those assisting the Commission. We have not found any evidence or any suspicious circumstances suggesting impropriety on the part of any C.R.B. officer, tenderer or other person in this regard. Any criticism made by us, on the matter of the assessment of tenders, is criticism of judgment merely and is not intended to suggest impropriety or to impugn the integrity of any of those persons involved.

2.3. Setting Up the Organization.

2.3.1. Arrangements for design.

Soon after the announcement of the decision to build Kings Bridge, the Melbourne Division of the Institution of Engineers, Australia, became interested in the matter. Because the project afforded a unique opportunity to the independent profession in Melbourne, it was decided by the structural sub-panel of the Consulting Engineers Panel of the Institution that a group of consulting engineers be formed to prepare a design, and to interest Victorian contractors in such design.

Messrs. Hardcastle and Richards, a firm of consulting engineers, paid the fee of £100 and obtained the Board's specifications. Subsequently, they were approached by Utah with a view to assisting with the preparation of a tender. Messrs. Hardcastle and Richards informed that company of the existence of the Design Group and members of the Design Group were then consulted. It was finally agreed between Utah and the Design Group that the Design Group would prepare designs and drawings in sufficient detail to enable Utah to submit a tender for the project.

The Design Group was incorporated on the 6th March, 1957 under the provisions of the Companies Act, as "King-street Bridge Design Limited" a company limited by guarantee. Hardcastle was appointed Managing Director. On the 25th March, 1957 a letter was received from Utah setting out the terms and conditions upon which the company was authorized to proceed with the preparation of tender designs, drawings and supplementary specifications for the King-street Bridge. Actually the formal agreement between the companies was not executed until the 19th January, 1959 but was deemed to have commenced on 16th January, 1957. (See Exhibit 39).

It is clear under the contract that the duties of K.S.B.D. were limited to preparation of design and consultation on "Matters arising during construction which involve the Final Drawings". Specifically it was not obliged to carry out job supervision or preparation of schedules or material lists.

The designers prepared designs in both mild steel and high-tensile steel. Before the final decision was made on the choice of these two materials, a conference was held on the 28th November, 1956 between Mr. Farrar of J. & W., Mr. Longo of Utah and Hardcastle. During this conference, Farrar telephoned Mr. Frankenberg, sales manager of B.H.P. in the presence of the other parties and repeated to them his conversation. There can be no doubt that at this point all parties understood that ample supplies of steel to B.S. 968 : 1941 would be available from B.H.P. and further that this steel was weldable if proper welding procedures were adopted and executed with skill. The effect of their understanding is summarized in letters passing between J. & W. and Utah dated 1st December, 1956 and 7th December, 1956. On the basis of this understanding and for economic reasons discussed elsewhere in this Report (Section 2.7.1.1.), the decision was made to proceed with the final design in high-tensile steel.

2.3.2. Utah contract with J. & W.

Unlike the contracts executed between C.R.B. and Utah, and Utah and K.S.B.D., Utah's contract with J. & W. was never reduced into a formal document, but is to be found in a series of letters. (See Ex. 15). By the 4th September, 1957 these letters had resulted in the contract for the fabrication of the steel work. (Tr. 1209).

On the 11th August, 1958 a contract was made for the erection of the steel and for the field paint work.

In its tender to Utah, J. & W. did not include any particular item for the cost of testing material.

2.3.3. Approval of J. & W. as sub-contractor.

By a letter dated 8th November, 1957, Utah, as required by clause 1.2.14 of the specification, submitted to C.R.B. names of several proposed sub-contractors, including J. & W. as fabricator of structural steel and the C.R.B. replied on the 11th November, 1957, accepting these sub-contractors. Subsequently, by letters dated the 12th and 15th May, 1958, J. & W. was submitted and accepted as sub-contractor for the erection of the steel. The high reputation of J. & W. as a fabricator of mild steel was of course well known to the C.R.B. However, the comment is made elsewhere that it was readily accepted as a fabricator in high-tensile steel without adequate enquiry as to its experience of this material or competence to make the necessary adjustment of its fabrication methods. (See 2.7.1.1 and 2.4.9).

2.3.3.1. *Appointment of Scarlett to act on behalf of Utah.*

On the 15th August, 1958 Utah wrote to C.R.B. in the following terms :—

> " Please be advised that Mr. K. A. Scarlett of Johns and Waygood Ltd. is authorized to act on our behalf in matters pertaining to the testing and inspection procedures required for acceptance of structural steel products for the King-street Bridge project.
>
> Such authorization is subject to our prior approval of any matters affecting changes in construction procedures, scheduling or variations to the contract ".

And on the 22nd September, 1958, C.R.B. replied as follows :—

> " We can only agree to dealing direct with Mr. Scarlett instead of with Mr. Fink, if it is understood that any statement made, or opinion expressed by Scarlett, is taken as being made on your behalf ".

Thereafter, apparently, Scarlett acted as Utah's representative in discussions arising in relation to testing and inspection.

2.3.4. J. & W. contract with B.H.P.

2.3.4.1. *Failure to order steel to specification.*

One of the critical events in the whole story of the bridge occurred when on the 20th May, 1958, J. & W. placed its first orders with B.H.P. for the manufacture of steel in accordance with B.S. 968 : 1941. The order (See Ex. 17), omitted any reference to the additional tests prescribed by the C.R.B. specifications. (Clauses 2–3–6 to 2–3–16 and 2–5–1). In the result J. & W. found itself bound to accept steel from B.H.P. without additional tests, and bound under its contract with Utah to supply girders fabricated from steel which complied with these additional tests. The elucidation of the mystery of how this extraordinary situation came about, caused us considerable difficulty, and the evidence of the J. & W. witnesses did nothing to reduce the confusion.

A factor which conditioned the whole of the early negotiations was that nobody in J. & W. comprehended the real significance of these tests. They just did not realize that these tests were specified to ensure the production of something differing in quality from steel manufactured to B.S. 968 simpliciter, as a safeguard against brittle fracture. Farrar's evidence was to the effect that, in the tender to Utah for the fabrication of steel, J. & W. failed to include any allowance for these tests by " an oversight ". No doubt, however, this " oversight " occurred because of the mental attitude of J. & W. arising from previous experience that " the production of B.H.P. certificates usually satisfied the clients ". We interpret this as meaning that tests such as those specified by the C.R.B. were not always insisted upon by the customers who were usually content to accept fabricated mild steel which had been certified by the steelmakers.

Between the time of tendering to Utah, (25th January, 1957), and the time when orders for steel were placed with B.H.P., (20th May, 1958), the J. & W. management had a number of conversations with B.H.P. officers and as a result were convinced of two matters—firstly, that B.H.P. was adamant in its refusal to supply steel with additional tests, and secondly, that B.H.P's attitude to Izod tests was that they were of little or no use.

The evidence of the B.H.P. witnesses, Ralston and Thompson shows that in fact, neither of these two conceptions was completely accurate. If B.H.P. had been pressed with sufficient determination, it is probable that it could have been persuaded to accept orders for steel in full accordance with the C.R.B. specifications—though, no doubt, at a higher price. Further, the B.H.P. attitude to Izod tests was less unequivocally condemnatory than J. & W. understood it to be. It is probable that at this stage, the J. & W. officers did appreciate their responsibility to Utah and the C.R.B. to ensure that the tests would be carried out, but felt confident that they could arrange for the tests to be done by testing authorities in Melbourne. Also, they doubtless realized that if B.H.P. had agreed to supply the steel in accordance with specifications, the cost would have been so much greater, that in a competitive tender it would have been prohibitive. They, therefore, took a " commercial risk " hoping that most of the heats of steel would comply with the C.R.B. tests and that such as did not, would be useable for other purposes. They were comforted also by the opinion which they then held, but which turned out to be misleading, that the C.R.B. would not insist on the tests being carried out to anything like the extent provided for in the specifications. In all these circumstances it now seems highly probable that whatever may have been the situation at the time of the tender to Utah, the orders to B.H.P. were given without the additional tests as a deliberate act and not as any sort of oversight.

The importance of this matter cannot be overemphasized. When J. & W. became convinced that B.H.P. would not supply steel in accordance with the C.R.B. specifications, its plain duty was to have informed Utah and through Utah the C.R.B. The whole matter could then have been brought into the open before any orders were lodged. No one can now say what would have been the result of such disclosure, or even if the other parties would in fact have treated the matter with sufficient seriousness. If they had done so, however, the whole story could have changed—even to the extent that the use of the specified steel might have been abandoned in favour of some other material, with consequent alterations to design.

At the Enquiry it was urged against J. & W. that its failure to order in strict accordance with the specifications and the terms of its contract with Utah constituted a serious breach of its contractual obligations. The answer made was that the specifications required J. & W. to fabricate in steel manufactured to B.S. 968 which would be submitted to, and pass, the Izod and other tests, but that nothing in the specifications required these tests to be carried out by the steelmaker, and that J. & W. was perfectly entitled to order as it did, and at its own expense, have the tests carried out by other suitable testing authorities.

We are unable to accept this argument, although it appears to have been at least tacitly accepted by the other parties at the time. A perusal of the specifications, section 2—3, and in particular clause 2–3–16 (*h*) shows plainly that the obligation of Utah, and therefore of J. & W., was to arrange for the weldability test specimens to be prepared by the steelmaker and tested at its works or at an approved testing laboratory at its cost.

The subsequent history of the matter discloses that J. & W. did not carry out the procedure with strict regard to the onerous obligation undertaken, which was to ensure the proper testing of all steel, and the setting aside without demur of any steel which did not pass the tests on the strictest basis. In fact, there was a failure to carry out all the tests to the full extent of the specification requirement, and J. & W. is found resisting testing by every available means, constantly urging reduction in testing, and finally over-persuading C.R.B. to relax the testing. In this respect we feel that J. & W. was clearly at fault.

2.3.4.2. *Responsibility of Utah for the J. & W. orders.*

The failure of J. & W. to order the steel strictly in accordance with the C.R.B. specifications was a matter not disclosed to Utah by J. & W. at the time. Indeed, on 5th September, 1958, J. & W. wrote to Utah a letter containing the phrase " The enclosed copies of test certificates for all material received at our works to date are for the C.R.B. files. You will note the Izod figures are not given on these certificates, but as requested by Mr. Eastick, it is agreed that the B.H.P. company will provide impact tests for all future heats ".

This unfortunate piece of misinformation was explained by Stocker and Scarlett as an error by the latter. It was claimed to have been corrected verbally.

The question then arises was Utah to blame for not overseeing the orders given by J. & W. to B.H.P.? Utah had made clear to J. & W. that the sub-contract must be carried out within the terms of the main contract and had arranged for C.R.B. specifications to be made available. Utah, no doubt, assumed that a company which had been purchasing steel for 100 years was capable of giving a relatively simple order. Moreover, even if it had occurred to Utah to request a sight of the orders for the purpose of checking them before they were placed by J. & W., this would have been greatly resented by J. & W., as being out of accord with usual practice. In the light of the misleading letter of 5th September, 1958, it would be scarcely fair to say that Utah should have discovered the situation sooner, although this might well have been ascertained by a careful perusal by Utah of the J. & W. tender which made no mention of the additional tests. When the situation was discovered, the C.R.B. as well as Utah, accepted the position. We, therefore, feel that no adverse finding should be made by us against Utah on this matter whatever ultimate vicarious liability may be found to exist in law.

2.3.4.3. *Responsibility of C.R.B.*

It is by no means easy to decide when the C.R.B. first knew of the form of order given to B.H.P. by J. & W.

A perusal of the correspondence—without any other information—would lead to the conclusion that it did not know until after 18th February, 1959, because in a letter of that date, complaint was made to Utah that Izod test certificates were not coming forward, and in a further letter of 26th February, 1959, the following paragraph occurs—

> " It has been suggested that, when steel was ordered, an Izod test was not specified, and if this were so, it would seem possible that some of the steel, already supplied, could be unsatisfactory and would need to be rejected. Should it be found that material, as ordered on a limited specification, has been supplied, B.H.P. cannot very well be held responsible for any delay in completion of the contract due to non-compliance with the contract specification. If material is supplied by B.H.P. and does not comply with your order and there is a delay in advising B.H.P. accordingly, B.H.P. cannot very well be held responsible for that delay."

At this stage there was correspondence between Utah and J. & W. on the matter, and at the request of C.R.B., J. & W. wrote to B.H.P. on 10th March, 1959, asking for a reconsideration of its attitude, which was refused in a letter of 7th April, 1959.

On the other hand, the evidence of Eastick makes it quite clear that he knew as the result of conversations with Thompson early in 1957, that B.H.P. was unwilling to undertake these additional tests, and it is equally clear that he was quite unconcerned about this. He was interested only in seeing that efficient arrangements would be made for testing in Melbourne.

Whether Eastick ever passed this information on to his superiors at that time (1957) is uncertain. However, Eastick claimed that at a conference on 21st March, 1958, at which Butler and Eastick represented C.R.B. and Miller and Fink, Utah, he made the position clear to all present, and the notes of the conference (Part of Ex. 51) bear this out. In any event, Eastick was the Board's Officer in charge of testing, among other things, and his knowledge must be taken as that of the Board. It is abundantly plain that Eastick and probably other C.R.B. officers except Wilson, shared with J. & W. the lack of understanding of the significance of the impact tests.

As late as 14th December, 1959, at a conference between Eastick, Bonwick, Scarlett and Ferris (See Ex. 75) it was possible for the question to be asked of Ferris, " Why are Izod tests required when B.S. 968 does not specify same?". Even Ferris's forthright reply that he would " have nothing to do with the use of B.S. 968 if adequate assurance of its resistance to brittle fracture was not available " seems to have passed over the heads of his enquirers. At the hearing, Eastick's evidence left us with the impression that even then he had not grasped the significance of the Izod tests.

Whatever be the time at which the C.R.B. had knowledge of the form of order placed by J. & W., there is no doubt that it accepted the situation with very little protest. Apart from urging J. & W. to make a belated appeal to B.H.P. for reconsideration, in March, 1959, no other action was taken. Perhaps at that stage nothing else could have been done.

The real criticism of the C.R.B. arises from its subsequent actions. Because of the failure to appreciate the importance of the impact tests, they were never carried out to the full extent of the specifications, and the least onerous of the extra tests were relaxed to a " random " basis.

While inspection in general was being maintained with such rigorous insistence on detail as to seriously irritate the J. & W. personnel, the impact tests were allowed to be greatly reduced. An even more serious error was to permit, or at least to acquiesce in, the practice of J. & W. of proceeding to fabricate girders from steel which had not been tested, subject to a satisfactory result being ultimately obtained. This was, we feel, very bad practice which the Board should have refused to accept. A further criticism must be added, of breakdown in communication within the Board's own establishment. Wilson, the man mainly responsible for the specifications, who had added these additional tests in order to protect the bridge from brittle fracture, was the one witness from the Board's staff whom we consider to have understood the significance of these tests. He was never informed, much less consulted, about these matters. Had this been done, we feel that a very different course would have been followed.

2.3.5. C.R.B. organization to control the contract.

After he had finished his work on the specification Wilson apparently spent most of his time in connection with the bridge foundations. The knowledge which he had gained from Ferris and elsewhere about brittle fracture and related matters does not seem to have spread from him to other officers of the C.R.B. and we must enquire why this should have been so.

The organization of the C.R.B. during the relevant period, in so far as it directly concerned Kings Bridge, was as follows :—

Chief Engineer	.. J. Mathieson
Engineer for Bridges	.. I. J. O'Donnell (when specifications were being prepared) C. A. Masterton
Senior Design Engineer	.. C. A. Wilson
Supervising Engineer, King-street Bridge	.. L. T. Butler
Senior Constructional Engineer (Bridges)	.. R. F. Eastick
Assistant Engineer	.. F. Jackson
Welding Inspector	.. N. V. Clarke (with two assistants)

A great deal of the information that we obtained about the conduct of the job came from the evidence of Wilson, Eastick and Clarke although Mathieson, Masterton, and Butler also appeared briefly at the Enquiry. A paper by Masterton (Exhibit 22) presented to the Institution of Engineers, Australia, gives what must be presumed to be the official account of the construction procedure and contains a number of statements that are worthy of comment. It should perhaps be mentioned that we do not know for certain how much of the paper was written by Masterton of his own knowledge and how much was based on information supplied to him by his subordinates.

At any rate the early part of the paper indicates that Masterton, possibly advised by Wilson, had a very lively appreciation of the inherent difficulties of welding, of the sort of failures of welded structures that had taken place previously, and of residual stresses and their implications. The introduction of the additional clauses into the specification, to try to cover the danger and difficulties, are then described and are followed by an account of Jackson's experiments, described in Section 2.4.3, to improve the knowledge of the C.R.B. officers in welding this material.

During the construction of the bridge the regular inspection of the welding was carried out by Clarke and his assistants working under the immediate supervision of Jackson : Eastick was the officer in general charge of the fabrication of the girders from the C.R.B. point of view while Butler was in charge of the project as a whole.

It is difficult to believe, in view of their actions, that Butler, Eastick and Jackson really appreciated the situation properly or that the knowledge that Wilson and Masterton possessed was effective at the time the contract was awarded. For example we read in Masterton's paper, under the heading " Welding High Tensile Steel ", the following :—

" It was not known if steel made to specification B.S. 968 had previously been fabricated by welding for girder construction. It had not been so used to any appreciable extent in Australia.

From experimental results in the U.K. it was ascertained that an effective welding procedure had to be established, maintained and constantly checked to ensure trouble-free welding.

In these circumstances it was, therefore, necessary for the Board to satisfy itself :—

(i) that high-tensile steel of Australian manufacture could be satisfactorily welded manually;

(ii) that welding procedure and controls were developed suitable for mass production under workshop conditions;

(iii) that a procedure was established suitable for the fabrication work being performed by automatic and semi-automatic processes; and

(iv) that the non-destructive testing of this welding could be satisfactorily arranged."

We can only comment that the Board allowed itself to be all too easily satisfied.

There are several other passages in the paper which describe what might have happened rather than what actually did happen. For example :—

" The contractor supplied test certificates from the manufacturer which gave the physical properties and the chemical composition of each plate for each heat." This was never done, as Masterton admitted in evidence. " The test certificates and the check analysis, together with the Izod and weldability test results, were entered in the record book, and if they fulfilled the requirements specified then the contractor would be officially informed of the approval to the plates covered by the Board's stamped number."

It may appear to be a small point but we think it significant that there was no proper record book. There was a chart (Exhibits 71 and 72) but we found that many plates were approved that did not conform to the C.R.B. specifications. Nor were weldability Izod tests ever carried out at the lower temperature, although it would appear from Appendix I. of the paper that Masterton thought they had been. On the other hand Appendix III. describes Jackson's experiments and makes it clear that testing was not carried out at the lower temperature.

From Masterton's paper and from the evidence we heard we conclude that there was a failure of communication within the C.R.B. organization. On the one hand important background information did not reach the officers on the job ; on the other the actions of these officers were not always fully realized at appropriate levels in the C.R.B. In particular, Wilson did not know until the Enquiry that the weldability Izods had not been carried out at 32°F. and was manifestly shocked by the information.

In this connection we find a gap in our knowledge. We know that Wilson and Masterton visited England and the Continent of Europe in 1956 partly to answer enquiries from possible tenderers—and also, perhaps, to stimulate such enquiries, and partly to make a general survey of current bridge-building practice. We do not know whether a full report of this visit was ever made to the Board as there was no evidence on this point. If no such report was made a great opportunity was missed, for Masterton's diaries contain a great deal of excellent information, some of it highly relevant to the subject of our Enquiry.

The value of such a report would have been two-fold. In the first place Wilson and Masterton themselves would have benefited by the act of reducing to an orderly sequence the heterogeneous notes and impressions of hurried visits to many offices and works. And, secondly, the circulation of their report could not have failed to improve the understanding of all concerned with Kings Bridge.

2.4. Competence and Experience of the Parties to Undertake the Project.

2.4.1. Relationship between the parties.

Throughout the Enquiry it has been evident that there was a complex interplay in the relationship between the various parties concerned. We address ourselves, therefore, to a consideration of the competence and experience of the parties on the one hand and of their responsibilities on the other. We are acutely aware of the difficulties besetting any definition of moral standards in this connection and will, therefore, so far as possible, confine our attention to the responsibilities as defined in the several contracts or stated in the C.R.B. specifications. At the same time we recognize that many companies assume responsibilities, of a professional character, that transcend their contractural obligations. It is good that this should be so.

In order to get a clear picture of the organization concerned in the construction of the bridge, this is set out diagrammatically as follows:—

C.R.B.—CONSTRUCTING AUTHORITY.

Wrote the specifications governing the whole project. Retained final authority for approving all materials and fabrication procedures, including inspection of completed girders.

UTAH AUSTRALIA LTD.—CONTRACTOR.

Awarded the contract for the design and construction of the complete structure but actually itself only constructed the foundations, substructure, and deck. The whole design and the supply and erection of the steel work were sub-contracted.

SUB-CONTRACTORS.

King-street Bridge Design Ltd.

Responsible for design, both of foundations and superstructure.

Johns & Waygood Ltd.

Responsible for fabrication, painting, and erection of steel work in the superstructure. This included the detail shop drawings and welding procedures.

Broken Hill Proprietary Company Ltd.

Steel plates and bars.

Murex (A/asia) Pty. Ltd.

Electrodes and fluxes.

It is clear from this that the contractor came between the constructing authority and the sub-contractors. This does not appear to have restrained direct discussion of design aspects between C.R.B. and K.S.B.D., as is evidenced by the numerous design conferences held. On the other hand, official contact was maintained between C.R.B. and J. & W. only through Utah. It will be realized from the above diagram that B.H.P. and Murex had official contact only with J. & W.

Although C.R.B. was the final authority for accepting steel, there was no discussion with B.H.P. at any stage of the contract regarding the quality of the steel supplied or difficulties associated with its fabrication, apart from some informal discussions between Eastick (C.R.B.) and Ralston (B.H.P.) in the early stages. Whether this was an inevitable consequence of the type of contract, or not, there is no doubt that discussions between C.R.B. and B.H.P. at the appropriate level, could have had important and beneficial consequences.

In setting out our views on the competence and experience of the parties it must be realized that our sole concern is to establish whether, in the circumstances related to the initiation and fulfilment of the contract, the parties were familiar with the low-alloy steel selected for fabrication of the superstructure. This matter relates to all the parties mentioned except Murex.

2.4.2. Mr. I. J. Ferris.

Ferris is the Principal Scientific Officer in charge of the welding and physical metallurgy group for the Defence Standards Laboratories of the Commonwealth of Australia. He is a well-qualified metallurgist with many years' experience that was highly relevant to the use of welded high-tensile steel in the Kings Bridge.

At the Enquiry it soon became evident that he had played an informal but important role before and during the construction of the bridge ; he participated in the work of the committee of investigation immediately after the failure ; he subsequently carried out a detailed study of the failed girders (Ex. 194) ; and finally he assisted the Commission by giving very valuable evidence.

At the time the specifications were being written he gave much advice to Wilson on the steel specifications generally, and on B.S. 968 in particular. He confirmed the desirability of introducing Izod tests and of limiting the thickness of the steel to 1 inch.

He gave advice which led to the introduction of the welding procedure described in the B.W.R.A. booklet ; advised the C.R.B. to employ an independent organization for the inspection and testing of the welding ; gave Jackson (C.R.B.) some training in welding techniques so that he might undertake inspection ; gave advice on radiographic inspection and the interpretation of radiographs ; and finally, advised Wilson to recommend that the bridge should be closed after the failure of span W.14.

During the fabrication period there seem to have been several occasions on which he was consulted by one or other of the parties who turned to him for advice.

Apart from a payment by C.R.B. to D.S.L. for the training of Jackson, there was no payment made either to D.S.L. or Ferris by any of the parties.

In the early stages of our proceedings the impression could have been gained that he was the man on whom the main responsibility for the failure must rest, for it seemed that many decisions of importance were only taken after visiting, or telephoning him. As the evidence unfolded, however, it became abundantly clear that he at least was one man who really knew what he was talking about. With the one exception that the Izod value (of 20 ft./lb. at 32° F.) he suggested should have been higher (and he explained the authoritative basis of his figure) his advice was absolutely sound. We were finally left with the opinion that, far from being held responsible for the failure, he must be given full credit for trying to get the various parties working on the right lines.

While we are more than satisfied that Ferris gave excellent advice we are not sure that he was always wise to give it especially to people who might not fully understand the significance of what he was saying. Nor can he always have been certain that he was being asked the right questions. We do not blame him for his ready willingness to help but we very much blame those who put him in such a difficult position.

In particular we blame the C.R.B. which leaned most heavily on him and which should have had a competent metallurgist on its own staff. No doubt there would still have been problems where Ferris's great experience would have been of value but the questions would then have been asked, and the replies received, with understanding.

If the bridge had been built of mild steel it is quite possible that no great difficulties would have arisen with this familiar material ; in that case no exception could be taken to the occasional use of Ferris to help with special problems. With the high-tensile steel that was actually used it was essential to have experts fully involved in, and in daily contact with, the problems of fabrication. In spite of all his knowledge and experience, Ferris just could not do what was required as a part-time consultant at the far end of a telephone line.

2.4.3. The Country Roads Board.

The C.R.B. selected B.S. 968 : 1941 after consultation with Ferris of D.S.L. However, the C.R.B. engineers had no experience with this steel, nor had they anybody on the C.R.B. staff who had the necessary metallurgical knowledge, not only to advise them of the difficulties but to aid them in the over-all control of the welding contract. As constructing authority assuming the right of approval of materials and procedures it needed the full-time services of a competent metallurgist. The welding experience of its officers was with mild steel. Whether they knew it or not, they were taking a bold step in specifying low-alloy steel—if this should prove to be the material selected by the successful tenderer.

We appreciate the view put by Darwin that no progress is made in engineering practice unless you are prepared to try something new. We think, however, that the caution normally exercised by engineers in such cases should have led him to cause a thorough investigation of the properties of this, to them, new material. Darwin and his staff take refuge behind the authority of a British Standard Specification. We think nobody on the C.R.B. staff understood all the implications of the new material. It was, at first glance, too like mild steel for them to realize its considerable differences. It is well known that the introduction of alloying elements into steel makes it more sensitive to cooling rate and to weld cracking. To overcome these difficulties on a routine basis it is essential to reorganize the fabrication shop. They did not appreciate this sufficiently to be able to assess the ability of J. & W. to undertake the necessary reorganization for this purpose.

The only effort the C.R.B. appears to have made to become acquainted with the material it was decided to adopt, is described in the paper by Masterton (Exhibit 22). In this he states *inter alia* " It was not known if steel made to specification B.S. 968 had previously been fabricated by welding for girder construction ". This statement refers to the state of knowledge of the C.R.B. at the time of writing the specification. In view of this, our opinion is that the C.R.B. did an absolute minimum of testing on the properties and weldability of the steel. From the experiments done, however, there could have been extracted the following pieces of information which should have warned of difficulties ahead :—

(*a*) This steel sometimes does not show a yield point. This feature should have been discussed with K.S.B.D.

(*b*) The composition of the samples of Australian steel used was variable and sometimes above the maximum limits specified in B.S. 968. This should have alerted C.R.B. to require J. & W. to have checks made on the steel delivered and to have had direct discussion with B.H.P.

(*c*) In an experiment to determine the magnitude of residual stresses in the flange-web welding, although appreciable stresses were indicated the matter was not followed through. The experiment was made on thinner plate than was subsequently used in flanges and there was no evidence that consideration was given to the use of pre-heating or of variability of welding sequence on residual stresses.

Considering the importance placed by Wilson on the need for notch ductility in the steel, one of the most remarkable aspects of the preliminary testing programme is the omission of a survey of this property on the samples investigated. In Masterton's paper only two plate Izod tests are mentioned and these were carried out at room temperature. Even at this early stage the significance of variation of notch ductility with temperature appears to have been overlooked. To be able to assess this property it was essential to have a series of Izod specimens over a range of temperatures from 0° F. to at least 120° F., from a series of thicknesses of plate covering the range to be used in fabrication of the girders. By comparison, this procedure was followed by Mott, Hay and Anderson, consulting engineers for the Runcorn-Widnes Bridge being fabricated in England from similar steel at about the same period. Without this knowledge obtained from several heats there was no certainty regarding the possible notch brittle behaviour of this steel.

In view of the fact that the C.R.B. had retained authority to approve the steel and all fabrication procedures, we consider that with the staff available it was not competent to carry out these functions.

Another matter is the ability of C.R.B. officers to undertake the inspection during fabrication of the girders. Admitting that they were working with a new material, they took the view that it would be better to avoid using inspectors who were familiar with the inspection of mild-steel welding, and preferred to train a new group.

The first step was to send Jackson, Eastick's assistant, to D.S.L. for a few weeks (See Exhibit 21) to acquaint himself with the characteristics of B.S. 968 steel and with the techniques of welding it. Ferris felt confident that Jackson understood what he was taught but that when he left D.S.L. he did not have sufficient knowledge to supervise

adequately the fabrication of the bridge. (Tr. 2639). Nevertheless, he did produce two reports (Exs. 53 and 54), on the welding of B.S. 968 steel, which are competent pieces of work as far as they went.

Incredible as it may now seem, this was the only direct experience that any C.R.B. officer had of welding B.S. 968 steel before they began the work of inspection.

Jackson's knowledge doubtless grew on the job but it is to be questioned whether he was really capable of training the inspectors even though they were competent welders of mild steel. He did, however, produce a sensible instruction sheet (Ex. 59) for the inspectors.

Jackson was abroad at the time of the Enquiry and was not available to give evidence. From what others said of him, and from the exhibits mentioned above, we formed the opinion that he was capable and conscientious, but whether he had enough experience to do what was expected of him, or whether he was adequately supervised himself, are very different matters.

We believe, therefore, that the C.R.B., in spite of its long experience in bridge building, was not competent to undertake the supervisory role it had assigned to itself in the specifications ; it did not fully realize the implications of the change to high-tensile steel and the steps it took to prepare itself for the job were inadequate.

In view of the opinions expressed above we think it worth while to include an account of the careful and considered steps by which, over a period of several years, the California Division of Highways moved into the field of welded high-tensile steel bridges. Appendix 4, therefore, consists of extracts from a paper by the Bridge Engineer in Planning of that authority.

The paper itself was included by Bonwick in his report to Utah (Ex. 24) following his visit overseas in early 1960.

2.4.4. Utah Australia Ltd.

There seems to be no doubt that the C.R.B. regarded Utah as the most impressive of the contractors whose tenders were low enough to merit serious consideration. There were good reasons for this attitude. The American principal of Utah was known to be a major contractor of wide experience with a long history of successful work. The Australian subsidiary, although not very long established, had carried out work at Eildon and in the Snowy Mountains which had demonstrated its capacity to bring to Australia much of the skill and drive that characterize the American construction industry. By employing senior staff from America, supplemented by Australian assistants and workmen, and by maintaining an adequate liaison with the parent company it had built up a good reputation, especially in the field of earth-moving and similar work.

Although Utah had had no experience of bridge building its proposal to construct the foundations itself, and to sub-contract the design and the fabrication and erection of the steel work, while maintaining over-all responsibility for the job, seemed to make the best of both worlds.

The only weakness of this plan proved, in the event, to be a fatal one : neither Utah nor the C.R.B. nor, for that matter, J. & W. itself realized that J. & W. was not competent to handle the problems inherent in the use of high-tensile steel. It is unfortunate that the American company, to which the designs and, presumably, the arrangements as a whole were submitted, did not ask some pertinent questions on this point.

As the general contractor, responsible for the job as a whole, Utah had a clear duty, first to satisfy itself that its sub-contractors were suitable and, when experience showed that they were running into difficulties, to do something about it.

It is obvious that the Utah personnel in Australia knew nothing about high-tensile steel and, like almost everyone else, took B.S. 968 on its face value. When Hardcastle's investigations in 1956 showed that it would save weight there does not seem to have been any thought that a full study should be made of the possibility of using it on a big scale in Australia. Enquiries in California—and the Executive Headquarters of the Utah

Construction Company are in San Francisco—would have shown that the successful and extensive use there of welded mild and high-tensile steel for bridges has involved what amounts to a transformation of the steel construction industry. Radiographic inspection of welds, which was so much resented by J. & W., is recognized as essential not only to check the work produced but to develop techniques and train welders. Beaton writes of " shops . . . being subject to the discipline of radiography " and the papers that Bonwick included in the report of his visit to U.S.A. and U.K. in 1960 (Ex. 24) reveal a technical awareness that was pathetically lacking on the Kings Bridge.

At the Enquiry, Counsel for Utah was at pains to show that the responsibility for ensuring the competence of the sub-contractors rested exclusively on the shoulders of the C.R.B. " The authority at least is satisfied, and that was all that any reasonable contractor, in our submission, would be required to do ". (Tr. p. 190). We do not accept this view. One of the purposes that the C.R.B. had in mind, in advertising the contract on a world-wide basis, was that a contractor might emerge who would bring to Victoria expertise that was not locally available. C.R.B. thought it had such a contractor in Utah but was disappointed in the event. We do not exonerate the C.R.B. from a share of the blame but we do not think that Utah did all that could reasonably have been expected of it. The resources that were readily available to Utah from its American principals were not brought to bear on the fabrication of the steel.

2.4.5. *King-street Bridge Design Ltd.*

The consultants who formed themselves into the above-named company presumably did so because no one of them felt that he had sufficient experience, or a big enough office, or both, to tackle a job of this magnitude unaided.

In the event four engineers, each from one of the constituent firms but none experienced in the use of high-tensile steel, were mainly concerned and of these Hardcastle seems to have played an important, if not the leading, part ; at any rate, he came forward at the Enquiry as the spokesman for the group.

By 1957, Hardcastle had had some eight years in practice as a structural designer and had won for himself a good reputation in this field. He had not previously designed a bridge, however, and it is therefore necessary to ask whether he should have taken on, or been entrusted with, a bridge project of this magnitude ; to the layman it might appear not. But the principles of structural design are very much the same whether a bridge is being designed, or a crane, or the framework of a building. We can find no evidence whatever that Hardcastle himself, or the team of which he was apparently the leader, should not have been entrusted with this work. We discuss, in Section 2.6.2., the only two features of the design that were open to criticism. It is now only necessary to point out that although Hardcastle said (Tr. 480) that the final decision to use high-tensile steel was made by Utah and K.S.B.D. jointly, and, although K.S.B.D. had reported, on the strength of a trial design, that B.S. 968 appeared to offer the most economical solution, the ultimate responsibility is that of Utah. It must be realized that the position of the designers in this contract was a subordinate one. They were employed by, and were responsible to Utah, whose engineers, Miller and Longo, who were regularly in contact with Hardcastle and who, after considering the matter and consulting J. & W., finally made up their minds to submit a tender based on B.S. 968 steel. We think that within the limits of the responsibility assigned to it, K.S.B.D. was competent to carry out the work for which it was engaged.

We retain our preference for a different arrangement, as discussed in Section 2.1., under which C.R.B. would have engaged a consulting engineer to design a bridge for the construction of which contractors would have been invited to tender. It would then have been his duty to have investigated the suitability of B.S. 968. As it was, as far as we know, Utah never called upon K.S.B.D. to make any real investigation or to give any technical advice on this matter at all.

We also think it important to comment, at this point, on the influence that the competitive situation, in which the designers were working, had on the decision to use high-tensile steel. It was necessary for them to produce not only a satisfactory design but also one which would be acceptable on price. Hardcastle's trial design showed a saving in weight and the very sketchy investigation, which was all that was practicable at the tender stage, suggested that it would also show a saving in cost. We cannot, naturally, know what

Hardcastle would have done if he had been retained as a consultant by C.R.B. to design the bridge, but he would at least not have been under pressure to produce a competitive design and could have had a full investigation of B.S. 968 made before choosing it.

This consideration underlines the objections to this form of contract which we have described in Section 2.1.

2.4.6. Johns & Waygood Ltd.

This company which was established more than a century ago, has acquired a good reputation as a fabricator of structural steel work. In recent years it has built a fine shop at Sandringham where a large tonnage of welded mild-steel work is produced annually. The equipment and facilities of this shop appear to be at least equal to those in comparable shops abroad working in a similar class of work to J. & W. but we must enquire whether they were adequate for the higher class of work required if B.S. 968 was to be satisfactorily fabricated.

The manager of the structural department was Mr. W. C. Farrar, whose extensive experience of design and construction began in 1929. He was generally responsible for all phases of the J. & W. contract and, in particular, was closely involved in the preparation of the tender to Utah. In this context Farrar said : " My knowledge of welding is such that I am able to prepare estimates committing our organization to firm bids on competitive tenders and if successful to organize the execution of them ". As neither he nor anyone else in J. & W. were personally familiar with fabricating B.S. 968 steel he relied on the assurance of B.H.P. that thousands of tons of similar material had been supplied by it to the Naval Dockyards and had been satisfactorily welded.

In preparing his tender Farrar considered and made some (but, as it turned out, insufficient) allowance for pre-heating, testing of the steel, and radiography of the welds. Regarding the latter he said : " Had the Board stated that field radiography inspection would be imposed if B.S. 968 steel was used but not if mild steel was adopted, the use of B.S. 968 steel would not have been considered for a moment ".

We have a good deal of sympathy for Farrar. He was preparing a tender, under some pressure of time, for a job in an unfamiliar material ; he tried to extrapolate from his extensive knowledge of handling mild steel to a new situation and relied on advice which was literally true but actually misleading. He was working to a specification the implications of which neither he nor, at that time, the C.R.B. fully understood. In consequence his expectations of the extent to which J. & W. would have to reorganize to deal with the contract, were quite inadequate.

The officers immediately in charge of the fabrication and welding were Mr. W. R. Stocker, and Mr. K. F. A. Scarlett, both of whom were very experienced in working with mild steel but lacking in experience of high-tensile steel and its properties.

J. & W. does not employ a metallurgist in its organization " because there would not be sufficient duties to keep him fully occupied and our policy has always been to seek advice from an outside expert for metallurgical queries. For example, regarding quality and supplies of steel we discuss our problems with B.H.P. who have their highly-qualified technical officers and well-equipped laboratories at their service. For electrodes and welding techniques we have ready access to, and in fact are encouraged to use, the competent technical experts of the various companies supplying electrodes and welding equipment to us and for any special investigations we would engage the services of an expert consultant or analyst depending on the nature of the advice required."

Although it is not our function to offer advice to the parties concerned in this project we feel that an opportunity for preventing similar mishaps in the future will be missed unless we draw attention to organizational deficiencies which contributed to the bridge failure.

Whilst it is evidently possible for a fabricator in mild steel to work without a metallurgist it is not possible with alloy steel. Had J. & W. been used to examining welds metallurgically, and been aware of all the technical pitfalls which beset the fabricator in alloy steel, the whole character of the organization could have been changed. A metallurgist would have aided the training of welders ; checked the chemical analysis of the plates so that the inevitable variations in composition could be assessed ; carried out the Izod tests ; helped with the assessment and control of pre-heat ; and supervised the use of

non-destructive weld tests by radiography and other means. This involvement of a fully-trained applied scientist in its work would have raised the whole tone of the J. & W. establishment. It would have been well worth while even for this contract alone and of lasting benefit to J. & W. as a leading fabricator.

As it was there was no one at J. & W. who understood, or had any interest in, the testing programme. The proper care and drying of electrodes, the calculation and measurement of pre-heat, which added to the technical difficulties and expense, the orderly determination of welding sequences and the dissemination of unambiguous instructions to the welders, and the keeping of adequate records were foreign to their normal practice. It did not become a member of the British Welding Research Association until June, 1960, and we have no evidence that its officers were abreast of the technical literature of their subject. All these things are indicative of their unscientific attitude.

These remarks must be read in the context of the high standard of technical competence required to handle alloy steel. So far as mild steel is concerned the equipment was entirely suitable, the welders skilful and the management experienced.

But in 1957 the company's organization, particularly in technical control, was not adequate to undertake the fabrication of girders in low-alloy steel.

By contrast, we heard from Mr. Reedy how Caterpillar of Australia Pty. Ltd., had organized itself to use large quantities of B.S. 968 plate in the manufacture of its earth-moving plant. While there is an obvious difference of scale between Caterpillar's work and J. & W.'s, this was a most impressive account of intelligent technical control which had enabled a local fabricator to know for itself, by analyses conducted by its own staff, within what limits of chemical composition this steel could be satisfactorily welded, and to lay down appropriate procedure. It was also evident that this mastery of the job was not lightly won but was the result of long, arduous and expensive investigation and experience.

2.4.7. Broken Hill Proprietary Co. Ltd.

While B.H.P. is generally regarded as a single organization, the fact is that the steel-making plants at Newcastle and Port Kembla enjoy a certain measure of technical autonomy. The Port Kembla branch, operating under the name of Australian Iron and Steel Pty. Ltd., is the one with which we are directly concerned since practically all of the steel used on the bridge was made there.

The sales organization in Melbourne apparently serves the whole company and Mr. O. B. Ralston, the service officer who conducted correspondence and had discussions with J. & W. about the contract, also represents both plants and works from the Melbourne office.

Mr. J. W. Thompson, who is a metallurgist by training, was the Assistant Executive Officer, Administration, at Port Kembla and was in charge of the chemical and metallurgical laboratories, research and inspection there during the period when the steel was being manufactured for this contract.

The company as a whole manufactures a wide range of steel and steel products; steel to B.S. 968 for Caterpillar and other customers, and the very similar D.W. plate for the Navy, had been made for some time prior to the bridge contract. On the basis of this experience, B.H.P. had no hesitation in agreeing to supply steel for the bridge and in assuring J. & W. that it would be weldable. We discuss elsewhere (Section 2.3.4) the attitude of the company to the additional tests specified by the C.R.B.

On the face of it, therefore, one would suppose that B.H.P. would have been well able to play its part adequately in the Kings Bridge contract. It had enough qualified staff and laboratory and plant resources to do all that was required. Yet we find that the production of some 2,000 tons of steel involved the diversion to other uses of far more steel than was sent to the bridge, while of that which was sent, several heats could, with advantage, also have been diverted. The reasons for this are not clear either to us, or, we feel, to B.H.P. itself. We understand that the two plants used different techniques for producing this material and we wonder whether there was any collaboration or discussion between them on the metallurgical aspects of the problem.

Although much good steel was delivered to the bridge the steel as a whole must be regarded as unsatisfactory for three reasons :—

(a) It was too variable in quality.

(b) Much of it was found to be difficult to weld because of the high carbon, manganese and chromium content.

(c) Some of it, notably heats 55 and 56, was notch brittle.

Brittle fracture is a problem that has caused anxiety in recent years to the designers of large plate structures, such as ships, storage tanks and bridge girders, especially when they are welded. It is a problem that is still not completely solved but a good deal of progress has been made ; the influence of stress raisers and low temperatures is recognized, suitable acceptance tests are being devised and so on. But, in the last resort, it is to the steelmaker that we must look to produce steel that is notch tough.

In other countries much progress has been made in this direction and, to this end, extensive experimental and steel production programmes have been carried through. Such work as was done in B.H.P. laboratories was on a much smaller scale and would be classed as " testing " rather than " investigation ". The apparent lack of interest in a topic that is of sufficient importance to justify a place in the forefront of the research programme is disappointing, particularly as there have been several brittle fracture failures in Australia, in recent years, in some of which B.H.P. steel must have been involved. We have full details of only one of these failures which is described in the Report of the Proceedings of the Fourth Conference of Professional Officers, representing the Controlling Authorities of the Water Supply and Sewerage Services of the Capital cities and of the States of Australia. The conference was held in October, 1949 and one paper describes the 33 brittle fractures that had occurred up to that time during the fabrication of some 6 miles (out of 23 miles total) of 69–in. diameter pipes. Some of the remarks made during the discussion of that paper might almost have been made at out own Enquiry and are worth quoting :—

> " B.H.P. had been approached in the matter but unfortunately they had not been very helpful. Their attitude seemed to be ' that is our standard grade of steel. If you don't buy it there are plenty of other customers who are willing to take it '."
>
> " . . . . if it could get a steel corresponding to B.S. 968 difficulties would be overcome. B.H.P. were prepared to make such steel if sufficient orders were forthcoming ". " One thing they (B.H.P.) did ask and that was that the British Standard should be relaxed somewhat in the tolerances ".

We quote these remarks because they indicate an attitude that apparently still prevails in B.H.P. To be fair, 1948 was a time when steelmakers were hard-pressed to meet the demand for steel, and B.H.P. has since made a great deal of B.S. 968 steel, as we know ; moreover, B.S. 968 : 1941 has not been as successful a specification as had been hoped and has since been replaced by B.S. 968 : 1962. On the other hand, B.H.P.'s attitude to its customers and to tolerances has scarcely changed and it still has much to learn about brittle fracture.

It may perhaps be argued that if research on this topic is being actively pursued in various places elsewhere, there would be no point in B.H.P. duplicating it. This may be so, but we think that there is a need for a strong technical group directing continuing investigations on basic problems associated with the steel-making, pit practice and mill practice of quality steels. Such a group might have resolved that, in view of work proceeding elsewhere, its own efforts should be concentrated on other problems, but there would at least have been people in the company reading current scientific literature with comprehension, and ensuring that up-to-date knowledge permeated the whole organization.

We recognize, of course, that B.H.P. has a research department at Newcastle, and technical laboratories at Newcastle and Port Kembla, but these laboratories do not appear to have been working in a concerted manner on the kind of problems with which we have been confronted in this Enquiry.

B.H.P. has a wonderful record and has played an incomparable part in the development of Australia. It has certainly not been slow to introduce all sorts of innovations in the field of steel production, to take but one example, but we consider that the events with which we have been concerned have shown up a weakness in its policy and outlook which made it, at the time of the contract, not fully competent to produce steel of the appropriate quality.

We describe elsewhere some incidents, among several that came to our notice, which suggest that there was room for an improvement also in the flow of information between the plant and the service department. Problems of communication of this kind are inevitable in a big organization and, unfortunately, they are seldom brought to notice except when something goes wrong. The system of internal communications operating at that time in B.H.P. should have been adequate, we think, and the incidents referred to were more the results of errors by the officers who used it than of the system itself. This matter is discussed again later, but for the present, we simply remark that it would have been better if more of what went on between Ralston and J. & W. had been recorded in memoranda and circulated within the company so that a project of this importance might have received closer attention by all concerned, especially in the early stages.

2.4.8. Murex (Australasia) Pty. Ltd.

The headquarters of Murex Welding Processes Ltd. are in England; the head office and works of Murex (Australasia) Pty. Ltd., are in Hobart, where a research laboratory is maintained, while an office for handling sales is located in Melbourne.

The company is a well-known one in the welding industry as a manufacturer of electrodes, fluxes and electric welding equipment.

In June, 1958, the Melbourne office of Murex was asked to recommend an electrode for the manual welding of B.S. 968 steel and, after certain investigational work at Hobart, a "low-hydrogen" electrode, Fortrex 35, was recommended. This electrode consists of a central wire coated with a flux which is specially designed so that the amount of hydrogen liberated during welding is kept to a minimum; this makes it particularly suitable for use with low-alloy steel of the B.S. 968 type.

Electrode acceptability tests were carried out to the C.R.B. specifications by Murex under C.R.B. supervision; the welded plates were examined by Royal Melbourne Technical College and, as a result, Fortrex 35 electrodes were approved and used throughout the project for manual welding. "Murawire" and "Muraflux" were later approved by the C.R.B. for automatic welding.

When J. & W. started the fabrication of the girders it ran into trouble because it was quite unprepared to weld B.S. 968 steel to the required standard. An arrangement was then made under which Mr. F. A. Ward of Murex was seconded to J. & W. to instruct welders in the use of welding techniques necessary to obtain good X-ray results when using low-hydrogen electrodes.

It is apparent that Ward set about the task of transforming the J. & W. shop so as to make it capable of producing work of adequate standard and he certainly improved the standard of welding. Ward is an experienced and intelligent man who holds a number of certificates which testify to his competence and had spent some thirteen years as a part-time welding instructor at Royal Melbourne Technical College. As well as relying on his own experience he was constantly in touch, through the Melbourne office, with the research department in Hobart.

The correspondence to and fro (Exs. 151 and 180) gives a very clear and convincing picture of a well-organized company with experienced metallurgists at the centre, constantly providing their technical representatives in the field with valuable advice and information. We have no doubt that Murex was well able to do what was required of it and, indeed, it was asked to undertake, and achieved, far more than one would have expected in the circumstances. The reorganization that Ward brought about at J. & W. went some way to bringing it to the standard required for this class of work.

2.4.9. Conclusion on the competence and experience of the parties.

We discuss in Section 2.7.1.1 the question whether C.R.B. was justified in including B.S. 968 in the specifications but, in any event, in the light of our discussion of the competence and experience of the parties, it is evident that C.R.B. should have taken certain additional steps which were not taken.

Clause 1–1–10, Assessment of Tenders, lists the factors which the C.R.B. announced that it would take into account in assessing the tenders. Among these is :—

(*f*) Ability of tenderer to finance contract.

There should, in addition have been a clause—

" (*g*) Ability of tenderer to carry out the contract."

The C.R.B. had, in fact, made provision for obtaining the information on which (*g*) might be assessed for, among the documents which the tenderer was required to complete, was Form G, which included :—

(*l*) List of Major Bridge and other Civil Engineering Works designed and constructed by Tenderer.

(*a*) Design (four columns with headings, Description, Constructing Authority, Amount, Design Time).

(*b*) Construction (four similar columns).

Further parts of the form enquired about the major items of plant used, the number of design engineers and draughtsmen or workmen employed per week at peak of design or construction, and the technical literature in which projects are described.

Utah's answer to all the questions was " See accompanying brochure labelled ' Utah Construction Company '." This brochure (Ex. 19) is an impressive account of the achievements of this great company in the construction of many important projects, including several that are of world renown. Its inclusion as part of Utah (Australia) Limited's tender clearly implied that the full resources of the American principal were to be thrown behind the Kings Bridge contract, and the C.R.B. appears to have left the matter there.

The specified tender documents did not require, and the actual tender did not supply, any details of the competence of the sub-contractors although clause 1–2–14 states that " The contractor shall not without the prior written consent of the Board (which may be given upon such conditions as the Board deems fit and which shall not relieve the contractor from liability under this contract)—

(*a*) sub-let any portion of the works ; or

(*b*) enter into any sub-contract for the execution of any portion of the works ".

It is clear, therefore, that the C.R.B. relied on the general competence of Utah and, later, of J. & W., and did not pursue any adequate enquiries on their specific competence to weld low-alloy steel.

Moreover, the C.R.B. did not take adequate steps to fit itself for its task as " Engineer " as described in the specifications. (See Section 2.3.5.)

These initial mistakes on the part of the C.R.B. had serious consequences for they led to a series of assumptions that had, in actual fact, little or no basis.

Thus : C.R.B. believed that B.S. 968 steel, if capable of passing the additional tests, could and would be satisfactorily welded by J. & W.

K.S.B.D. believed that C.R.B. would not have included B.S. 968 as a permissible material unless satisfied that it was suitable.

Utah was assured by J. & W. that it could do the job and, not being required by the specification to submit evidence of J. & W.'s experience and not itself realizing the full implication of working with high-tensile steel, accepted these assurances without making adequate enquiries.

J. & W.'s officers were told by B.H.P. that the steel was weldable; did not realize that for "weldable" they should have read "weldable by firms that have reorganized themselves to a new level of technical competence"; thought that C.R.B. would not have included B.S. 968 without due consideration; and finally, knew that C.R.B. would be inspecting the fabrication procedure and the finished work and expected that this would suffice to ensure a satisfactory job.

All these parties must share the blame for embarking on a project for which they were not fully competent. Only Ferris and Murex played their parts properly.

We believe that C.R.B. rather expected that the successful tenderer would have been an overseas contractor, experienced in the design and construction of the particular type of bridge offered, and, in that event, C.R.B.'s own limited experience might have been of less significance.

The first welded high-tensile steel bridge built of Australian steel by a Victorian firm should have been a much smaller structure on which all concerned could have served an apprenticeship in a new craft.

2.5. The Specifications.

2.5.1. General description.

The specifications fall into four main divisions, as follows:—

Division 1—Conditions of tendering and conditions of contract.

Division 2—Materials.

Division 3—Design.

Division 4—Construction.

The conditions prescribed in Division 1 required each tenderer to prepare a design conforming generally to the leading dimensions which had been fixed by the C.R.B., and to submit a Schedule of Rates giving the unit cost quoted for each item of work. The product of the estimated quantity of each item and the unit cost gave the estimated cost of each item and hence, by summation, the estimated cost of the whole work. The actual cost of the work was similarly obtained from the product of the actual quantity of each item and the unit cost quoted in the Schedule of Rates.

Division 2 specified in considerable detail the properties of the materials which might be used in the bridge and the tests which samples of those materials had to pass before being accepted. As is usual in such documents, much use was made of standard specifications issued by such bodies as the Standards Association of Australia, the British Standards Institution and the American Society of Testing Materials.

According to clause 3–1–5. (*a*), the superstructure might be constructed of reinforced concrete, pre-stressed concrete, riveted or welded steel, structural aluminium alloy or combinations of any of these. Division 2 accordingly provided specifications for most of the materials likely to be required for structures of the above types although, as it turned out, the specifications for the high-tensile steel which was eventually used were mixed up with those for mild steel. We are concerned throughout this Report with events which may have stemmed from this unfortunate lack of precision.

It may be remarked that no specification at all was provided for aluminium alloy. The C.R.B. explained that had the successful contractor offered this material, a supplementary specification would have been issued in accordance with clause 1–3–9.

Division 3 of the specification dealt with design and in it were laid down the loads which the bridge was to be capable of carrying, the permissible working stresses in the various materials, the permitted limiting dimensions of various components and, generally, the great many conditions to which the designs had to conform. The total effect of all these clauses was to leave the designer very little freedom of manœuvre for the exercise of his own judgment. Clauses 1–3–4 to 1–3–8 inclusive laid down the arrangements by which the C.R.B. were to check the drawings and computations; in practice these arrangements were supplemented by design conferences at which the C.R.B. indicated the corrections that were to be made to the drawings. Exhibit 157 is the file of the minutes of these

conferences, 119 in number, and it is clear that, while it may be going too far to describe the C.R.B. as "the real designers of the bridge", the C.R.B. certainly approved every calculation, drawing and detail, and required to be changed anything of which it did not approve, or which did not conform to the specifications.

Division 4 contained the specification clauses which regulated workmanship and construction procedure; in particular, clauses 4–6–53 to 4–6–67 covered welding and inspection. Some of these clauses, notably 4–6–55 (Control of distortion and shrinkage stresses), 4–6–56 (Pre-heating), 4–6–60 (Quality of welds), 4–6–61 (Corrections), and 4–6–63 (Inspection, general) became the subject of much disputation between J. & W. and the C.R.B. They are discussed in Section 2.5.4. below.

There were in the specifications a number of clauses the existence of which seem to have been overlooked or ignored by the parties, or compliance with which was waived. Some examples are:

Clause 2–1–1. (Particularly as it defines the word "purchases").

Clause 2–3–9. (Drillings for analysis).

Clause 2–3–24. (Despatch of material).

2.5.2. The design specifications.

Division 3 of the specifications prescribed the way in which the designer was to set about his task and the limitations within which he had to work. It is necessary to enquire whether this specification was so restrictive that designers were unduly circumscribed and also whether any of the clauses were actually harmful in any way.

As to the former, Hardcastle stated in evidence that he did not feel himself restricted by the specification but he also stated that it was much more detailed than other such specifications to which he had worked and that it "certainly controlled our approach to design".

On the other hand, there were two respects in which Professor A. J. Francis, who assisted John Holland and Co. with their design, disagreed with the terms of the specification. The first of these is of sufficient interest to merit a brief discussion. Clause 3–4–10 stated that "in designing piles to resist lateral loading, and for design as a column, no lateral support from surrounding materials shall be taken into account where these materials are, filling, silt, or clay". It would be not unfair to describe this clause as being completely safe but somewhat conservative even in view of the very poor foundation material adjacent to the Yarra River.

Francis, relying on experimental and other evidence that need not concern us, attempted to take advantage of clause 3–1–10, which stated that "a rational analysis based on a theory acceptable to the Board . . . will be considered as compliance with the specifications", and designed a piled foundation which took some account of lateral support from the silt and clay of the river bed.

Utah, on the other hand, made enquiries from the C.R.B. as to how tenders which departed from the specifications would be received, and was told, in substance, that they would not be entertained. Accordingly, it was decided as a matter of policy to adhere strictly to the specifications and to submit a design including Benoto cylinders for the foundations. The tender letter of 29th January, 1957, however, includes these words:—

> "If the C.R.B. can furnish further information concerning lateral support to the piles and also detailed properties of the silt, its location and extent at each of the pier locations, then based on such further information, on the assumption that lateral support of piles can be assumed above the gravel layer it is believed that a foundation using piles could be designed, which, if acceptable to the C.R.B., could result in a substantial money saving".

The C.R.B. did not pursue this suggestion but accepted the tender based on Benoto cylinders which were eventually successfully used. Utah and the designers were, therefore, more accurate than John Holland and Co. in its assessment of the C.R.B.'s views on piles; by tendering on the basis of Benoto cylinders, and contenting themselves with merely drawing attention to the economic advantages of piles, they went a long way to securing the

contract without doing violence to their engineering conscience; while the intended use of piles was one of the reasons given for the rejection of John Holland's tender in spite of its lower price.

We do not blame the C.R.B. for its attitude. As a public authority, responsible for public safety, it was perfectly entitled to take this decision. We simply point out that other responsible and experienced engineers held a different view. Such a conflict of judgment is inevitable in any engineering project.

The second matter in which Francis held different views from the C.R.B. was in relation to the thickness of flanges briefly discussed previously in Section 1.1.3. Although there is a rather unfortunate confusion with clauses 3–4–13 and 3–6–14 of the specification, clause 3–10–6, where one reads that " no plates greater than 1 inch in thickness shall be used in welded construction ", is unambiguous. This clause led straight to the use of cover plates.

Here again we are involved in a question of judgment. The necessary thickness of the flange plates in the centre of the span can be achieved either by having a single plate of, say, $1\frac{1}{2}$-in. thickness, or by welding a cover plate on to the main flange plate. In this case the combined thickness would be somewhat less than $1\frac{1}{2}$ inches because the thinner plates in B.S. 968 can be used at a somewhat higher working stress than the thicker.

Both methods have their advantages and disadvantages. The C.R.B., relying on Ferris's advice, reasoned as follows :—" The thicker the plate the greater is the risk of brittle fracture because of the inherent properties of steel. It is therefore better to use two or three thinner plates welded together and to rely on our tests to ensure good steel and sound welding. Cover plates are well recognized and widely used devices and, if carried far enough towards the end of the girder to guard against fatigue, provide the best solution ".

The opposite view can be stated thus :—" Thick plates in the centre of the girder will be joined to thinner plates towards the ends by butt welds which are inherently better than fillet welds because they are more easily X-rayed, and are not so liable to residual stresses arising from the welding process nor to stress concentrations under applied loads. Provided we take care to avoid notches and other discontinuities which will produce local stress concentrations we can reduce the risk of brittle fracture to a reasonable level. Moreover, we will eliminate long runs of fillet welding and, in particular, avoid the termination of the cover plate which, although often used, is not a good design feature ".

Expert opinion is divided on which is the better view to adopt and the C.R.B. cannot be blamed for preferring one to the other. We simply point out that Francis disagreed, but we must not fall into the error of saying that the bridge would still be standing if it had been built to his design rather than to Hardcastle's; we have no means of knowing how the thicker steel would have turned out in actual practice nor how it would have fared in the hands of the fabricators.

It must be pointed out, though, that the specification both required satisfactory Izod results as a guarantee of adequate notch-toughness and also limited the thickness of the steel for the same purpose. This left the designer with no scope to apply his own mind to the risk of brittle fracture and the best means of avoiding it.

In conclusion, we think it can be said that opinion is swinging away from cover plates towards the use of thicker plates and especially towards the use of special steels and techniques in critical regions. Thus Messrs. Mott, Hay and Anderson, when designing the Runcorn-Widnes Bridge (See Ex. 93) in 1956, used plates in B.S. 968 $1\frac{3}{4}$ inches thick with butt welds, but arranged for the completed girders to be subjected to a stress-relieving heat treatment. They seem to have got the best of both worlds for their bridge is still standing, at the time of writing, after one of the most severe winters on record.

2.5.3. High-tensile steel.

The clarity of the specification for the steel left a great deal to be desired. Thus clause 2–5–1, high-tensile structural steel of weldable quality, provided that the steel should conform to the requirements of B.S. 968: 1941, that the steel should be made by the basic open-hearth method, and that the War Emergency Revision of 1943 should be deleted. Testing was to be carried out in accordance with clauses 2–3–6 to 2–3–16.

Reference to these clauses which, incidentally, are under the heading " Mild Steel ", shows that several tests not contemplated by B.S. 968 were called for. The most important of these were the Izod and weldability tests which were intended to protect the bridge from brittle fracture.

In the Izod test a specimen of the steel to be examined is machined so that its cross-section is a square of side 10 m.m. A groove or notch of specified dimensions is cut in one side of the specimen which is then mounted in the anvil of a pendulum impact machine. The pendulum weight is swung so as to hit the specimen which breaks at the notch ; the energy absorbed by the steel in fracturing is measured by the subsequent height of the swing of the pendulum. It is generally considered that this test gives a good indication of the notch toughness of the steel and is thus a fair guide to its liability to brittle fracture.

An alternative test of a similar kind is the Charpy test in which the specimen rests as a beam between two supports instead of projecting as a cantilever from a single anvil as in the Izod test. The Charpy test is replacing the Izod mainly because the temperature of the specimen can be more easily maintained at the desired value.

In clause 2–3–7 the required Izod values are given as 32 ft./lb. at 70° F. and 20 ft./lb. at 32° F.

The weldability tests were quite elaborate and required two plates to be butt-welded together in a certain manner and then cut up into slices, by machine cuts at right angles to the weld, so as to provide a number of specimens each containing a central weld. Four of these specimens had to withstand being bent round a former without cracking ; two were subjected to tensile tests and four were prepared for Izod testing by machining notches ; in two of the Izod specimens the notch was located in the centre of the weld and in the other two in the heat-affected zone (H.A.Z.) of the plate adjacent to the weld.

The Izod specimens were to be tested according to the requirements of clause 2–3–14 (*b*) in which two temperatures, 32° F. and 70° F. were mentioned. For reasons that were never satisfactorily explained, the tests at the lower temperature were never carried out.

The specification leaves much to be desired at this point. Only four Izod specimens were called for, two with a notch in the weld and two with a notch in the H.A.Z. Thus if tests had been conducted at two temperatures there would have been but a single test under each condition. This is not good practice ; whenever possible three tests should be made. It is unfortunate, too, that the lower temperature was the one to have been discarded for this was the more critical from the point of view of brittle fracture.

It can well be argued that if the reason for including these tests had been unequivocally stated in the specification the C.R.B. inspectors, at least, would have been able to approach this task with more understanding.

Some ingenuity is required to discover the results that were to be attained in all these tests for the clauses just referred to were under the general heading of " Mild Steel " and the values quoted presumably referred to that material. However, B.S. 968 gives values of the ultimate tensile stress and yield stress for plates and sections of various thicknesses ; no values for the Izod tests are given, of course, since these tests are not part of that specification.

The final result of all this was to produce a steel specification with many clauses in addition to those of B.S. 968. The intention of the C.R.B. in so doing was naturally, to secure a steel with appropriate qualities, especially in respect of notch toughness, but the confused way in which the specification was actually written and the ambiguous use of the term B.S. 968 undoubtedly contributed to the unfortunate series of misunderstandings, or whatever they were, that surrounded the supply of steel for the bridge.

It is not unusual for the writers of specifications to add clauses to standard specifications and, in some instances, standard specifications include optional clauses that may or may not be insisted upon. We are convinced that the train of events would have

been quite different if the C.R.B. specifications had made no mention of B.S. 968 but had included a full specification, *ab initio*, for the steel that was desired. This could have forced Utah and J. & W. to have negotiated a contract with B.H.P. for the supply of a special steel and many lamentable incidents, which occupy much space in this Report, might not have taken place.

The following table summarizes the main differences between the C.R.B. steel specifications and B.S. 968 :—

C.R.B. Clause.	Title.	B.S. Clause.	Difference.
2–3–7 ..	Impact properties	..	Not in B.S. 968
2–3–8 ..	Weldability	..	Not in B.S. 968, but similar tests are given in B.S. 2549 : 1954 as a test of the electrode
2–3–9 ..	Chemical analysis	9	Clause 2–3–9 more detailed than clause 9
2–3–12 ..	Tensile tests	4	Speed of testing specified in clause 2–3–12
2–3–14 ..	Impact tests	..	Not in B.S. 968
2–3–15 ..	Number of tests	5 and 7	Quite different requirements
2–3–16 ..	Weldability tests	..	Based on B.S. 2549 but significant differences

2.5.4. Welding.

The clauses dealing with welding were carefully drawn with the intention of producing a high standard of workmanship. The difficulties that were actually encountered arose, in the main, from the following :—

(*a*) At the time of writing this specification, Wilson was not fully aware of the complications of welding B.S. 968 steel, and it therefore became necessary to issue supplementary specifications.

(*b*) The nature of the welding process and the defects that can occur are such that the production of an unambiguous document describing what is, and what is not, acceptable is inherently difficult.

(*c*) J. & W. did not expect that the very high standard of welding described in the specification would be insisted on by the C.R.B.

The clauses that gave rise to special difficulty are now briefly discussed.

Clause 4–6–63 includes the following :—

"The welding of structural sections will be inspected by the engineer (i.e., the C.R.B.) during and after fabrication. Methods of inspection which may be used include the following :—

Visual inspection (including the use of penetrant dyes, acid etching and photography).

Magnetic particle inspection.

Radiographic inspection."

J. & W. maintained that previous experience of C.R.B. contracts had led to the expectation that radiographic inspection, if used at all, would be used to a small extent only, and that the amount insisted upon required the welding to be of "pressure vessel standard".

The C.R.B. maintained that the clause meant exactly what it said and it proceeded to set up a team of inspectors whose duty was to inspect every inch of welding using whatever method was thought appropriate. Penetrant dyes were used extensively on fillet welds and radiography mainly on butt welds.

Clause 4–6–60 (*a*) reads: "Weld metal shall be solid throughout except that any small gas pockets and small inclusions of oxide or slag may be over looked if well dispersed and if none exceeds $\frac{1}{16}$ inch in greatest dimension, and if the sum of the greatest dimensions of all such defects in any square inch of weld metal does not exceed $\frac{3}{8}$ inch". The

interpretation of this clause, and the insistence by the C.R.B. on what was regarded by J. & W. as being a standard higher than was implied in the phrase " equal to the best general practice in modern bridge shops " (Clause 4–6–7), led to continual friction. Eventually, a new radiographic standard (Ex. 58) was adopted based on the work of Beaton of the California Division of Highways. Objection was taken to this new standard on the grounds that, as Beaton's work had not been published at the time when the contract was signed, the contractor could not possibly have expected it to be used. Nevertheless, it was used apparently for the remainder of the construction period.

Clause 4–6–61 prescribed the procedures to be used to correct any defects found as a result of the inspection procedure just described. There is no doubt that a large amount of repair work was in fact carried out (See Ex. 103 and 222) and subsequently passed by the C.R.B. The evidence supports the contention of J. & W. that, in spite of the difficulties which were admittedly encountered in welding, every repair demanded by the C.R.B. was carried out to the satisfaction of the C.R.B.

Clause 4–6–56 required care to be " exercised to prevent weld cracking ". " If required by the engineer " the parts to be joined were to be " pre-heated to a temperature of at least 130° F . . . ". Tables 1 and 2 and Appendix D of B.S. 2642 give the relationship between the pre-heat temperature and the welding conditions for rutile and low-hydrogen electrodes. This specification was itself superseded, on the instructions of the C.R.B., by a booklet (Ex. 23) written by B. J. Bradstreet and published by the British Welding Research Association entitled " Arc-Welding Low-Alloys Steels " which described a fairly elaborate procedure for determining the degree of pre-heat and resulted in different temperatures being required from those given by either clause 4–6–56 or B.S. 2642. This topic is discussed further in Section 2.9.4.

The pre-heat temperatures actually used were the subject of argument throughout the hearings of the Commission. In clause 4–6–55 instructions are to be found on the welding procedures to be used to minimize distortion and shrinkage stresses, while clause 4–6–12 (*a*) states that " completed work shall be of the correct dimensions, square and free from twists, bends, and open joints. No member at any time during fabrication shall depart from the straight or from the specified profile by more than ¼ inch in any length of 10 feet ". These requirements were apparently later replaced by those in clause 507 of the American Welding Society's Standard Specification for Welded Highways and Railway Bridges, which are somewhat less severe.

While the J. & W. and C.R.B. witnesses were all unanimous that at least the pre-heat specified by the C.R.B. was always used, it was admitted by the senior officers of J. & W. that they were reluctant to use the full pre-heat required by the B.W.R.A. booklet referred to above. One reason for this was undoubtedly the fact that the greater the preheat the greater the difficulty of controlling distortion. It was never satisfactorily established that the permissible deviations of the finished girders from the nominal dimensions were compatible with the pre-heat required.

2.5.5. General comments on the specifications.

Specification writing is an art whose subtleties are seldom illuminated by the light of day let alone by the blinding searchlight of a prolonged public enquiry.

The writer is, or should be, concerned to describe the materials and workmanship in such terms as will ensure adequate, serviceable quality but not such perfection as would be unduly expensive. It must be confessed that specifications usually err on the side of perfection and call for work of such flawless virtue as no contractor could achieve without going bankrupt. It is then left to resident engineers, clerks of works, and inspectors to temper the written word with the wisdom and experience of long practice.

A good specification, on the other hand, substitutes objective tests of quality, wherever possible, for pious demands for the unattainable so that inspectors have impartial rods with which to measure the work it is their duty to judge.

By this criterion the C.R.B. specifications were, in general, good but the clauses covering steel were scattered about in an unfortunate manner and were not as clear as could have been desired. We have criticized some of these clauses because they were

not free from ambiguities and especially because the ambiguity over the nomenclature of the steel had such unfortunate consequences. However, the C.R.B., while admitting these criticisms, claimed that no reasonable person could really have been misled; any deficiencies in the specification provided a useful excuse for—but could not possibly have caused—the events that led to the failure of the bridge.

Our only criticism of the substance of the steel clauses, as distinct from their form, relates to the Izod value required and the lower temperature specified; these were taken by Ferris from B.S. 1500: 1949 for unfired pressure vessels. We now think that the value of 20 ft./lb. was too low and that the temperature of 32° F. was too high; the lowest temperature recorded in Melbourne is 27° F. (See Sections 2.7.1.2. and 3.4.3.)

Nevertheless we desire to commend very strongly the inclusion of such a clause.

There remains only the question of whether the form of the specification was suitable for this type of contract. We make no firm decision on this point but content ourselves with the remark that the full advantage of a design and construct contract may not be realized unless a *performance* specification is written. We think that such a course would allow the contractor more scope to offer an ingenious solution and would force him to bring his designer more closely into partnership with his construction team. But we concede that the writing of such a specification would pose many problems for the construction authority especially in devising suitable acceptance tests.

With the important reservations mentioned, therefore, we conclude that the C.R.B. specifications were satisfactory.

2.6. Design.

2.6.1. General suitability.

We are agreed that, speaking generally, the design was a good one. It employed a large number of plate girders, which are very commonly used in steel bridge construction, arranged alternatively in cantilever and suspended spans. The details were arranged in such a way as to make the progressive erection of the completed girders a straightforward and systematic process. The girders themselves had clean and simple lines, easy to maintain and attractive in appearance.

The structure as a whole was an elegant addition to the Melbourne scene; it certainly greatly improved the flow of traffic in that part of the city.

Criticism of the design must, therefore, be confined to the use of high-tensile steel and to the details of the design of the tension flanges of the girders. The latter are discussed below. In proceeding as they did on both these features the designers, who were represented at the Enquiry by Hardcastle, were supported by the confirmatory opinions of many other engineers and they certainly cannot be held solely, or even mainly, responsible for what went wrong.

2.6.2. Design of the tension flanges, of the girders.

It has already been indicated (Section 2.5.2.) that the method of securing the necessary thickness of the flange in the centre of a girder by adding one or more cover plates is very common practice. This is a feature which has been carried over into welded-bridge practice from long experience with riveting but the continuity between the adjacent plates, which is an inherent and, from some points of view, an advantageous feature of welding introduces complications that do not occur with riveting.

It is customary, for reasons of economy, for the cover plates to extend only over such regions of the flanges as need to be strengthened. The points of termination have to be settled and, in this case, clause 3–10–29 (*d*) and Table 2 of Division 3 of the specifications laid down the procedure. The object of this procedure was, very properly, to ensure that the ends of the cover plates were located in regions where the general stress level was low enough for the danger of fatigue to be discounted.

We understand, however, that the formulae given in Table 2 were modified after it had been decided to use high-tensile steel. This would imply that the fatigue strength of this steel is higher than that of mild steel in the same proportion as the ultimate tensile strength is higher. We do not think that this is so and, in our opinion, it would have been wiser to use the same fatigue formulae for high tensile as for mild steel. The result of this would have been that the cover plates would have been longer and their termination nearer the ends of the girders; we do not think, however, that this change, although beneficial, would have significantly altered the subsequent train of events.

The shape of the cover plate end had also to be decided and here a detail was used that has been successfully used in thousands of instances. It is shown in Fig. 11. In its report (Ex. 1) the Committee of Investigation criticized this detail, saying: "This is known to produce high local concentration of stress and is considered an undesirable feature". The Committee can hardly have been unanimous on this matter, for Exhibit 36 (U.S. Bureau of Public Roads drawing SB-2-53 showing standard Composite I-beam Bridges) shows a cover plate detail almost identical with that used in Kings Bridge and is signed by Mr. E. L. Erickson, a member of the Committee.

The stress situation in the main flange plate adjacent to the end of the cover plate is very complicated and is affected by several circumstances. These are discussed below in some detail, but it should first be pointed out that the attention of investigators in the past has mainly been concentrated on the magnitude and significance of the stresses induced by the superimposed loads on the structure. The residual stresses left in the structure by the processes of manufacture have not received anything like the same attention, partly because they are far more difficult to investigate satisfactorily, but also because they are usually considered to be of less importance. This view can only be justified if it is certain that the steel is sufficiently ductile to allow high local stresses to be dispersed into surrounding regions by local yielding. The various sources of stress are now discussed.

2.6.2.1. *Residual stresses resulting from previous welds.*

When two pieces of metal are welded together it is inevitable that, on cooling, tensile stresses will be "locked up" in the weldment at some points and compressive stresses at others.

If a single plate is uniformly heated and allowed to cool the thermal expansion and contraction can take place freely and there are no residual stresses. But if, while it is hot the plate is firmly attached by welding to another plate, the shrinking which would otherwise occur on cooling is prevented to some extent and a state of stress is produced. In a weldment the weld metal itself is also involved for, as soon as it solidifies, it attempts to shrink but, being firmly attached to the adjacent plates, it too has stresses induced in it. The details of what happens depend very much on the temperature distribution throughout the joint and the rate at which it loses heat particularly as it approaches normal temperature.

The above remarks apply both to mild and to high-tensile steel. Internal stresses are also caused by the volumetric expansion, which has to be accommodated by the cooling mass, associated with the carbon in the steel; at high temperatures the iron and carbon are combined in such a way as to form austenite which changes, at lower temperatures and with an increase of volume, to ferrite plus pearlite.

The higher the carbon and manganese content the lower is the temperature of this change and the harder is the resulting product.

It follows from the above that as one weld succeeds another, during the fabrication of a structure, complex patterns of residual stress are superimposed.

The main runs of welding in Kings Bridge were the fillet welds connecting the webs to the flanges and the cover plates to the flanges. It is to be expected that the final stress situation after fabrication depends on the order in which these main welds and, naturally, all the other welds, were performed. We therefore attempted to find out whether the cracks at the ends of the cover plates were associated with the order in which the various welds were performed and, while the evidence is not conclusive, it would seem that the worst results were obtained when the cover plate ends were welded last, as was the case with the four girders of the failed span. (See also Secs. 2.9.6.1. and 2.9.6.7.)

2.6.2.2. Residual stresses resulting from the progressive completion of a single weld.

It will be apparent from the foregoing that the final state of stress is liable to be affected by the way in which a single weld is built up. The Interim Code for Manual Metallic Arc Welding in Building Construction, published by the Standards Association of Australia as S.A.A. Int. 352, has this to say : " The direction of the general progression in welding on a member shall be from points where the parts are relatively fixed in position with respect to each other, toward points where they have a greater relative freedom of movement."

In attaching the cover plates to the flanges the long fillet welds were automatically welded first and the welds along the tapers and across the transverse ends were then done manually. This operation was some times carried out as the last step in attaching the cover plate to the flange but before the web to flange fillets were made, and sometimes as a final operation after the web and flanges had been welded together.

So far as the order of making the taper and transverse manual welds was concerned, much time was spent at the Enquiry in trying to establish what actually happened and in discussing what should have happened. Taking the evidence as a whole we conclude that some welders did one thing and some another and that the records now available do not enable us to say whether there is any correlation between the order of procedure and the subsequent appearance of cracks.

There was no evidence that the transverse end weld was ever completed before the welds down the tapers were begun but experiments by A.I.S. (Ex. 184, Section 7) suggest that this might have been the best way to proceed.

2.6.2.3. Stresses at the ends of the cover plates caused mainly by superimposed loads.

It is to be expected that at the cover plate end there were residual stresses produced, in the manner described above, both by the welding of the cover plate to the flange and also by more remote welding operations.

We now come to the main stresses for which the bridge was designed—those caused by the dead load of the girders and deck and those caused by the incidence of traffic. In addition, mention must be made of the attachment of the concrete deck to the upper flanges of the girders by means of shear connectors. The shrinkage of the concrete, as it hardened and matured, would impose a compressive stress on the upper flange and a corresponding tensile stress on the lower flange. Professor Francis' estimate of the increased tensile stress in the lower flange from this cause was 2,190 lb./sq. in. (Ex. 30).

The stresses caused in the tension flanges by the load effective at any moment can be estimated by conventional bending theory, but this does not take into account the concentration of stress at the ends of the cover plate caused by the sudden change of stiffness of a flange to which a partial length cover plate is attached.

The first report of the Steel Structures Research Committee (H.M.S.O. 1931) contains theoretical and experimental evidence about the stresses in welded joints which, if extended to the cover plate problem, indicates clearly that the stresses in the welds connecting the cover plate to the flange will be different from those given by conventional theory. In particular it is to be expected that the elastic force transmitted by unit length of the end welds will be intensified.

2.6.2.4. Intensification of stresses by the shape of the weld.

The three previous sections have described briefly three sources of stress at the ends of the cover plates. It remains to point out that the actual stress experienced at a particular location depends not only on the general stress level to be expected for one reason or another but also on the exact shape of the weld at the point. Thus a smooth and gradual transition is much to be preferred to one in which there are re-entrant angles which produce an intensification of the stresses. In particular any sharp angle at the toe of the weld, or undercutting or overlapping of the weld or, worst of all, an actual crack will result in a local region of higher stress intensity than the average in the locality.

All these effects can be cumulative and, when this happens in the tensile sense, the local stress at a critical point can rise to a much higher value than the designer's calculations, which generally deal with average stresses only, would lead him to expect.

2.6.2.5. *Triaxial stresses.*

It will be appreciated that, within the thickness of the steel in a weldment, the various effects described above can result in stresses having components in mutually perpendicular directions. Specifically, at the toe of the weld at the end of the cover plate there can be stresses acting parallel to the length of the girder ; stresses perpendicular to the length of the girder and parallel to the plane of the flange ; and stresses perpendicular to the length of the girder and parallel to the plane of the web.

It is known that the behaviour of steel is greatly affected by the magnitude and sense (i.e., tension or compression) of these stresses and that, in particular, the existence of component tensile stresses that are nearly equal in magnitude is conducive to brittle fracture. While the subject is far from being fully explored it now seems certain that a steel which is quite ductile under a uniaxial stress can become brittle if sufficiently large perpendicular stresses are also present. It is also thought that all steels will fail in a brittle manner under sufficiently high triaxial stresses, i.e., when the three perpendicular components are equal.

It also appears that the transition temperature, at which a change from ductile to brittle behaviour takes place, is raised by the action of component tensile stresses perpendicular to the direction of the main stress. This may account for the fact that the transition temperature obtained from Izod or Charpy tests is not always the same as experience of the same material in an actual structure would indicate.

An interesting discussion of these matters is given by Bijlaard in the book " Residual Stresses in Metals and Metal Construction " edited by Osgood (Reinhold, 1954 p. 127).

The phenomena just described make it particularly dangerous to weld across the tension flange of a girder for residual stresses can be set up in three perpendicular directions. Since the main stress (longitudinal) is tensile the tendency to brittle fracture is increased and the transition temperature raised at points where the other component residual stresses are tensile.

A further consideration is that, if a crack should appear, it is likely to be across the flange and so across the line of the main longitudinal tensile stress. A crack of this nature may, in some circumstances, propagate by brittle fracture ; in others the resistance of the beam to fatigue failure will be reduced. It should also be noted that the risk of a crack propagating by brittle fracture is greatly affected by its location. If it is in such a position that it can extend without hindrance right through the structure it is much more dangerous than if extension does not lead to a large release of energy. It will thus be seen that a crack on the outside of a tension flange is more serious, for example, than a crack in the web some distance from the flange.

For these reasons, and also because any undiscovered weld defect in this position is a special source of weakness, some designers prohibit the use of transverse fillet welds at any point on a tension flange ; others permit them only towards the ends of the girder where the bending stress is low. The specifications for Kings Bridge were based on the latter view.

We find it difficult to say whether, at the time of writing the specifications, the C.R.B. should have decided differently but we are of opinion that, on the basis of knowledge now available, transverse fillet welds should be prohibited on tension flanges.

2.6.2.6. *Connection between the dangers of fatigue and of brittle fracture.*

The phenomenon known as the fatigue of metals may be briefly described as the tendency of a metal to fracture under a fluctuating or reversing stress. Its danger arises from the fact that a metal will fail under a smaller load, applied thousands or millions of times, than would be required to make it fail in a single application.

This danger is naturally more acute in machines with reciprocating or rotating parts than in bridges where only the live load, due to traffic, fluctuates ; the dead load, due to the self-weight of the structure, is constant. The extent of the fluctuation of total load depends on the ratio of live to dead load and is usually greater in railway than in road bridges.

At any point in a structure the likelihood of fatigue failure depends on the maximum stress and on the range of stress experienced there ; on the number of fluctuations likely to be experienced during the life of the structure ; and on the inherent resistance of the material to the phenomenon of fatigue. The connection between fatigue and brittle fracture is that both are most likely to occur at points of high stress especially if the inherent stress is intensified by stress raisers such as unfavourable geometry, poorly shaped or defective welds, surface and other defects produced in the steel-making process or notches or cracks. Residual stresses, especially when tensile and in perpendicular directions, seem to add to the risk of both types of failure.

The Kings Bridge specifications took account of the possibility of brittle fracture. They also gave recognition to the susceptibility of the cover plate end to fatigue failure by fixing its position in a region of low bending stress. It was argued at the Enquiry that, because of the above connection between fatigue and brittle fracture, the designer should have been alerted to the danger of the latter occurring at the same point. This may well be so but the C.R.B., whose officers compiled the specifications should have been even more alert. However, Eastick's evidence suggests that he did not realize that the Izod tests had been included to try to guard against brittle fracture.

2.6.2.7. *Consideration of residual stresses by designers.*

Although the exact role played by residual stresses in promoting brittle fracture is still not fully understood there is no doubt that it is a significant one. The question therefore arises whether the designer should take them into account in some way in the course of making his calculations.

It will perhaps be apparent from what has already been said that the precise estimation of the magnitude of residual stresses is really a hopeless task ; the most that can be said is that they will certainly be present in a welded structure and that they may well approach or reach the yield point of the material. On the other hand they are usually, but not always, local in character. Finally it must be said that their effect becomes more serious as the size of the structure increases.

With these facts in mind, therefore, and in the present state of knowledge, the designer proceeds qualitatively rather than quantitatively. He can do his best, by careful detailing, to ensure that the inevitable concentrations of stress are kept to as low a level as possible ; he should ensure that the fabricator adopts satisfactory procedures during welding ; in order that local yielding may disperse high stresses by redistribution, he must ensure that only steels of adequate ductility and notch toughness are used ; and finally he may decide to resort to a stress-relieving heat treatment.

One of the features of the Kings Bridge project that most disturbed us was that the designer was not in close contact with, nor responsible for, the actual fabrication. Hardcastle told us, for example, that he was not aware that some of the steel used did not exhibit a clear yield point. We have no means of knowing how he would have reacted to this knowledge if it had reached him at the time but it is certain that this information would have had more significance for him, whose knowledge of the anatomy of the structure was so intimate, than for anyone else.

In the foregoing we have been mainly concerned with residual stresses from the point of view of their local effect at special points. There remains the question of any effects that are greater in scale ; one such matter did receive very close attention during the construction of the bridge.

It was known that welding the stiffeners to the web plate would have the effect of shortening the web and it was therefore intended that the operation of attaching the stiffeners to the web should be completed before the web to flange welds were made. However, it would not then have been possible to use an automatic welding process for the long web to flange fillets.

Experiments were therefore undertaken to find the magnitude of the web-shortening resulting from the attachment of the stiffeners so that an estimate could be made of the residual stresses likely to be induced if these stiffeners were welded to the girder after the flanges were in position. The results of these experiments were submitted to Professor

Roderick, of Sydney, whose view was that the arrangement would be satisfactory provided that the possibility of buckling could be eliminated from consideration. Accordingly, Hardcastle made some calculations which showed that the danger of buckling was remote and the arrangement was approved and used throughout the fabrication.

It is evident from a perusal of Roderick's report, and of the records of the conference with him, that he mainly considered the possibility of a reduction of the so-called plastic moment of resistance of the girder by the residual stresses. In so doing he implicitly relied on the capacity of the girder to develop a full plastic moment by yielding, as he was entitled to do because of the specification of the steel. He was not asked for an opinion on the possibility of brittle fracture intervening before yielding could take place and did not himself advert to this possibility.

2.6.2.8. *Shape of the cover plate ends.*

Because of their possible influence on the failure of the bridge, the shape of the cover plate ends was much discussed at the Enquiry. There has been much research on this problem, mostly directed to comparing the resistance to fatigue of different arrangements of the cover plate ends ; because of the influence of local stress situations on both fatigue and brittle fracture it can be assumed that a design which is favourable from one point of view will also be favourable from the other.

The results of these investigations are somewhat conflicting ; thus the Welding Handbook, 4th Edition, Volume 1, says that " No practical arrangement or detail has been found which is quite as good as simply cutting the end of the cover plate square without any taper ". On the other hand Munse and Stallmeyer (British Welding Journal, March, 1960), came to exactly the opposite conclusion and found that a design similar to that used in Kings Bridge, but with the transverse weld omitted, was the best under some conditions. However the difference between these designs was not very marked and no detail using cover plates was as satisfactory as the alternative approach of butt-welding the thinner plate to the thicker one and tapering the latter, in width or thickness or both, to match the former at the weld.

It is not to be assumed from this that this last design, by itself, is sufficient to guard against failure. The Duplessis Bridge, in Canada, (Highway Research Abstracts, June, 1951) and an unnamed bridge described by Campus (Osgood : loc. cit. p. 8) both suffered brittle fractures initiating at butt welds connecting tension flange plates of different thicknesses. The steel used in the first of these bridges was, however, of poor quality while weld defects were found in the second case.

Nevertheless, while it seems that designers are now tending to avoid cover plates it cannot be said that the practice is universal nor that the designers of Kings Bridge can be blamed for using them in 1957. As has been said, they have been successfully used in thousands of instances.

2.6.3. The responsibility of the designer for specifiying welding details.

It is not usual for the general structural drawings to show details of the welding procedure which are normally left to the fabricator to work out for himself. In practice the latter prepares from the designer's drawings a series of shop drawings giving the detailed dimensions from which the plates are cut and other information required by the workmen but, again, the number of weld runs, whether downhand or overhead, the gauge and type of electrode and so on, are often left to the welding supervisor to settle on the shop floor.

No doubt this procedure is quite satisfactory on routine jobs in mild steel but it seems to us to be inadequate in some circumstances. It is not difficult to think of structures, even in mild steel, where the order in which the welds are made is important from the designer's point of view. When more difficult materials are being welded the development of satisfactory techniques and the assessment of their significance in the final structure seem to us to require considerations of a somewhat sophisticated nature on which the designer's mind should be brought to bear.

An important advantage of the design and construct type of contract is that there need be no obstacle to full collaboration between the fabricator and the designer in such matters ; the latter can hardly be expected to have a close, detailed knowledge of the metallurgical and other implications of welding techniques but he can well be expected to realize how the final results of those techniques may effect his structure.

We are therefore astonished that it should have been agreed, as stated by Wilson, that " the welding procedures . . . would be determined by the fabricators and there would be no reference on the drawings in that regard." This is to throw away one of the advantages mentioned by Darwin for this type of contract. (Section 2.1.).

We believe that the Kings Bridge project shows that if designers do not take some responsibility for welding procedures then they are taking serious risks. The extent to which this is necessary on a particular job, and the way in which responsibility is best to be exercised are outside the scope of this Report but they merit careful consideration by the engineering profession.

2.7. Steel.

2.7.1. The steel specified.

2.7.1.1. *The justification of including B.S.* 968 : 1941.

Steel to B.S. 968 : 1941 included by the C.R.B. as one of the options for use in the superstructure of the bridge, is known as a low-alloy steel and it is designated as " high-tensile (fusion welding quality) structured steel for bridges, &c.". It was a war emergency specification. The C.R.B. specifications clearly implied that if welded girders were to be included in the construction they could be fabricated either in mild steel or in high-tensile steel. The main advantage in the use of the latter was that higher design stresses could be used because this steel was stronger than mild steel. Consequently, the girders would be lighter, thus reducing the load on the foundations. As the foundations were the cause of much anxiety in the early considerations of the C.R.B., any feature reducing the dead load of the superstructure could from that point of view be considered desirable. Nevertheless, in assessing the economy of high-tensile steel it is necessary to look beyond the mere saving of weight. This material costs more than mild steel per ton and it is more difficult to fabricate. For these reasons we were surprised at Bonwick's statement that its use resulted in a saving of £175,000.

Since all conditions for the satisfactory welding of low-alloy steel were well known and since economies in the dead load on the foundations could be produced thereby it was reasonable for the C.R.B. when drawing the specifications to include B.S. 968 : 1941 steel as an optional material.

However, there are certain implications which follow from this. It was known to the C.R.B. that no major welded structure had been constructed in this material in Australia (See Masterton's article on " Kings Bridge ", Ex. 22). It was not sufficient to know that other fabricators including the Royal Australian Navy had used this steel. If the girders were to be fabricated in Australia it was necessary to know that there was a fabricator with the organizational and technical ability to make the change from mild steel to low-alloy steel construction, and that the steel as made in Australia would have the necessary properties—for it is well known that many manufacturing features enter into the production of a satisfactory steel.

In this connection it must be borne in mind that :—

(*a*) At the stage when the specifications were drawn the C.R.B. did not know that the steel would either be made or fabricated in Australia and as this was a British Standard Specification they not unreasonably assumed its authenticity as a steel suitable for the fabrication of bridges by welding construction.

(*b*) If, on the other hand, as turned out to be the case, the steel production and the fabrication were to be done in Australia, there is no doubt the B.H.P. would have assured C.R.B. that it could make satisfactory steel, and there is equally no doubt there were fabricators in Australia, who,

whilst perhaps not having experience with this particular steel, were organized for work under the exacting standards used for pressure vessel fabrication.

We conclude that the C.R.B. was justified in including steel to B.S. 968 : 1941 as an optional material for constructing the superstructure, provided the specifications ensured that it would be fabricated by a firm having adequate experience with its characteristics. For this purpose there should have been inserted in the specifications a clause providing that where a tenderer proposed the use of B.S. 968 : 1941 or for that matter pre-stressed concrete or aluminium alloy, he should be required to demonstrate his ability and competence to handle such material.

It follows that before accepting a tender based on B.S. 968 : 1941 the C.R.B. should have been satisfied that the above condition relating to the fabricator and the steel supplier could be fulfilled. The C.R.B. neither adequately investigated the properties of B.S. 968 steel as made by B.H.P., nor the ability of the sub-contractor to reorganize along the necessary lines.

In the absence from the specifications of a special clause as suggested, clause 1–2–14, which prohibited the contractor from sub-contracting any part of the work without the prior written consent of the C.R.B., could have been invoked in relation to J. & W. It appears that J. & W. and B.H.P. were accepted as sub-contractors and steelmakers, respectively, on the strength of their general reputation without any special enquiry about their experience in fabricating or manufacturing B.S. 968. The C.R.B. should have required this competence to have been adequately demonstrated.

We arrive, therefore, at one of those many curious situations which have been a feature of this Enquiry. While the C.R.B. was justified in permitting steel to B.S. 968 : 1941 as an option, it was not justified in accepting without further enquiry the tender of Utah including B.S. 968 : 1941.

We are convinced that the C.R.B. failed to make the necessary experiments and enquiries because (with the exception of Wilson) no officer of the Board realized their importance. They accepted the title of the British Standard Specification as implying that the steel could be welded satisfactorily by an established fabricator irrespective of whether he had had experience with low-alloy fabrication or not.

2.7.1.2. *Reasons for modifying B.S.* 968 : 1941.

(*a*) Notched Bar Impact Tests.—Wilson appears to have been the only man in the C.R.B. to have been thoroughly convinced that failure by brittle fracture was a possibility, and in this he was advised and supported by Ferris of D.S.L. To this end he wrote into the original design specifications some safeguards to cover the possibility of failure of the bridge by brittle fracture, applying these features of the specifications with equal force to both mild steel and B.S. 968, if welding construction was to be used.

To cover this possibility he introduced into his specification two features :—

(i) The thickness of plates or sections was limited to a maximum of 1 inch.

(ii) An Izod notched bar impact test to be carried out at two temperatures, namely, 32° F. and 70° F.

It was said that the temperature of 32° F. was chosen because it was the lowest operating temperature to which the steel work of the superstructure would be exposed under Melbourne climatic conditions. However, enquiry at the Meteorological Bureau would have elicited the information that the minimum temperature ever recorded in Melbourne was 27° F. and engineering caution would have reduced this to 22° F. Indeed at another part of the specification this was done : Clause 3–6–46 requires that the provision for expansion and contraction of the girders should cover the range of temperature 20° F. to 120° F. It is obvious that 32° F. was chosen because it is easily reproduced, being the melting point of ice. To understand the significance of this requirement needs a brief description of the phenomenon of brittle fracture. It should be understood that, as mentioned above, this phenomenon is associated with mild steel as with low-alloy steel, so that in what follows no distinction is made.

When steel is subjected to load it deforms : at low stresses the amount of deformation is small and, if the load is removed, the deformation disappears. At higher stresses the deformation is more marked and is permanent in the sense that a high proportion of it remains even after the load is removed. The transition from one stage to the other takes place gradually in some steels while in others it is characterized by "yielding" when a comparatively large amount of deformation takes place under nearly constant stress. Steels in which a large amount of deformation takes place before fracture are ductile.

The presence of notches creates an entirely different situation, and in this connection a notch may be merely a sudden change of section associated with a design feature, a defect in the plate resulting from steel works practice, a crack resulting from defective welding practice or from flame cutting a plate to dimensions, an under cut in a weld, a nick caused by a sharp blow in handling and any similar sharp change of contour on the surface, or even a sudden change in physical properties such as occurs in the immediate vicinity of a weld.

In such cases, under tensile stress, a fracture is liable to start at the apex of the notch, and once started it may be either arrested or propagated according to the properties of the steel. The property of arresting the development of a crack is known as notch ductility. A notch ductile steel will arrest a crack already started ; a notch brittle steel will allow the crack to propagate with substantially no deformation at low stress and the steel fractures as though it were a piece of pottery.

Notch ductility depends on many features associated with the chemical composition and manufacturing conditions of the steel, the sharpness of the notch, the speed with which load is applied and the temperature of the steel when the stress is applied.

It will be seen therefore that to ensure notch ductility in any plate of steel it is essential to test it to check its behaviour to stress in the presence of a notch, and to do this at different temperatures. It is found that steels of the type considered change regularly from a notch brittle state at low temperatures to a notch ductile state at higher temperatures. This is known as the transition range and this determines, in so far as it can be determined, the response to stress of steel containing notches of the type described. For any given quality of steel such as mild steel or low-alloy steel the transition range may vary according to the steel manufacturing conditions, so that it by no means follows that steel of a given chemical composition, will have a defined degree of notch ductility at a given temperature. For this reason, it is essential to apply a test and the one most favoured is a Charpy or an Izod test which has already been described.

The results obtained are a valuable guide to the suitability of the steel, but its actual transition characteristics in a given structure may not be identical with those shown under the simple conditions of the test.

In order to determine the transition characteristics of a steel a number of notched bar test pieces is required. These are then brought individually to a given temperature in the range to be investigated from the lowest at which the steel is to be used up to about 220° F. The specimens are broken in the Charpy or Izod machine at the temperature to which they have been brought, and the energy absorbed in fracturing is noted.

The C.R.B. should have specified and insisted upon the carrying out of a complete series of tests of the kind described until it became familiar with the properties of the steel, instead of specifying two temperatures only, 32° F. and 70° F.

All low-carbon steels are subjected to this phenomenon of changing with temperature from a ductile to a brittle state.

There is, however, another source of embrittlement of low-carbon steels. This is the phenomenon of strain ageing. It is associated with the presence of nitrogen, and is a characteristic of a given heat of steel and independent of the ambient temperature. This brittleness is generated when susceptible steel is subjected to a slight deformation followed by heating to a low temperature (around 250° C.). Since all flange plates were straightened by rolling through staggered rolls, and were heated prior to welding and during

welding, a susceptible steel might develop strain embrittlement and be susceptible to brittle fracture irrespective of the ambient temperature. It follows that the notch ductility of the steel as supplied by the steelmaker may deteriorate during fabrication and there is nothing a fabricator can do to prevent this change. It is inherent in certain heats of steel.

This kind of embrittlement was not envisaged by the C.R.B. and no test to cover this feature was written into the specification. Apparently also the steelmaker did not advert to this possibility, for it is possible to " fix " residual nitrogen by small additions to the steel in the ladle. It does not then cause strain ageing.

(*b*) The Weldability Test.—A further materials test included in the C.R.B. specification was a weldability test intended to ensure that the properties of the steel as supplied were maintained during the welding operation, and also to test the properties of the weld metal deposited from the electrode. Again, this test was to be applied whatever the steel used, i.e., it was not restricted to B.S. 968. It was a complicated test involving a lot of machining, and it was probably this feature which gave rise to the impression that excessive testing was involved in the contract, an impression which ultimately led to a relaxation of testing about the middle of 1959.

This series of weldability tests included Izod tests (*a*) on the weld metal, and (*b*) on the heat-affected zone of the weld. Two tests were to be made on each, and at p. 83, book 1 of the specification it is stated (by implication) that they should be made at 32° F. and 70° F. However, in fact, both were tested at " ambient " temperature, and none at 32° F.

SUMMARY.

In view of the unknown behaviour of B.S. 968 : 1941, steel in conditions leading to brittle fracture :—

(*a*) It was reasonable and right to specify a notched bar test at the lowest temperature to which the bridge would be subjected.

The question as to whether the value chosen for the Izod impact value was the right one is discussed elsewhere (Section 3.4.3.).

(*b*) As the weldability tests were designed to test both the weld metal and the heat-affected zone of the parent plate these tests also were reasonable and should have been carried out on each individual heat of steel.

(*c*) In view of the very limited experimental work which had been done on the properties of this steel by either C.R.B. or J. & W. before fabrication commenced, it would have been advisable for the preparation of the test weld to have been done by one of J. & W.'s own welders. This would have given them a better knowledge of the welding characteristics of the steel. Instead, the preparation of the test welds was given to the most expert welders of Murex, which was quite permissible under the C.R.B. specifications but J. & W. learned nothing from these regarding welding technique.

(*d*) In view of the ultimate failure of the bridge by brittle fracture there can be no doubt that, in the light of present knowledge, the additional tests imposed by the specification were justified, though in certain respects inadequate.

(*e*) It must be realized that even the best specification is useless unless its conditions are complied with. This necessarily involves carrying out the procedures specified and subjecting the results to intelligent scrutiny. This was not done.

2.7.2. The steel actually supplied.

In Section 2.3.4.1., we discussed fully the matter of J. & W. having ordered steel from B.H.P. to comply with B.S. 968 : 1941, simply, while having contracted through Utah to fabricate in steel complying with the extra clauses of the C.R.B. specifications.

It is sufficient to say here, that because of the two different standards created, it is necessary to consider the steel *vis-à-vis* the two specifications.

2.7.2.1. *Did the steel supplied by B.H.P. meet the B.S.* 968 : 1941 *specification?*

The evidence on which this can be answered is partly in Exhibit 72, a compilation of acceptance tests by the C.R.B.

Without going into detail it can be said that, from a commercial point of view the steel supplied was in our view largely in accordance with B.S. 968 : 1941 in so far as tests were carried out.

This statement must be qualified in several ways :—

(*a*) The specifications call for extra tensile tests when more than one thickness is rolled from a heat. This was not always done.

(*b*) Similarly, the number of bend tests, and the type, whether longitudinal or transverse was not always as specified.

(*c*) On the original B.H.P. certificate showing heat analysis no mention was made, in general, of the chromium content. The B.S. 968 specification limits manganese to 1·8 per cent. and the combined manganese and chromium content to 2·0 per cent. As the manganese was frequently near the upper limit, this feature of the specification could not be checked from the certificate unless the chromium content was guaranteed less than 0·2 per cent.

So long as the steel was made by the open-hearth process, the only chromium present was about 0·05 per cent.—a residual from the steel-making conditions. However, Ralston claimed that B.H.P. was justified in leaving up to 0·35 per cent. chromium as residual. If this were so then it was definitely necessary to report chromium on the certificate in order to check against the specification. In actual fact, when in February, 1959, B.H.P. (Port Kembla) switched production from open hearth to electric furnace, it deliberately added 0·25 per cent. chromium and still did not report this on the test certificate.

Even, however, with this addition, the chemical analysis taken from the ladle test ingot which purports to give as nearly as possible the average composition of the heat was, by and large, within the specified limits, though frequently at the maximum permissible limit.

It must be borne in mind that the fabricator has to deal with steel as it is, and not as it might be or as it purports to be on an analysis certificate. In other words either he should be able to accept the certificate as giving the true composition of the steel plates he receives, or he should do some systematic checking. In many cases when plates were checked (Table 3, Section 2.9.6.4.) the results were found to differ from the ladle analysis.

There are well-known reasons why a difference may occur between the " ladle analysis " and the " plate analysis ". These are all controllable metallurgically. Steelmakers vary in their attention to detailed practice, and this constitutes part of the " art " of steel-making.

Briefly there are three sources of variation of composition throughout a given mass of steel. The first relates to the size of charge. Open-hearth heats may be several hundred tons. Towards the end of the process finishing additions of various alloys are made to adjust the carbon and manganese. There is a sheer physical difficulty in getting such a mass of steel uniform.

At Port Kembla the open-hearth charges were of about 250 tons and this was tapped into two ladles of 125 tons. The electric furnace heats, however, were only 17 tons and did not, or should not, present the same difficulty, i.e., they would be expected to be more uniform on tapping.

The second source of variation is concerned with the ladle. Sometimes final additions of alloys are made and the churning action of the molten metal is relied on to mix the whole uniformly.

A.I.S. is in accord with these views for in Exhibit 184, section 1, it is stated " the electric furnace at 17 tons presented a much simpler operation and the advantage of being able to remove slag and re-form slag of controlled composition and conditions, made specification control more regular ".

The third source of variable composition is the natural phenomenon of ingot segregation. The molten steel is teemed from the ladle into ingot moulds. The ingots solidify from the outside and the concentration of such elements as carbon, manganese, sulphur and phosphorus increases towards the centre and top of the ingot. By suitable practice this segregation can be localized and can be removed by cropping in the rolling mill. In this way ingot segregation differs from ladle and furnace segregation. It is regular and can be removed in normal mill practice. The other two would give steel which varies from ingot to ingot and the chance of the test ingot of about 2 lb. being representative of the many tons of metal is remote if the charge is not uniform.

Heats 55 and 56 have been the subject of exhaustive checking during this Enquiry and it has been shown definitely by all the laboratories concerned (D.S.L., McPherson's, and A.I.S.) that there were differences between the ladle analysis and the plate analysis taken from the failed girders. This is not the place for a detailed consideration of this problem. Sufficient to say that the following order of variation has been found by D.S.L. & McPherson's :—

Analyses.

Heat No.	Type	Ladle		Plate (max.)		Difference	
		C.	Mn.	C.	Mn.	C.	Mn.
38	O.H.	·22	1·56	·25	1·67	·03	·11
40	,,	·23	1·75	·26	2·03	·03	·28
55	E.F.	·21	1·70	·28	1·80	·07	·10
56	,,	·23	1·58	·27	1·75	·04	·17

It will be noticed that the difference between ladle and plate analysis, as far as carbon is concerned, is rather greater with the two electric furnace heats than with the two open-hearth heats. The differences are not great except with heat 55 and would not have been serious if the ladle analysis had not already been at the maximum permissible limits of the specification. The regularity with which high carbon and manganese have been found in heats 55 and 56 suggests ladle segregation rather than ingot segregation.

The significance of the chemical composition is that it controls the mechanical properties (yield stress, maximum stress, ductility as measured by elongation in the tensile test and notch ductility) and the weldability. In general, the higher the content of carbon, manganese and chromium the stronger and the less ductile is the steel ; the higher the carbon the less notch ductile and the less easily weldable is the steel.

2.7.2.2. *Submitted heats.*

There was a great deal of discussion during the Enquiry relating to a group of six electric furnace and two open-hearth furnace heats (all fully killed) which were said to be slightly high in tensile strength. These heats were referred to as " submitted heats " and are further discussed in Sections 2.7.2.3, 2.10.3.2 and 3.4.1. It was disclosed at the Enquiry that these heats, including the first heats made in the electric furnace, had been made with the deliberate addition of 0·25 per cent. chromium. This was not disclosed on the analysis certificates. After careful consideration of all the evidence we have reached the conclusion that this addition was not, per se, a factor causing the failure of the W.14 span, but in conjunction with high carbon and manganese added to the difficulties of fabrication.

However, two of the heats (Nos. 55 and 56) were incorporated in the tension flanges of the group of girders which collapsed on July 10th, 1962.

Before following this matter further, it is necessary to look at the steel supplied from the point of view of the C.R.B. specifications.

2.7.2.3. *Did the steel meet the C.R.B. specifications?*

As has been pointed out, the C.R.B. design engineer, Wilson, wrote some additional clauses into the specifications covering Izod impact tests and weldability tests thereby modifying the B.S. 968: 1941. The Izod tests were included to protect the bridge from brittle fracture.

One of the factors which affects notch ductility is the thickness of the plates. Consequently, the specifications called for Izod tests at two temperatures on each thickness of plate from each heat of steel. The records, however, disclose that when there was more than one thickness of plate in a given heat only one thickness was subjected to Izod tests and there is nothing to indicate to which thickness the actual tests were applied.

It was a term of the C.R.B. contract with Utah (See Clause 2–3–24) that material should not be despatched from the maker's works or from stock until Utah had certified that it complied with the requirements of the C.R.B. specification. In fact, during the Enquiry, no party has adverted to this clause and it must be presumed that no attempt was made by the C.R.B. to enforce it. J. & W. looked upon C.R.B. as the authority to give the final approval, but did not hesitate to fabricate steel before this approval had been given. Utah took no part in the assessment of tests, but merely passed on the certificate from J. & W. to C.R.B.

Of the 58 open-hearth heats supplied, 54 were tested, not in accordance with the strict requirements of the C.R.B. specifications, but to the extent of the actual requirements of the C.R.B. They all passed the Izod test at 32° F., most of them easily complying. Curiously some of them (Heats 4, 9, 20, 25, 26, 30 and 31) failed at 70° F., but on retesting (not strictly according to the specifications) were passed. Most of the heats passed the weldability tests at first attempt and all on retesting.

It has been very evident during the Enquiry that those concerned with the Izod testing programme did not understand its significance and went through the motions merely to comply with the specifications. This particularly applies to Scarlett (J. & W. and Utah), Eastick and Jackson (C.R.B.). This attitude was engendered by a defect in internal communication in the C.R.B., for apparently Wilson ceased to be associated with this aspect once fabrication started, and was never aware that the conditions of the specifications had not been completely fulfilled. He does not appear to have passed on his enthusiasm for the additional tests to those subsequently responsible for carrying them out. The controlling personnel of J. & W. had no interest in testing of any kind, and B.H.P. had persuaded them that in any case the Izod tests were futile. Utah personnel simply had no interest in the quality of material being fabricated for the contract. "Why", they said in effect, "do something which C.R.B. is sure to do?"

When it became known to Eastick and Jackson that the remainder of the steel would be supplied from relatively small electric furnace heats of 17 tons instead of from 125-ton open-hearth heats, they decided to test heats selected at random.

In actual fact, there was little change in the position from this aspect, for from the 58 open-hearth heats only 1,200 tons was delivered to J. & W. This is an average of approximately 20 tons per heat and is not much larger than the average of 15 tons per heat from the electric furnaces.

The total number of heats actually supplied was, however, far greater than had been originally anticipated by J. & W. When the order was being discussed with B.H.P., J. & W. understood that it would have to accept quantities from the one heat of not less than 150 tons at a time, and J. & W. placed its orders, with insignificant exceptions, accordingly. If the steel had been supplied in 150-ton lots there would only have been fifteen or so heats involved instead of the 110 or so actually used in the bridge. The testing programme was correspondingly enlarged. In addition the amount of retesting required, especially in the electric furnace heats, turned what would have been a quite reasonable testing programme—even if it had been carried out completely in accordance with the C.R.B. specifications—into a very extensive one.

By contrast with the steel from open-hearth heats the over-all picture of the steel from the electric furnace heats is not good. Because of the changed attitude of the C.R.B. to testing they were not tested as thoroughly as the open-hearth heats. Weldability tests

were carried out on 29 heats and of these 26 failed in one respect or another, but most of them passed on a first retest. This was a much higher proportion of failure than had occurred in the weldability tests on the open-hearth heats, namely 18 out of 58.

This is an indication that the electric furnace steel produced was inferior to the open-hearth steel. One of the reasons given for changing to electric furnace steel—but which we are by no means convinced was the compelling reason—was that the fully-killed steel produced in the electric furnace would be of better quality and would aid J. & W. to meet the more exacting specifications of C.R.B.

From the time that random testing was adopted the story of acceptance testing is a sorry one, and we feel no good purpose would be served by analysing the reported tests in detail. Several of the heats failed in the weldability Izod test and were never retested (Heats 49, 50, 51, 79 and 80). Several failed, were retested and failed again. In fact, it was admitted by Scarlett that testing was continued until satisfactory values were obtained (See heats 54, 55, 70, 74 in Ex. 72).

The climax of the testing story comes with the group of 8 "submitted heats" (See Sections 2.7.2.2 and 3.4.1). The results of the first tests on these heats were so bad that J. & W. and C.R.B. should have been not only alerted but alarmed. Regrettably, these heats were treated as part of the general run. No plate Izods were carried out, and the weldability tests were first made four months after the submission. Of the 8 heats, 5 failed in the tests, 4 of them badly. Heats 52, 53, 54 and 55 failed on the first retest but were again retested,

Heat 55 still failed, so was tested a fourth time. Almost a year elapsed before these tests were completed, but although in September, 1959, these four heats were still showing "fail" on retest, and should in fact have been rejected, J. & W. wrote to B.H.P. requesting delivery.

A more callous disregard of the value of acceptance tests it would be difficult to imagine, and the responsibility for this must rest equally with J. & W., Utah, and C.R.B., for officers of all these organizations were in control of material inspection (Scarlett for J. & W. and Utah, Eastick and Jackson for C.R.B.).

There is, however, an interesting fact to which we attach some significance. The flange in girder W.14–2 which first fractured (some months before the final failure) was made from heat 56 (if the records are reliable), which passed the weldability tests at the first attempt. It was not subjected to plate Izod tests at the time, but in tests since the failure has given quite variable results.

2.7.3. The steel as found in the failed girders.

Since the collapse of the W.14 span of the west elevated carriageway, many samples have been taken for chemical analysis, and for various physical tests including Izod tests. These are set out in some detail in Sections 3.4.1. and 3.4.2. and only brief mention will be made at this stage.

The records show that the tension flanges (bottom) of all four girders at the location of the fracture were constructed from steel from heats 55 and 56 (electric furnace—"submitted heats"). The webs of the girders at the same location were recorded as being made from heat 40 (open hearth) though there is internal evidence that in fact the web of girder W.14–1 was made from heat 38 (open hearth).

Chemical analyses have shown that the carbon and manganese contents of much of the steel from heats 40, 55, and 56 were outside specification—ranging as high as 0·28 per cent. carbon and (heat 40) 2.0 per cent. manganese (See Exs. 155, 184, and 194). The maximum allowed by specification is 0·23 per cent. carbon and 1·8 manganese. The manganese plus chromium figure is, however, close to or above the maximum permissible in heats 55 and 56 because of the presence of 0·25 per cent. chromium (intentionally added), (See Ex. 170).

Much has been made by B.H.P. of a table of tolerances sponsored by the American Iron and Steel Institute. From this it is claimed that for steel of this quality tolerances over the maximum specified carbon of ·04 per cent. and of manganese of ·05 per cent. were permissible, and acceptable as grounds for rejection. There is no mention of any such tolerance in B.S. 968 : 1941.

We are of the opinion that a competent fabricator should have carried out check analyses on at least some of the heats received, and amongst these would surely have been included the "submitted heats"—

(*a*) because attention had been drawn to them by the supplier, and

(*b*) because of the high incidence of failure under the specified tests.

Undoubtedly, some of this steel, including heats 55 and 56, was out of specification with regard to chemical composition, and should have been rejected. There was ample time to attend to this as the W.14 girders were not fabricated for more than a year after the steel was received.

The Izod tests made on the failed girders have revealed an interesting pattern. In general, heat 55 shows very low value though occasionally, as happened when it was tested to exhaustion for acceptance, it turns up with a good record. Such a case is found at the southern fracture in girder W.14–4. These values (31 ft./lb. at 32° F. and 40 ft./lb. at 70° F.) are so surprising in view of the testing history of this heat that it is suspected a plate from some other heat has been wrongly marked.

The samples of heat 56 from the girders show mixed Izod values, sometimes conforming with the C.R.B. specification and sometimes not.

It could not have been rejected on the acceptance tests which were made, and it is futile to guess whether, had Izod tests been made on the plate, the low values or the high values would have been found. We are certain, however, that under the procedure of testing till satisfactory results were obtained, this heat would have been accepted.

In fact, no single heat was rejected throughout the contract. We are faced with the situation that a specification written by Wilson of C.R.B. in a form well ahead of its time, was made futile by the failure to apply it critically. The value of a specification is determined by the rigour of the inspection system used to implement it.

We are of the opinion that heat 55, along with several others of the submitted heats should have been rejected. We cannot, on the evidence, say that heat 56 should have been rejected. On the tests made it was justifiable to include it but whether this could be said if the full tests had been applied we have no means of knowing.

2.8. Electrodes.

Three welding processes were used, manual, automatic, and semi-automatic. All the welding at the cover plate ends, both the taper welds and the transverse weld was manual. The electrode used was Fortrex 35 made by Murex. This electrode is of the low-hydrogen type recommended for use with low-alloy steel.

There has been no evidence that the electrodes per se were other than entirely satisfactory. However, the flux surrounding the metal core is liable, as with all fluxes, to absorb moisture from the atmosphere. This moisture is decomposed in the arc at the melting temperature of the steel, releasing hydrogen which is absorbed by the weld. It is well established that such hydrogen is a main cause of cold cracking in the heat-affected zone of the weld. Because of the nature of the flux covering, the low-hydrogen type of electrode contains a minimum of inherent hydrogen and any absorbed water can be entirely eliminated by drying. This involves the use of hot boxes in which the electrodes are placed prior to being used by the welders. (See Section 2.9.1.)

This procedure was new to J. & W. and, just as it never appreciated the metallurgical difficulties associated with low-alloy steel, so it never really understood the need for meticulous care in the drying of electrodes. It was another example of the organization failing to rise to the occasion.

The packets in which the electrodes are delivered carry instructions for use including a recommended drying temperature of 150° C. to be maintained for half an hour. It is interesting to note that in experiments conducted by A.I.S. (Ex. 184) it was found that 350° C. was necessary to ensure that all water was driven off. Although the hot boxes used by the welders were thermostatically controlled, there was no evidence of any instruction given to anybody regarding the length of time the electrodes should be in the box before being used. Since adequate drying of the electrodes is essential for avoiding cracking in welds, any failure in this respect could have been an important contributory cause of the failure. If the electrodes used for the welding of the cover plate ends were not effectively dried, and used whilst in the dry condition, they could have been a cause of the toe cracking which developed at the transverse weld at the end of the cover plates. The W.14 girders were fabricated towards the end of the contract, i.e., October, 1960, by which time, it might be thought, all the essential technical controls would have been established. However, about this time Clarke mentions in his diary a number of occasions when he found it necessary to request J. & W. supervisors to collect odd electrodes lying around and return them to the hot box. The fact that he made a note of this is an indication that J. & W. personnel were not seized of the necessity for attending scrupulously to this matter.

As against the record in Clarke's diary the evidence of all the J. & W. witnesses, and Ward (Murex), was to the effect that electrodes were properly treated. However, Ward left J. & W. about April, 1960, and we strongly suspect that after his departure, by the time Clarke made the notes referred to above, the treatment of electrodes had become somewhat lax.

We are compelled to the opinion, therefore, that laxity in shop organization in the treatment of electrodes may have contributed to the failure of the bridge.

In so far as the weldability tests were carried out, the quality of the weld metal was invariably satisfactory, and we are quite satisfied that the electrodes per se played no part in the failure of the bridge.

2.9. Welding.

2.9.1. The characteristics of a weld.

When two pieces of steel A and B are to be welded together, this can be done either by a butt weld or a fillet weld. This latter type, shown at C in Fig. 1, will be used to illustrate the characteristics of welds as it was this kind which was concerned in the failure. The weld shown is a three-bead weld. No. 1 bead is the root run. It is run first and subsequently is covered by and partially incorporated in the two subsequent runs. In a similar way the second run is partially incorporated in the third run. It is an essential feature of a satisfactory weld of this kind that the first run shall fuse into the two plates and that the subsequent runs shall fuse into the first run and into one or other of the two plates. In this way there is formed a continuous metallic union between A and B. Each run or bead is laid down by striking an arc between an electrode and the work being welded. The steel electrode melts and provides the metal of the weld. In general, it is of a different composition from the plates being joined.

We thus get two plates A and B, known as the parent metal and C, the weld metal. Although A and B differ in chemical composition from C, the physical properties should be very similar, so that there is a " bridge " between A and B which will behave under stress as though the whole was continuous.

There are, however, some physical differences between the parent and the weld metal. It is for this reason that the composition of the weld metal is different. Since the latter is placed in position in the molten state it has the characteristics of a casting—in particular the grain size is very coarse. The two plates, which in the ingot stage were in a similar condition, have been hot rolled. This has had two major effects : (*a*) the crystal size was considerably reduced and (*b*) the plate acquired a laminated structure due to the elongation of segregates. This kind of structure is highly developed in low-alloy steels of the B.S. 968 type. In effect, the molten electrode is "cast" into a cavity made in the parent metal by the high temperature developed in the electric arc, and the heat developed is sufficiently intense to melt also part of the parent metal.

From this brief description it will be evident that in the immediate vicinity of a weld there is a sharp temperature gradient. The molten steel produced has a temperature of about 1,700° C., whilst the parent metal may be at ordinary temperatures, or if it has been "pre-heated" it may be, say, 200° C. Over a short distance there is a fall of about 1,500° C.

Steel is a very complex alloy because at ordinary temperatures it can be in different states according to the rate of cooling from above a red heat. If cooled sufficiently rapidly it is hardened—if cooled slowly it is soft, and there are intermediate stages.

The making of a weld involves the establishment of a temperature gradient from 1,700° C. to say 200° C. As a result there will be a zone in the parent metal around the weld which has not been molten, but which has been heated sufficiently to bring about the above-mentioned change. As soon as the weld has been completed and the source of heat is cut off, there will be a sudden cooling due to the conduction of heat into the cold regions of the parent metal. That part of the latter which has been above 800° C. will be in a hardenable condition, and give rise to a zone of steel (about $\frac{1}{8}$ inch thick in the case we are considering) which will be the harder the more rapidly it has cooled.

This is the heat affected zone (H.A.Z.) of the weld indicated by shading in Fig. 1. The kind of difference in hardness which can be expected with B.S. 968 is a follows (hardness expressed on Vickers Diamond Pyramid scale) :—

Parent Metal	*Heat Affected Zone*	*Weld Metal*
180/200	300/500	200/250

There is another effect of the temperature gradient. When heated, steel expands, and if this expansion is resisted by reason of the fact that the hot zone is surrounded by steel at a lower temperature, a compressive stress will arise which will deform the hot zone. On cooling, this same zone will try to return to its previous dimensions. The return will also be resisted and this will cause a complex stress pattern in the neighbourhood of the weld. (See Sections 2.6.2.1. and 2.6.2.2.).

The heat-affected zone, being harder than its surroundings, is more brittle and is, therefore, not so amenable to stress relief by yielding as is the surrounding metal.

To this situation a third element is added if the welding is done under conditions where hydrogen can be generated in the arc. This condition is found when the flux covering contains hydrogen or when the flux covering is not dry. Thus, for this project, a "low-hydrogen electrode" was used, i.e., one containing little combined hydrogen in the flux. It was, however, essential to dry the electrodes immediately before using.

If hydrogen is liberated in the arc, it is absorbed by the molten metal and some of it remains dissolved in the solid. As the weld metal cools, the solubility of hydrogen in the solid steel decreases, and some of it diffuses into the H.A.Z. which thereby becomes saturated. As cooling proceeds this zone reaches a temperature range (400° C. to 200° C.) in which it transforms to the low-temperature condition. During this transformation hydrogen is expelled from solution since it is less soluble in the low-temperature state. It is this expulsion of hydrogen which sets up very localized internal stress, causing cracking in the relatively hard and brittle heat-affected zone. It is considered that a hardness of 350 (V.D.P.) is the maximum permissible to avoid H.A.Z. cracking.

2.9.2. Influence of composition and rate of cooling on the heat-affected zone.

From what has been said it is evident that the H.A.Z. of a weld is a very critical region. The question arises as to how it is affected by the welding procedure. The important factors are the composition of the steel, its rate of cooling, and the presence of hydrogen. So far as the composition is concerned, the important alloying constituents are carbon, manganese, and chromium, but for the present purpose the two latter can be taken together. Carbon is the element which converts iron into steel, and it is by far the most important element in connection with the welding of steel. Pure iron has a hardness of about 80. The addition of 0·23 per cent. carbon makes possible, with a sufficiently high rate of cooling, a hardness of 450/500.

The less time there is for the steel to change from the high to the low temperature condition the harder it is. Consequently, as we have seen above, the slowly cooled plate of B.S. 968 steel has a hardness of 180/200, whereas the more rapidly cooled H.A.Z. has a hardness of 300/500. A small change in the carbon content can make a big change in the hardness.

The effect of manganese and chromium is to slow down the rate of change from the high to the low-temperature condition. The higher the content of these two elements the more readily is the hard state of the steel retained.

Consequently, to avoid undue hardness in the H.A.Z., the carbon and manganese should be kept away from the maximum limits set by the specification. However, welding is only a procedure for fabricating a structure—in this case a girder—and the girder is designed on the assurance that it will have certain specified properties. The corresponding property to hardness is the ultimate tensile strength (and yield stress). To obtain the specified properties the steelmaker must adjust the composition, taking into consideration the rate of cooling of the plates as they come from the rolling mill. From this is seen that the steelmaker may want to keep near the maximum limits of the specification as regards carbon and manganese particularly with thicker plates which cool more slowly whilst the fabricator would find his task easier if the steelmaker kept below them.

It is this dichotomy of interest which leads to the kind of difficulty with which we were confronted in this Enquiry, and it points to the necessity of the closest collaboration between the steelmaker and the fabricator concerning the composition of the steel.

2.9.3. The concept of thermal severity.

The rate of cooling of a weld has been shown to be a factor in controlling the hardness of the H.A.Z. and the hardness is one factor in controlling the liability to cracking. The various factors which in turn control the rate of colling of a weld are :—

(*a*) The geometry of the weld.

(*b*) The temperature of the weld before welding commences.

(*c*) The heat input during welding which is controlled by the gauge of electrode used, the rate of melting (burn off) of the electrode per inch run of weld, and the welding current.

The first of these items governs the thermal severity of the weld and is associated with the conduction of heat away from the molten pool of metal (C. in Fig. 1). The thicker the plates, and the greater the number of paths along which heat can be conducted, the greater is the thermal severity of the joint. The unit used to define this is the heat flow through one thickness of $\frac{1}{4}$-in. plate. For example, a butt weld of two $\frac{1}{4}$-in. plates would have a thermal severity of two, whilst that for two $\frac{1}{2}$-in. plates would be four, and that of a T-weld in $\frac{1}{2}$-in. plate would be six. Thus the thermal severity of a joint is expressed by a number (T.S.N.) which is simply four times the total thickness of the plates through which heat can flow away from the joint. It follows that the greater the T.S.N. of the joint, the greater will be the cooling rate under any given set of welding conditions.

As mentioned earlier, the temperature of the molten pool of steel under the arc is about 1,700° C. The transfer of heat from this pool is more rapid the greater the temperature difference between the weld and the surrounding steel which is conducting the heat away. That is, with a joint of given T.S.N. the cooling rate of the weld is greater the lower the temperature of the weldment at the completion of welding. It follows that the cooling rate can be reduced by pre-heating the weldment. This pre-heating can be done by gas flames being played around the location of the joint, making sure that all conducting paths are raised in temperature. It is generally conceded that the zone heated should be at least 3 inches wide in every direction.

There is, however, another way of introducing heat into the weldment so as to reduce the cooling rate. This is by increasing the rate of deposit of the molten metal from the electrode. It will be evident that the energy input during welding is controlled by the welding current. With heavier gauge electrodes heavier currents can be used, and it is possible to melt and deposit the weld metal at a greater rate. In this way the temperature of the steel in the immediate neighbourhood at the completion of the weld is higher the greater the rate of melting. This is an alternative or supplementary way of reducing the temperature gradient and so of reducing the rate of cooling.

2.9.4. Standard recommendations for welding B.S. 968: 1941.

As a guide to fabricators the British Standards Institution has written B.S. 2642: 1955, " General requirements for the metal arc welding of medium tensile weldable structural steel to B.S. 968 types".

This was at first adopted by C.R.B. It controls the quality of electrodes to be used by cross-reference to B.S. 2549: 1954, and also lays down details relating to butt and fillet welding.

In clause 25 of B.S. 2642 attention is drawn to the conditions of storage of electrodes and the need for these to be kept dry. This is in connection with the possibility of introducing hydrogen to the weld from moisture in the electrode flux covering. The matters considered in section 2.9.3 are governed by clause 26 which sets out the items to be taken into account in determining the weld procedure. In this it acts as a guide only, directing the fabricator to those items to which he must give attention for the production of satisfactory welds in B.S. 968. The items are:—

(*a*) Classification, type and size of electrodes.

(*b*) Current and minimum open circuit voltage, and, for automatic welding, arc voltage.

(*c*) Length of run per electrode or (for automatic welding) speed of travel.

(*d*) Number and arrangement of runs in multi-run welds.

(*e*) Welding positions.

(*f*) Welding sequence.

(*g*) Pre-heating.

In an Appendix (*c*) details are given relating to the various factors which prescribe the welding conditions using low-hydrogen electrodes to produce crack-free welds. Charts show the relationship between gauge of electrode, run length per electrode (butt weld) size of fillet, thermal severity of the joint and the necessary pre-heating temperature.

Early in 1959, i.e., before fabrication had commenced, C.R.B., J. & W. and Utah agreed to substitute for B.S. 2642, a brochure produced by B.W.R.A. entitled " Arc welding low-alloy steel". This was, therefore, used as the welding control guide throughout the project.

This booklet "is for welding engineers, shop and site supervisors and those responsible for drafting specifications, and summarizes the practical results of a considerable volume of research work done by B.W.R.A." It is much more descriptive and discursive than B.S. 2642 and could well have been adopted as a textbook to supplement the training of welders for this type of steel.

It highlights the features of the H.A.Z. of welds and draws attention to those factors which affect the formation of cracks. These are listed as:—

Cooling rate of the heat-affected zone—size of weld run, size of joint, temperature of joint.

Composition of the steel.

Type of electrode.

Stress in the heat-affected zone.

Two methods are then described for determining the " weldability" of any piece of steel, i.e., the ease of fabrication of a given steel under the imposed conditions available to the fabricator. One method, and the one recommended, is an experimental one in which welds are made in a standard way using the actual steel to be employed on the project. This is called the controlled thermal severity (C.T.S.) test. The objective is to determine those welding conditions which will avoid heat-affected zone cracking, without unduly restricting the fabricator. The complete test needs three test assemblies using three plate thicknesses—$\frac{1}{4}$ inch, $\frac{1}{2}$ inch, and 1 inch giving joints with thermal severity numbers of 2 to 12, and the same electrode is to be used as will be used on the project. As a result of this test a " weldability index" can be allotted to a given heat of steel, and this index, denoting a characteristic of the steel, can then be used to determine the pre-heat required, using a

given size of a given type of electrode. The great advantage of this experimental approach is that the fabricator learns a lot about the particular features required for the successful welding of this steel. The weldability index is in seven grades designated A to G with the difficulty of welding without cracks increasing in that order.

Weldability in this sense is largely governed by the composition of the steel and particularly by the carbon, manganese and chromium contents in B.S. 968. By experience it has been found that the weldability index can be estimated by using a formula which takes into account the known facts that crack-free welding is more difficult the higher the carbon content and that manganese and chromium can be allotted a " carbon equivalent " since they act in the same direction but are less potent. The total percentage carbon plus this carbon equivalent is spoken of briefly as the " carbon equivalent " of the steel. A relationship has been found between the weldability index as determined from the C.T.S. test and the carbon equivalent as expressed by the formula. The weldability of a given steel must be associated with a given type of electrode. Since hydrogen is a potent cause of H.A.Z. cracking the low-hydrogen class of electrodes can tolerate a higher carbon equivalent than other classes. The correlation between the two methods for determining the weldability index, using low-hydrogen electrodes is—

Weldability Index	*Carbon Equivalent Percentage*
A	Up to 0·25
B	0·26 to 0·30
C	0·31 to 0·35
D	0·36 to 0·40

The booklet points out that the weldability index can be calculated with a fair degree of accuracy, usually within one index letter, by using the formula. Obviously to have even this degree of accuracy it is necessary to know the composition of the actual plates to be welded. As pointed out (2.7.2.1) there may be an appreciable difference between the plate analysis and that shown on the analysis certificate from the steelmaker. This difference could be sufficient to change the weldability by one index letter. The booklet states that when only the steel specification is available, it must be assumed that the steel composition is at the high end of the permissible composition range. For B.S. 968 this would mean 0·23 per cent. carbon, 1·8 per cent. manganese, and 0·2 per cent. chromium, giving, from the established formula a carbon equivalent of 0·34 per cent., near the top end of the range for weldability index C. The necessity for the fabricator to check the actual plate analyses for himself is shown in Section 3.4.2 in which it is revealed that some steel for the Kings Bridge project had a carbon content 0·26 to 0·28 per cent.

Having decided the index, the booklet then describes how to determine the pre-heat temperature which for a joint of given thermal severity and using a given size run-length combination of a given type of electrode will give crack-free welds.

It was an admirable, concise, effective description of how to do what the fabricator had undertaken to do. At the Enquiry we learned how few people, even supervisors at J. & W. had in fact even seen a copy.

Clarke (C.R.B.) had made some abstracts from it in his notebook and this appears to have been the only source of information relating to the B.W.R.A. booklet which J. & W. personnel saw.

2.9.5. Non-destructive examination of welds.

Welding is a skilful art even when it is done by automatic machines. It is still more of an art when done manually. It will be evident from what has been said that successful high-class welding needs—

(*a*) good fit of the sections of the joint ;

(*b*) good fusion and penetration of the parent metal so that an effective union is assured ;

(*c*) a correct contour of the external surface ;

(*d*) freedom from under-cutting and overlap ;

(*e*) absence of cracks whether internal or coming to the surface.

It follows that careful inspection of all welds on which the strength of a structure depends is an essential feature of welding construction.

Broadly, inspection methods divide into destructive and non-destructive. The former can only be used to determine the weldability factors on the one hand or the suitability of the production welding procedures on the other. In other words they are used to test the materials and procedures.

It is, however, essential that the final structure shall be examined, and for this, non-destructive methods must be used.

The most obvious method is a simple visual examination of the whole weld, with perhaps a magnifying glass as a visual aid. This constitutes a first rapid survey and serves to detect categories of defects under (*c*) and (*d*) above, and also some of those under (*e*).

It is, however, the hidden flaws covered by (*a*), (*b*), and (*e*) that give rise to the need for more sophisticated methods of inspection. We need " eyes " which will see what is below the surface or even what is at the surface though not readily visible. Four main methods are in use.

2.9.5.1. *The penetrant dye method.*

The weld surface is cleaned and a light penetrant oil is brushed or sprayed on. The oil seeps into any cavity or crack open to the surface and after a few minutes the excess oil is washed off. After drying either naturally or by a warm air blast, a liquid " developer " is sprayed on to the surface to be examined. This contains a fine white powder and on drying this remains as a film. The oil which had penetrated the defect is drawn back into this film and produces a dark stain. A crack will show as a dark line on a white background. Extensive lengths of weld can readily be covered and cracks will be found which were not readily visible directly.

2.9.5.2. *The magnetic powder method.*

This is applicable to steels because wherever there is a discontinuity, an applied magnetic field is distorted. It is useful in detecting similar defects to those observable by penetrant dye, but will also detect defects close to, but not penetrating, the surface. There are several variants of the method. In one the weld is first sprayed with a white powder film like the one used with the penetrant dye. This forms a background. The weld is then subjected to a strong magnetic field and a fine magnetic powder is blown gently on to the surface to be examined. Wherever there is a discontinuity, such as a crack, the finely-divided magnetic powder (black on the white background) accumulates. The nature of the defect is then indicated. This method also is readily applied to extensive lengths of weld, provided they are accessible to the application of the magnetizing field.

2.9.5.3. *Radiation examination.*

X-rays and gamma rays (from radioactive materials) can penetrate metals just as light can penetrate glass. Any discontinuities in the metal can be recorded on photographic film. Thus a cavity, or a crack if lying in the right direction, will show on an exposed film as a dark spot or line. The interpretation of the film requires experience and skill. It is possible to examine long lengths of accessible welds, such as butt welds, making individual " exposures " equal to the length of film used. It is not very suitable for examining fillet welds owing to the complex geometry, and the difficulties in inclining the beam and film so as to utilize effectively the differences in absorption between the sound metal and the defect. This applies particularly to T-joint welds such as the web-flange weld in the girders.

2.9.5.4. *Ultrasonic wave method.*

This depends on the generation of extremely short-length waves of the same nature as sound waves. They are so short (i.e., of such high frequency) that they are inaudible to the human ear. Metals are " transparent " to such waves which are unable to pass through gases. Consequently, if such a beam meets a crack or any other discontinuity in the metal which contains a gas, the beam is reflected. The equipment consists of a

generator to produce the short waves, and a detector usually an oscillograph to pick up the " echo " reflected from the defect. The echo shows as a sharp peak on the oscillograph trace.

As with X-radiography, the interpretation of the indications requires experience and skill particularly with complex joints.

2.9.5.5. *Use of non-destructive testing in the girder fabrication.*

The C.R.B. inspectors used visual examination and the penetrant dye method extensively for their fillet weld survey. They used X-radiography extensively for butt-weld examination.

The cover plate end weld was a difficult one for instrumental examination, though possibly the ultrasonic method could have been developed for this purpose. The type of defect which in our opinion led to the failure, namely, the toe crack in the flange at the transverse weld, was amenable to detection by the penetrant dye method.

Our views on whether this was used or not are expressed elsewhere. (See 3.3.3.).

2.9.6. Welding on the Kings Bridge project.

It has been pointed out elsewhere that, at the beginning of the contract to fabricate the superstructure of Kings Bridge, J. & W. was inexperienced in the welding of low-alloy steel. It was essential, at this stage, for a small group of experienced welders to be given theoretical and practical instruction in the welding procedures ultimately to be used in fabrication.

But nobody at J. & W. recognized that there was an educational problem involved in the contract, and so nothing was done in this direction.

The J. & W. inspector and supervisor, Campbell, who gave instructions to the welders, admitted that he himself had never read the B.W.R.A. booklet and had only copied a table from it out of Clarke's notebook.

It is therefore understandable that the evidence of supervisors, charge hands and welders contained many contradictions. The impression is gained that the clear, concise and authoritative information contained in the B.W.R.A. booklet—which had been adopted by all parties concerned—was simply not made available to those most in need of it. There was no period when J. & W. really set out to determine for itself and for the education of its supervisors, charge hands and welders the weldability characteristics of B.S. 968 steel.

During 1959, when construction techniques were being evolved, automatic welding procedures were developed. Ultimately these produced results which, after many defects were repaired by manual welding, were accepted by C.R.B. inspectors as satisfactory. The automatic machine was used for butt welding both flange plates and web plates and for fillet welds of cover plate to flange and flange to web. At first the stiffeners were manually welded into position but later this was done by a " squirt gun ", a semi-automatic machine. For automatic welding " Murawire " and " Muraflux " were used.

Because of the design of the cover plate it was not possible to complete the welding to the flange automatically. At each end there was a tapered part which reduced the width of the cover plate from 14 inches to 3 inches (See Fig 11). These tapered sides and the transverse 3-in. end were welded manually generally using three runs. For this work the Fortrex 35 low-hydrogen type electrode made by Murex was used.

2.9.6.1. *Weld sequence.*

We have already considered (See 2.6.2.2) the influence of sequence on residual stresses. There was no pre-determined sequence for the laying of the three cover plate end welds. The general impression gained from the evidence was that the two taper welds were made before the transverse weld. If this is so, this critical weld was made at a location of maximum restraint.

Experimental assemblies of this cover plate end made during the extensive investigations in A.I.S. laboratories since the failure (Ex. 184), have indicated clearly that the sequence of welding round the end of the cover plate is an important factor in the production of toe cracks at the transverse weld.

There was no evidence at the Enquiry that J. & W. made any experimental replicas to determine the thermal severity of this joint. There was, however, evidence that the end weld was made sometimes before and sometimes after the web was welded to the flange and further that this important sequence was determined by shop organization expediency rather than as a result of pre-determined principles. The importance of the web/flange sequence so far as the cover plate end weld is concerned is two-fold.

(*a*) The presence of the web weld increased the thermal severity of the joint in a peculiar way in that an extra heat-conduction path was located only across the centre of the transverse weld. This increased the complexity of the internal stress condition. There is evidence of the increased thermal severity at this location in the A.I.S. investigations where it is shown that the hardness of the H.A.Z. of the transverse weld is greater under the web weld than elsewhere—indicating a higher cooling rate.

(*b*) The weld, already restrained laterally by the two taper welds, was restrained in thickness as well if the web was already in position.

2.9.6.2. *Weld defects in fabrication.*

During the Enquiry attention was concentrated almost wholly on the transverse weld at the end of the cover plates. This was natural as this was the location of the fractures which caused the W.14 span of the bridge to collapse. It was revealed in evidence that no crack in this location had been detected during the fabrication period. We express our views elsewhere on the reason for this. (See 3.3.3.). Although the only part of the bridge which has failed is the W.14 span, it is necessary to consider the general position relating to cracking during fabrication, and the possibility of cracks being present in the remaining spans of the bridge. Our reason for doing this is to issue a warning that cracks in the structure which are transverse to tensile stresses, may develop into fatigue cracks under the repeated loadings imposed by traffic. The Enquiry has revealed that the cracks which caused the collapse of the W.14 span were associated with the fabrication techniques used, and the quality of the steel supplied, and are therefore not necessarily confined to a given type of weld or a single span.

The main information relating to cracking during the fabrication lies in two sources :—

(*a*) The C.R.B. statistical analysis of details of inspection of the defects in the main girders (Ex. 222) ; and

(*b*) the various reports (chiefly by Murex and A.I.S.) relating to difficulties which developed during the progress of the contract.

The C.R.B. survey relates to both visual and radiographic examination of the main fillet welds and butt welds—both automatic and manual.

From this survey we conclude that in the fillet welds there are possibly fifteen serious undetected transverse cracks and in the butt welds of tension members (flange and cover plates) there are possibly ten serious undetected transverse cracks. The difference between detected and repaired cracks, and undetected cracks is that the latter are still in the bridge.

The important question remains, as to whether all cracks were present at the time of inspection. It is the view of several witnesses (Ferris and Professor H. Muir included) that weld cracking can occur up to three days after the weld has been made. This time is determined partly by the welding conditions, partly by the ambient temperature and partly by the stresses applied to the girder during handling. There is no evidence to indicate that the C.R.B. inspectors knew of this delayed cracking. It is possible therefore that cracks formed in some of these welds after they were examined, and these (if any) will also still be in the bridge.

2.9.6.3. *Survey of cracks at cover plate ends.*

Since the collapse of W.14, M.M.B.W. as part of its reconstruction programme, has examined many cover plate ends on the tension flanges in the remaining spans of the bridge. As it had been established by E.T.R.S. (Exs. 48 and 49) using non-destructive methods of examination, that many of these welds were probably cracked in much the same way as those in the W.14 span, M.M.B.W. decided to remove these ends by a milling operation. Whilst doing this, in order to aid the Commission in its Enquiry, a log was kept of the location, length and depth of any cracks revealed (Ex. 218). When such cracks were found the milling was continued until the crack was cleared. The depth of milling required to clear the cracks varied from $\frac{1}{16}$-inch to $\frac{5}{8}$-inch but there were several cases where the crack passed completely through the flange. These latter are considered separately.

Attention is drawn to the fact that these cracks were not detected by the C.R.B. inspectors, although 35 of them were at the toe of the transverse weld (the remainder were mostly at the toe of the root weld and so were not detectable by the penetrant dye method). The M.M.B.W. observation raised the previously stated possibility regarding undetected cracks in fillet and butt welds to the level of strong probability.

We have made an analysis of the data provided by this operation of M.M.B.W. in order to help us to assess—(*a*) whether the cracking was associated with any particular heats of steel (*b*) whether the cracking was associated with any particular period during the contract (*c*) whether the cracking was associated with sequence of welding.

2.9.6.4. *Association of cover plate end cracking with heats of steel.*

Table 2 shows the heat number and type of steel and number of cracked ends revealed by the M.M.B.W. milling operation, plus the eight ends of the W.14 girders.

TABLE 2.

Heat No.	Type	Number of ends showing cracks	Heat No.	Type	Number of ends showing cracks
2	O.H.S.K. ..	1	56	E.F. ..	4
16	,, ..	15	58	O.H.F.K. ..	4
22	,, ..	8	70	E.F. ..	1
28	,, ..	1	71	,, ..	1
29	,, ..	15	72	,, ..	1
30	,, ..	2	73	,, ..	2
39	,, ..	1	79	,, ..	2
48	E.F. ..	1	80	,, ..	6
50	,, ..	8	81	,, ..	5
55	,, ..	5	84	,, ..	3
					Total 86

O.H.S.K.—Open-hearth, semi-killed. O.H.F.K.—Open-hearth, fully-killed. E.F.—Electric furnace, fully-killed.

Altogether the M.M.B.W. has given us results from the examination of 160 cover plate ends from the suspension girders. We thus have :—

Total ends examined 168. Cracked 86. Not cracked 82, or approximately 50 per cent. are cracked.

The ends containing cracks were made from seven semi-killed open-hearth heats, one fully-killed open-hearth heat and twelve fully-killed electric furnace heats, and the composition shown in the related heat certificate is as given in Table 3.

TABLE 3.

Analyses of Heats Concerned with Cover Plate End Cracks.

These analyses have been taken from the A.I.S. test certificates (Ex. 20). The figures in brackets refer to plate analyses made either before or after the steel was delivered to J. & W. and represent the maximum values reported during the Enquiry.

Heat	Type	C.	Mn.	Cr.	Heat	Type	C.	Mn.	Cr.
2	O.H.	·19 (·24)	1·66 (1·80)	·13	56	E.F.	·23 (·27)	1·58 (1·71)	·24 (·29)
16	,,	·21 (·24)	1·70 (1·92)	·10 (·08)	58	O.H.	·22 (·23)	1·69 (1·75)	·25 ?
22	,,	·21 (·28)	1·55 (1·91)	·12 (·38)	70	E.F.	·19	1·64	?
28	,,	·22	1·75	·27	71	,,	·21	1·66	?
29	,,	·23	1·56	·30	72	,,	·22	1·63	?
30	,,	·22	1·56	·30	73	,,	·20	1·61	?
39	,,	·19	1·61	·16	79	,,	·21	1·65	?
48	E.F.	·21	1·72	·24	80	,,	·22	1·69	?
50	,,	·22	1·65	·21	81	,,	·23	1·61	?
55	,,	·21 (·28)	1·70 (1·80)	·23 (·25)	84	,,	·19	1·60	?

NOTE.—The heat analyses show carbon to be in the range from ·19 per cent. to ·23 per cent. and manganese from 1·5 per cent. to 1·7 per cent. If these were a reliable indication of the composition of the steel supplied it would be an indication that steel composition could be discounted as a source of welding difficulty. It will be noticed, however, that wherever the plates as delivered have been analysed there is invariably an appreciably higher content of carbon and manganese than shown on the test certificate. It would seem therefore that where composition is critical the analysis should always be checked.

The semi-killed steel heats account for 43 of the cracked ends, i.e., 50 per cent. It will be noticed that three of these heats—Nos. 16, 22 and 29 account for 38 of the 43 cracked ends, i.e., 88 per cent. This suggests that the steel of these three heats was defective or of low weldability. It is on record that heats 16 and 22 had given trouble during fabrication and had been the subject of metallurgical investigation.

However, there is another factor to be taken into account. Of the 93 tension flange plates associated with cover plate ends in suspended girders (Ex. 143) which were made from open-hearth steel, 77 were from the 3 heats mentioned. If we eliminate these 77 from the total of 168 we get 91 cover plate ends of which 48 were cracked. We thus have :—

Approximate percentage of all ends examined which show cracks .. 50

Approximate percentage of all ends examined which show cracks other than from heats 16, 22 and 29 52

so that it is evident that the inclusion of the three heats reduces rather than increases the percentage of cracked ends. We conclude that so far as weldability was concerned the three heats concerned were neither better nor worse than the others.

Accepting that 50 per cent. of all cover plate ends show cracks of one kind or another, there is no reason why similar defects should not be found in other than the suspended girders.

From Ex. 143 we find that there are 187 tension flanges (top) in the cantilever girders. It is possible therefore that about 100 of these show cracks. They are under the concrete deck, but we understand that of those examined several have been found to be cracked.

We point out, however, that whilst all cracks in tension members are dangerous, some are worse than others. From the M.M.B.W. records we believe that 41 cracks in the suspension girders were serious toe cracks before they were removed and possibly 50 similar ones remain in the cantilever tension members.

An examination of Table 3 strongly indicates that the welding difficulties causing these cracks came from the high carbon and manganese contents of these heats, to which J. & W. did not react. It is of interest to note that information concerning out of specification

analyses came to light as a result of J. & W. taking action itself. It had plate checks made early in March, 1960—presumably because of welding difficulties in December, 1959 to February, 1960. Having got the results, J. & W. was not alerted to the source of the troubles, in spite of similar reports from Murex about the same time (See letter 5th March, 1960, Murex to J. & W.) showing the analysis of a cracked plate :—

	Carbon	*Manganese*	*Chromium*
	·28	1·9	·38
and from heat 22	·24	1·8	·56

Murex warned J. & W. that particular care would be needed with pre-heating temperatures in order to avoid cracking with material of this type.

2.9.6.5. *Association of cover plate end cracking with date of fabrication.*

We have analysed these results further in order to find whether girders with cover plate end cracks were fabricated in any particular period. For this purpose we have used Ex. 218 (with additional information from M.M.B.W.) and 88 (dates of completion of girders).

The information we have relates to girders fabricated in the thirteen months from December, 1959, to December, 1960. The cracks are found to be fairly regularly distributed in girders fabricated throughout the year as follows :—

	Cracked Welds
December, 1959–March, 1960	25
April–June	22
July–September	19
October–December	20

We find also that the three heats 16, 22, and 29 were used throughout the year.

We conclude that there was no stage of fabrication particularly associated with the production of cracked welds.

2.9.6.6. *Association of cover plate end cracking with web-flange welding sequence.*

It has been mentioned that the transverse weld at the cover plate end, although only 3 inches long and an easy weld to lay, either by down-hand or vertical-up procedure, can be difficult to weld without cracking, unless precautions are taken with respect to pre-heat and sequence. Two sequences were involved—

(*a*) whether it was laid before the web was welded to the flange, and

(*b*) whether it was welded before or after the taper welds. Whenever the questions regarding these sequences was asked in the Enquiry, whether of K.S.B.D., the C.R.B., or J. & W., we got the same answer : "Nobody gave it a thought, and it was done just when it was most convenient for shop practice."

Regarding the major sequence web-flange end of cover plate we have some data in Ex. 89 from C.R.B. records, but the information is not sufficient for us to make a clear analysis in conjunction with Exhibit 218. We only know definitely that on certain girders the cover plate end was the last weld to be made. Regarding the minor sequence it seems that the transverse weld was probably always done after the two taper welds. Francis considered that this was the worst sequence to use.

Assembling the available data from the M.M.B.W. operation we get the following result :—

(*a*) Of the cover plate ends completed with the transverse weld the last to be made on the girder, approximately 65 per cent. showed cracks.

(*b*) Of the remaining 67 cover plate ends, for which we have no record of the web/flange cover plate end sequence, 37 (55 per cent.) showed cracks and 30 (45 per cent.) were free.

Since about 75 girders had been made before the period covered by this survey, it is possible that most of the second group above were fabricated by the same sequence as the first group. In this case we reach the conclusion that the chances in favour of producing a cracked weld if the transverse weld is the last made was about 3 to 2.

2.9.6.7. *Cracks which have developed beyond the toe crack stage.*

In analysing above the data from Exhibit 218 (plus the additional information which has come to light since that was tendered) we have not distinguished cracks which have remained as toe cracks and those which have developed into partial brittle fractures. It is not possible to do this without making an assumption regarding the average depth of toe cracks.

In order to examine the question of the general liability of the steel to brittle fracture we now assume arbitrarily that a crack which is deeper than $\frac{1}{4}$ inch has propagated from an initial weld crack. Examples are shown in Table 4.

TABLE 4.

Girder	End	Heat	Depth	Location
			inch.	
E.3S3	North	29	$\frac{5}{8}$	Toe
E.14S2	"	50	$\frac{5}{8}$	Toe and root
E.14S3	"	50	$\frac{5}{16}$	Between toe and root
E.14S4	"	50	$\frac{5}{16}$	" " "
E.1S4	"	80	Through $\frac{3}{4}$	Toe
E.3S1	South	29	$\frac{3}{8}$	Toe
W.12S1	"	58	$\frac{9}{16}$	Between toe and root
E.6S2	North	22	Through $\frac{3}{4}$	?

Combining this with Section 2.7.6.4 it will be seen that 13 of the 86 welds showing cracks, have developed into partial brittle fractures. Most of these were from toe cracks or cracks closely associated with the toe. Seven different heats of steel (including heats 55 and 56) are involved compared with 23 heats (three of which showed no cracks) in the cover plate end survey. Another way of expressing this is to say that of 86 transverse cracks, 73 have not developed into brittle fractures, and as these are distributed through eighteen heats it is an indication that the general run of steel was sufficiently notch-ductile to resist propagation of brittle fractures under the conditions to which it had been subjected.

2.9.6.8. *Other cracks in the bridge.*

The Commission requested E.T.R.S. to examine a " sample " of the girders to determine whether the attention which had been focussed on the cover plate ends might have led us to the mistaken idea that the remainder of the bridge would be found satisfactory. Sufficient has been revealed in the analysis in this section of our Report to show that no complacency can be maintained in this matter.

E.T.R.S. undertook to examine cover plate to bottom flange fillet welds of the E.14 span, and also the manual fillet welds at the taper ends of these cover plates. The survey could only be done on a sampling basis as far as the longitudinal welds were concerned. Four lengths of 1 foot each were selected on each of the eight welds, and the sixteen taper welds were also examined. No cracks were detected by the magnetic particle method, which reveals cracks reaching the surface. The ultrasonic method was then applied (Ex. 232) and cracks from 4 inches to 15 inches long were indicated in five of the eight taper welds at the north end of the girders. The ultrasonic method used was then checked jointly with D.S.L. on the taper welds of W.14. As a result (Ex. 237) Hudson " has little doubt . . . that the ultrasonic indications found in the northern end taper welds of the E.14 series of girders are in fact indications of cracking in the flange. . . . It is concluded, therefore, that quite extensive inner run toe (or possibly root) cracking is present in the E.14 girder flanges under some of the taper welds ".

Ferris in a supplementary report (Ex. 231) gives conclusions relating to an examination of taper welds and manual fillet welds around the end plates of the W.14 girders. In both locations cracks were found.

From these examinations and our analysis of the welding defects survey, we conclude that the cracking at the cover plate ends which we have so carefully examined throughout our Enquiry, is characteristic of the whole of the welding of this detail in the bridge and is not a unique feature associated with these particular welds in the W.14 girders which dramatized the effects resulting from steel quality and welding procedures because all factors were operating at once.

2.9.7. Summary of views relating to welding.

From the foregoing sections relating to the welding on the Kings Bridge project, we reach the following conclusions :—

1. J. & W. did not adequately prepare its staff and workmen for welding low-alloy steel.
2. No pre-determined sequence was set down for laying the cover plate end weld, the importance of which was not realized.
3. The tension flanges of the suspended spans in the bridge possibly still contain about fifteen serious undetected transverse cracks in the main fillet welds and ten serious transverse cracks in the flange and cover plate butt welds.
4. Other cracks may have formed since the welds were inspected during fabrication.
5. Of 160 cover plate ends (tension flanges of suspended girders) removed by M.M.B.W. in its reconstruction programme, just over half were found to be cracked.
6. It is probable that a similar proportion of cracked welds is still in the bridge in the top flanges of the cantilever spans.
7. The high carbon and manganese content of some heats increased the difficulty of producing crack-free welds. However, the cracking found is so general that we consider the fabrication techniques to be largely responsible for them.
8. Cracking at the cover plate ends is general throughout the bridge. The collapse of the W.14 span focussed attention on the cover plate ends, but we think that several causes operated simultaneously to determine this location of the failure.

2.10. The Fabrication of the Girders.

In order to make a point or to justify a conclusion, for example as to the competence of one of the parties to undertake the project, some incidents have already been mentioned, in previous sections, which might with equal logic have been included in this one.

In the present section, the attempt is to paint a picture of what went on during the progress of the project. In so doing, we have found it necessary to indulge in a certain amount of recapitulation—not so much, we hope, as to be tedious—but sufficient to convey an accurate impression of the scene.

Even so, this does not purport to be a complete account of all the matters that emerged during the hearing but it does, we believe, bring together the most important matters with which we are concerned.

2.10.1. Preparation and checking of design and shop drawings.

After the award of the contract to Utah, K.S.B.D. set to work to expand the tender drawings into a complete set of design drawings supplemented by the calculation sheets on which they were based. The drawings and calculations were checked in meticulous detail by C.R.B. and it is for this reason that we say, that whatever responsibility K.S.B.D. carries for adopting cover plates and the detail of their termination, must be shared by the C.R.B. (See Sects. 2.5.1. and 2.5.2.).

The design drawings give only the main dimensions and so it was necessary for J. & W. to prepare from them a series of shop drawings carrying every dimension necessary for the fabrication of the girders. These drawings were also checked by the C.R.B.

It is at this point that we see the need for someone who, for want of a better term, we call a " welding engineer ". We visualize him as a man, skilled both in the technique of welding and in the art and science of structural design, whose duty it would be to ensure both that what was being asked for by the designer could, in fact, be satisfactorily fabricated, and also that the proposed fabrication procedures would fulfil, or at least not frustrate, the designer's intentions. We think that the shop drawings should carry a schedule showing the welding procedures to be adopted at each joint, the order of laying down the welds, the required pre-heat, any special points to be noted by the inspectors and, indeed, all the details required to carry out the work.

Whether or not this be the best way of filling the gap between the designer and the fabricator, the fact remains that in this case nothing of the sort was attempted with the results described below.

2.10.2. Ordering the steel.

In Section 2.3.4.2. there is a full description of the series of events that resulted in J. & W. ordering steel to B.S. 968 without stipulating the additional C.R.B. tests. There is, therefore, no need to go over the story again except to say that we still cannot conceive how B.H.P. could accept this order from an old and valued customer—

(*a*) if they knew about the additional tests ;

(*b*) if they realized the significance of additional tests from the point of view of brittle fracture.

The evidence of Ralston and Thompson led us to think that the decision not to accept an order including the extra requirements of the C.R.B. specification (i.e., additions to B.S. 968 : 1941) had been made at too low a level in the organization. However, after seeing a letter of 25th August, 1959 from the General Manager, Newcastle Steel Works, to the Managing Director, B.H.P., and other communications in Exhibit. 204, we realize that the policy of not accepting additions to specifications had been discussed at higher levels. We also conclude from these internal communications that the full requirements of the C.R.B., in this connection, were quite well known and appreciated, though the witnesses we heard gave the impression that the full specification had never been received by B.H.P.

This was not a minor matter : what was involved was a specification for an important metropolitan highway bridge in a capital city with difficult traffic problems, and the designers of a public authority had decided that it was necessary to protect the bridge from possible failure by brittle fracture.

Brittle fracture had been before the public eye in connection with welded ship and other failures and the views of the C.R.B. engineers merited discussion rather than categorical rejection. This situation is not unconnected with the type of contract which was drawn up for this project. It is evident that, as the C.R.B. reserved to itself the final approval of the steel, there should have been a much closer association between C.R.B. and B.H.P. in these matters.

It is quite clear to us that J. & W. had unlimited faith in B.H.P.'s ability to supply satisfactory steel for the project. To tell such a fabricator, knowing that it had no metallurgical department to use as a check, that Izod testing of this steel was, to all intents and purposes, wasting time, was to play on its faith, and to confirm it in an attitude to testing in general which we now know to be characteristic of J. & W. The issues were much larger than could be dismissed in this way, although we appreciate that J. & W. was told in good time (i.e., before tendering) that the steel could not be guaranteed to pass the stipulated Izod test. However, we also appreciate that having ordered to one specification and contracted to another more stringent one, J. & W. could have been left to sort out material which would pass the C.R.B. specifications from among what was received.

Before leaving this matter of policy on notched bar testing we must point out that U.K. steelmakers had given a great deal of attention to the brittle fracture problem in the early 1950's ; much research effort was directed to the theoretical and manufacturing aspects of the problem. One firm made a statistical analysis of 60 heats of steel, relating the rolling conditions to the notch ductility by applying the Charpy test to determine the ductile/brittle transition range for several thicknesses of bar.

This was in marked contrast to the attitude of A.I.S. metallurgists in dismissing the test as " completely useless and illogical from a metallurgical point of view ", (letter 25th March, 1959 from Chief Metallurgist, A.I.S., Port Kembla). They appear to have made no study of the notch ductility properties of this steel as produced in their own plant, and if they were aware of the results of impact testing carried out in the B.H.P. organization the full significance of these was not appreciated.

Our views on the lack of this type of technical research are based on the opinions and attitude of witnesses for B.H.P., not one of whom suggested that such work had been done or had ever been contemplated, and who were rather on the defensive to prove that the B.H.P. attitude had been a correct one.

2.10.3. The supply of steel.

Apart from two basic open-hearth heats made at the Newcastle plant of B.H.P. the steel for this contract was made by A.I.S. at Port Kembla. Approximately half was from basic open-hearth semi-killed heats and the remainder from fully-killed electric furnace heats. (An open-hearth heat, in this sense, is the product teemed from a 125-ton ladle).

In spite of modern scientific aids steel-making is still something of an art ; it is not necessarily the case that steel made at one plant will be the same as that made to the same specification at another plant.

We will endeavour, therefore, to sort out those aspects of the steel supply which relate to general company policy (B.H.P.) from those which relate to the particular plant where the steel was produced. (A.I.S., Port Kembla).

2.10.3.1. *Matters of general policy.*

The first major matter of general policy was that the steel would be made at the Port Kembla plant.

We assume that this decision was related to technical considerations associated with the characteristics of the two plants and this matter is of no concern to the Commission. We do not know whether the steel would have been better if it had been made at Newcastle and we simply observe, as has been mentioned before, that the practice of the two plants was not the same.

The second general matter—that the company would not undertake to produce steel to the full C.R.B. specifications—has already been discussed in Section 2.10.2.

Finally we come to some questions of B.H.P.'s relations with its customer which we presume to be company matters and not specific to the plant producing the steel.

On several occasions during the project J. & W. were in difficulties of several kinds relating to defects which developed in welding. Port Kembla officers on such occasions made helpful investigations and suggestions. At one stage, they even offered to send expert welders to J. & W. to demonstrate what they considered to be proper techniques. J. & W. did not accept the proffered help, apparently because in the meantime Ward (Murex) had been seconded to it for this purpose.

There was one matter which was apparently decided at too low a level. This related to C.R.B. heat 22, which was first mentioned in correspondence from J. & W. to B.H.P. early in 1959 ; this was an open-hearth heat to which chromium had been added. In the investigation report of 12th March, 1959 no mention was made of chromium but it was suggested that material from some other heat might have been inadvertently delivered with heat 22. However, in the report which went to J. & W. (16th March, 1959) this information was not passed on. The same heat gave welding trouble during 1959 and on 18th March, 1960 there was a further report from Port Kembla in which it was revealed again that some other heat had been mixed with heat 22. Chromium was now mentioned as it had been found by the consulting chemists, Sharp and Howell, to whom J. & W. had submitted samples for analysis. The cryptic comment is made in this report : " To completely eliminate the (mixed) material it would be necessary to sample each bar marked 267607 (C.R.B. 22) and analyse for chromium content ".

Again the evidence is that this information was not passed on to J. & W. We are therefore faced with an established fact that wrong material, which was not covered by the test certificate, had been sent to the fabricator. This possibility of mixing is well known in any steel mill. When it is discovered, every effort should be made to trace the wrong material, particularly in an important project like Kings Bridge. It was the plain duty of B.H.P. to check every plate of this heat for J. & W., but it did not do this.

We point out (Ex. 143) that there are 35 girders in the bridge in which heat 22 has been used for tension flanges at locations involving cover plate terminations.

Another matter of general policy relates to the unilateral introduction of a set of composition tolerances culled from an American source (American Iron and Steel Institute, Steel Products Manual) (Ex. 170). B.H.P. claims that according to this, the carbon maximum, which in B.S. 968 is placed at 0·23 per cent., could be 0·27 per cent.; there is nothing in B.S. 968 which suggests this. No doubt a steelmaker, who recognizes the natural phenomenon of ingot segregation and the technical difficulties of obtaining a uniform composition throughout a large mass of steel in the furnace and ladle, naturally assumes tolerances of this kind. This has been referred to in Section 2.7.2.1. But when a specification categorically fixes a maximum, presumably in this case to cover the weldability as well as the mechanical properties, we think a fabricator is entitled to expect the steel supplied to be within the composition limits set.

On several occasions during the Enquiry attention was drawn to clause 20 of B.S. 968 : 1941, which gives the customer the option of rejecting material which during fabrication is found to be out of specification. We have interpreted this clause as relating strictly to the maximum values set out for chemical composition in clause 9 of B.S. 968 : 1941. B.H.P. witnesses and Counsel strongly resisted this viewpoint and claimed the tolerances mentioned above. In view of this attitude the following extracts from internal B.H.P. correspondence are of interest :—

Ralston to A.I.S., Port Kembla, 11th April, 1958.

> " The question now arises—what reasonable variation outside the specified limits of chemistry and mechanical properties should be expected ? And, also, if any heats test on the limits of the specification, would we be prepared to make additional tests to ensure that plates supplied from these heats are in reasonable agreement with the specification ? "

Chief Metallurgist, Port Kembla, to Sales Department, Melbourne, 29th April, 1958.

> " We wish to distinguish between variations outside the specified limits and variations from the pit analysis if the latter is within the specification. We should not enter into discussions on variations from the pit analysis and the customer and the C.R.B. should be informed that with semi-killed steel such variations are quite possible.
>
> The provisions outlined above we believe should be accepted by the customer rather than his insisting on strict adherence to the specification as stated in clause 20 of B.S. 968 : 1941.
>
> For use on work which is highly stressed or on which design factors are not clearly defined we can only suggest that individual plates or bars be tested as for example in the boiler specifications which provide for testing each pattern. Penstock Boiler Quality for the H.E.C. Tasmania costing £5 10s. per ton over the standard rate for B.S. 2762 : 1956, is tested in this manner."

From the second paragraph we conclude that our interpretation of clause 20 was also accepted by the Chief Metallurgist of A.I.S. in 1958.

We also conclude that as B.H.P. was advocating strict adherence to B.S. 968 : 1941, it could hardly claim that the chemical composition, as set out in clause 9, should be subject to the American tolerances.

In view of the big quantities of this steel being diverted to less important uses, it is not difficult to understand this claim for analysis tolerances beyond the B.S. 968 limits,

but it is not satisfactory for B.H.P. to say that, "We told J. & W. of these tolerances". J. & W. was known not to have a metallurgical department and was therefore not likely to—and, in the event, did not—establish a set of check tests on the plate as received.

We think, in fact that the practice of B.H.P. in providing metallurgical service for its customers, may have had the effect of causing fabricators such as J. & W. to rely too much on them. In the long run B.H.P. can give better service by encouraging its customers to set up their own service departments. If this were done the valuable information given by B.H.P. would fall on more receptive ears and better informed minds.

This concludes our survey of those aspects of general B.H.P. policy which would apparently have applied equally to both the Newcastle and Port Kembla steel plants. However, we would point out that like many other matters at our Enquiry these matters do not fall into neat classifications. For example, whilst there was a policy relating to composition tolerances, the individual plant personnel determined what influence this should have on their practice and what limits they would adopt to obtain the specified physical properties.

2.10.3.2. *Steel plant policy and practice.*

The first matter to which we draw attention relates to the certificates of analysis supplied with the steel from each individual heat. We realize that this is the conventional way of presenting what should be the average composition of the steel, but we think that it should show all the features required to check compliance with a specification. Since it was steel works policy to work to the maximum permissible chemical limits of the specification, in order to obtain the required strength of the steel, it was necessary to know the chromium content to check the total of manganese and chromium. It is conventional with mild steel to report no more than five elements (three of which in semi-killed steel could normally be considered "residual"). This matter has already been referred to in Section 2.7.2.

Certificates from Newcastle that we have seen carry a printed column in which the chromium content can be entered; we were told that this was because Newcastle regularly manufactures alloy steels.

The only exoneration we can find for the steelmaker in this respect is that it followed an established convention.

Another feature of the steel supply, which has been mentioned earlier and which has given the Commission considerable trouble to understand relates to the decision of A.I.S., early in 1959, to change from semi-killed open-hearth steel to fully-killed electric furnace steel. This was represented to us by Ralston and Thompson as an attempt by A.I.S. to help J. & W. out of their contractual difficulties with respect to the C.R.B. specification. This view was confirmed in a letter of 8th February, 1961, from General Manager, Port Kembla, to Managing Director, B.H.P. which reads as follows:—

> "Because of the severity of the C.R.B.'s requirements, accepted by J. & W. but not ourselves, it was decided, in an effort to assist J. & W. to meet the C.R.B. demands, to produce the steel in the electric furnace. We have since produced B.S. 968 heats in the open hearth to the satisfaction of various customers but would no doubt encounter similar difficulties again if requirements additional to B.S. 968 were imposed.
>
> We would emphasize that J. & W. were told quite definitely that we were not prepared to accept requirements over and above the B.S. 968 specification. However, J. & W. encountered some difficulty in fabrication through failure to observe correct welding practice for this grade and this did not assist them in the over-all problem of dealing with the C.R.B.
>
> It would be true to say that our reason for manufacturing this material in the electric furnace was attributable to the severity of the Board's requirements and our desire to assist J. & W. as far as possible in meeting those requirements.

The failure and rejection of a full open-hearth heat on the basis of the C.R.B.'s tests would have disrupted deliveries far more than the failure of a single electric furnace heat. We were also forced to narrow our specification range and this was more practical in the electric furnace.

We also refer you to the other reasons quoted to you in our letter dated 15th January, 1960."

If these were the main reasons for the change, they represented a belated change of view on the part of the Port Kembla staff, and an admission that they were under some obligation to help J. & W. to meet the C.R.B. requirements. It was, however, a curious decision. The change to fully-killed steel was likely to result in improved notch ductility, but up to this time no difficulty had been found in this respect with the semi-killed open-hearth steel being supplied. What was really needed by the fabricator was an improvement in weldability and this required a lowering of the carbon and managanese contents. However, two of the " submitted heats " (55 and 56) were on ladle analysis at the maximum for carbon and two others (58 and 59) only ·01 per cent. below it. In addition 0·25 per cent. chromium had been added. Several of the heats examined since the failure have shown carbon contents between 0·25 per cent. and 0·28 per cent. In fact there was a marked deterioration in the properties of the steel (See Section 2.7.2.3.), both as regards notch ductility (in some heats) and weldability in general.

We believe, however, from a consideration of internal communications and from a perusal of section 11 of Exhibit 184 that the reasons for the change were largely internal and that A.I.S. had considerable difficulty in producing B.S. 968 for this contract. We were told that to supply the first 1,000 tons, " thousands of tons ", had to be diverted to other uses. This was apparently because such material did not meet the mechanical test requirements of B.S. 968 and because it was unsatisfactory for other reasons such as piping, laminations and surface defects. The latter were certainly a cause for concern by C.R.B. inspectors, even in material which was delivered, and this may have played some part in the decision to change to electric furnace production ; the result, however, was a poorer quality of steel.

The first intimation that any change had taken place appeared in a letter of 27th February, 1959, from A.I.S. relating to the " submitted heats " (See Section 3.4.1.), in which it was stated that " test failure occurred in a number of fully-killed electric furnace and open-hearth heats ".

It is noteworthy that up to that time :

(*a*) all the semi-killed open-hearth steel had passed such of the C.R.B. tests as had been applied (See Section 2.7); and

(*b*) J. & W. had not completed the fabrication of a single girder (The first girders were not completed until September, 1959).

As the change took J. & W., Utah, and C.R.B. by surprise it is evident that A.I.S. took this decision entirely independently, although neither the C.R.B. specifications nor B.S. 968 : 1941, mention electric furnace steel. When J. & W. referred these " submitted heats " to the C.R.B., the latter consulted Ferris and Hudson who both considered that the change should result in an improved steel which could be accepted if it fulfilled all other requirements. We now know that, far from ensuring that the requirements were fulfilled, the C.R.B. relaxed testing at this time.

The change to smaller heats led to Eastick and the C.R.B. deciding to resort to " random " testing of heats. (See also Section 2.7.2.3.). Eastick felt justified in this because of the verbal assurances from Ralston that fully-killed steel would be more " notch tough ", and up to this stage, the heats of open-hearth steel had passed the applied tests in this respect. The still better performances that he had been led to expect seemed to Eastick to be good grounds for relaxing the frequency of testing.

In view of all the circumstances we conclude that the decision to change from open-hearth semi-killed steel to electric furnace fully-killed steel was largely determined by internal considerations at Port Kembla. In particular, it was to reduce the amount of diversion of steel which did not comply with the requirements of B.S. 968 : 1941 and had little to do with the extra tests required by the C.R.B. specifications.

We consider that the change in steel making practice early in 1959 was detrimental to the quality of the steel supplied, and was a direct cause of the failure in that the tension flange plates in the failed girders were made from steel in heats 55 and 56 which were both made in the electric furnace.

2.10.4. Inspection of the steel.

The tests to which the steel was to be subjected comprised a chemical analysis, inspection of the surface of the plates for blemishes and defects, tests of the physical properties such as the tensile strength and yield point, the impact strength (Izod), and tests to determine the weldability of the steel. These tests are described in Specification Clauses 2–3–1. to 2–3–25.

Several of these tests have already been discussed but there remain a number of general matters which require consideration.

The question of whether the steel should have been inspected before it left Port Kembla was discussed before us at some length. It transpired that Ferris advised Wilson to get Lloyd's Register of Shipping, Land Division, to inspect the steel at the steel works and that Wilson discussed the matter with Butler who, in turn, asked for the Board's direction. Butler's letter (Ex. 32) includes the sentence " It would appear doubtful whether the Board would benefit by 10s. per ton " (a reference to the cost of Lloyd's inspection) and carried the following handwritten note signed by Wilson : " This was considered verbally by the Chief Engineer and/or the then Chairman and not agreed to. There is no official written record ". However, the C.R.B. apparently did write in 1960 to suggest the use of Lloyd's inspectors but J. & W. refused and the matter was not pressed.

We are quite clear that it would have been far better if the steel had been unambiguously identified, tested, and cleared for despatch by an independent inspector at Port Kembla works. As it was, the steel was only inspected and tested after delivery and the first requirement of a testing programme, proper identification of the specimens, was not satisfied. This not is an easy requirement to fulfil ; one piece of steel looks very like another and it is, therefore, important to have very systematic arrangements for stamping heat numbers on specimens and for recording the results. It must be realized that it was intended that " two tension, two cold bend, and two impact tests shall be made from each size of section or plates rolled from any one heat of steel or identifiable batch from stock " (Clause 2–3–15.) ; and that four bend tests, two tensile butt-weld tests, four impact tests, and a macrographic test were required for weldability (Clause 2–3–16 (*f*)). This was a formidable programme to organize and carry out. It was never fully implemented and was ultimately relaxed.

In practice, the specimens were machined by J. & W. and tested by E.T.R.S. or at Royal Melbourne Technical College in the presence of Scarlett and Jackson. A curious feature of the testing was the procedure adopted for retesting failed samples. The specification is unambiguous : Clause 2–3–15. states that "should any of the test specimens first selected not fulfil the test requirements for the material being tested, two additional specimens of the type which did not fulfil the test requirements shall be taken, and should either of these fail to fulfil such tests, all the material so represented shall be rejected." It is perhaps not surprising that J. & W., having accepted the material from B.H.P. before the tests were made, took the view that what did not pass the first time would pass if it was tested a sufficient number of times ; but it is indeed very surprising that the C.R.B. concurred.

On the other hand, tests are not prescribed without reason. Quite apart from their discriminatory value—to reject faulty material—they can be an invaluable guide to production. B.H.P. knew this and in actual fact carried out, for its own purposes, many more tests than it reported (See Ex. 186) ; but J. & W. did not realize that the weldability tests, if intelligently used, could have helped to determine fabrication procedures. Instead of this it was constantly pressing the C.R.B. to relax the testing programme.

While we do not regard the matter as of crucial importance we think it is symptomatic that many of the Izod tests at the higher temperatures were tested at " ambient temperature " and not at 70° F. as specified. With this steel the Izod value changes rapidly with temperature and so a few degrees up or down can make the difference between success or failure.

In some heats (e.g., 1, 4, 9 ; see Sec. 2.7.2.3.) the Izod value at 32° F. was higher than that at the upper temperature ; this can only be explained if the samples were wrongly identified or the tests improperly conducted or recorded, or if the steel was very variable.

The specifications are not absolutely clear on the point although it certainly seems that the intention was that the steel should have been inspected before it left the maker's works (See Clauses 2–3–16 (*h*) and 2–3–25.). Further it was not to be despatched from the maker's works or stock until Utah had certified that it complied with C.R.B. specifications. The C.R.B. also had ample powers to ensure that the steel was fully tested and approved before it was used (Clause 2–1–1.). In spite of all this, J. & W. insisted that it was nothing more than a " commercial risk " to use steel before it had been tested and Eastick apparently did not insist.

We have most difficulty of all in understanding the decision to change to "random" testing. The average tonnage of steel per heat was not greatly reduced (See Section 2.7.2.3.) by the change to electric furnace production although everyone expected the new process to give better control of steel-making. But if it was essential to reduce the number of tests, selective tests on samples shown by the certificates to be high in carbon and manganese should have been chosen, and the " submitted heats " should have been thoroughly tested.

All the matters mentioned above illustrate the advantages that could have been gained from the presence of a competent metallurgist in J. & W.'s works. In particular, we again draw attention to the reckless way in which the " submitted heats " were accepted ; the importance and significance of this matter could not have escaped a metallurgist.

We summarize our views on the inspection and testing of the steel as follows :—

J. & W.—

(*a*) having failed to order the steel to C.R.B. specifications failed to inspect it properly prior to fabrication ;

(*b*) was under pressure from Utah to complete the fabrication and erection of the girders on time and, in turn, constantly pressed C.R.B. to relax testing ;

(*c*) did not understand, and had no respect for, the importance of a sound and comprehensive testing programme.

C.R.B.—

(*a*) did relax the intensity of testing without proper thought ;

(*b*) did approve steel which did not come within its specification ;

(*c*) did not apply its full rights as to testing.

As a result of all this, several heats of steel were accepted that should have been rejected.

2.10.5. Fabrication and welding procedures.

Welding high-tensile steel is a more complicated business than welding mild steel and requires careful attention to detail if it is to be successfully carried out. J. & W. had had no previous experience of B.S. 968 steel and much of their difficulty can be traced to having to learn as it went along on a big contract with a tight delivery schedule.

It soon became apparent to J. & W. that without outside help it would never succeed and so Ward of Murex was installed " to put their house in order ". It was not at all easy to obtain, train and examine enough welders to keep the work running smoothly but we think that this difficulty was eventually overcome. We formed the impression that the welders who came before us could, with proper direction, have done a first-class job.

We do not think that instructions were given to the men in a satisfactory manner ; indeed we never really found out what, if any, was the system in use. It appears that the calculation of the necessary pre-heat was a very haphazard business, to say the least, and the pre-heat temperature eventually reached the men by word of mouth. This in itself is not necessarily reprehensible but proper records should have been kept so that the reason for any faults subsequently detected might have been traced.

The following extract from " Memorandum on Faults in Arc Welds in Mild and Low Alloy Steels ", published by the B.W.R.A. in 1950, is of interest in this connection :—

> " Full information as regards weld sizes and details, plate preparation, assembly and fit-up, welding procedure, sequence and so on should be given on the drawings, or by separate charts or other suitable means, to ensure that the works staff is fully conversant with the requirements of the drawing office, and of the planning or production departments.
>
> During fabrication, all welding should be carried out under constant supervision, by welding operators trained and tested to weld under conditions and on the type of welds appropriate to the class of work on which they are employed.
>
> It is advisable that identification and record systems be used by which responsibility for the execution and supervision of welding can be established at any time during or following completion of the work ".

On the other hand the excellent records kept by C.R.B. have been of great value to us. They have enabled us to learn the heats of steel incorporated in every girder of the bridge, and the welding sequence used in most cases. Unfortunately, on the transfer of C.R.B. to new headquarters, records relating to the work of individual welders were destroyed. Had these records been available we would have known which welders had made the welds which failed and some of the doubts we have on procedures used could have been resolved. The fabricators kept no such records.

In Section 2.9.3. we have dealt with the question of pre-heat. We were told that J. & W. would have preferred to use bigger electrodes and heavier currents but this matter was never properly explored with the C.R.B.

Metallurgical opinion is unanimous on the importance of heat in-put but, on the other hand, we have evidence that some experienced fabricators like to keep the initial temperatures as low as possible consistent with the prevention of weld cracking. This is not just a matter of expense although the cost of pre-heating is not negligible ; nor is it that a good rate of production may be maintained. The aim is rather to reduce both distortion and the production of high residual stresses. We do not presume to adjudicate on a matter that is doubtless only to be resolved by careful compromise. But we whole-heartedly agree with Eastick's view that if J. & W. had spent the period between the award of the contract and the beginning of fabrication in systematically investigating these matters and preparing themselves for the job the whole history of Kings Bridge would have been different.

Experiments conducted during the Enquiry have convinced us that the determination of the proper welding procedure for the various girder joints in this steel is not at all easy and merits much more careful study than was possible once fabrication was under way. The method used by J. & W. was just not good enough, especially when the steel delivered was as high in carbon as eventually turned out to be the case. The culminating point was reached when, towards the end of the contract (September, 1960), for reasons that are not clear, the " weldability index " was lowered for a few weeks. This meant that the pre-heat temperature would be lowered. It was apparently during this period that the W. 14 girders, which eventually failed, were fabricated. As we now know, the carbon content of these plates was exceptionally high so that the pre-heat temperature should have been raised rather than lowered. We consider that the resulting inadequate pre-heat was one of the causes of the toe cracks at the cover plate end welds in the failed girders.

Two further illustrations will suffice to demonstrate J. & W.'s unpreparedness to undertake the contract : if pre-heat is to be used properly the temperature of the steel must be checked but J. & W. welders had no means of doing so. It was only when Clarke told

them about temperature-sensitive thermocrayons that they were able to check pre-heat temperatures at all. Finally, in this context, we refer again to the fact that it was only on Ward's advice that J. & W. installed hot boxes in which to store and dry the electrodes before use.

We now turn to the incident of the 500-ton press, which occupied more time at the Enquiry than it deserved, but which does illustrate rather vividly the attitude of some of the parties. It came to light only because of the rather underhand activities of Fisher who had been installed by Utah in the J. & W. works as an observer. We do not think very highly of Utah for this manœuvre, even though Fisher apparently went beyond his brief in his enthusiasm to detect and report any misdemeanours on the part of J. & W., and we commend Bonwick for putting the arrangement on a proper basis.

It seems that J. & W. was having difficulty in ensuring that the camber of the completed girders was in accordance with the drawings. It occurred to Stocker that if he were to subject them to a suitable force in a big press he could bend them into the correct shape. C.R.B. was asked for its approval but, before this was received (complete with a number of precautions that were to be observed), Stocker conceived the idea of a private trial by night. The idea was, no doubt, that if it succeeded C.R.B. would accept the procedure as having been justified by the result. No measurements of the strain were made, nor any thought given to the consequences of cold work on steel of this kind, but apparently the attempt was abandoned before it went very far.

So far as J. & W. is concerned we conclude as follows :—

(*a*) The J. & W. organization, particularly at senior level, was not adequate to undertake the construction of these girders in B.S. 968 steel.

(*b*) Under the strict control of C.R.B. inspectors and with the help of Murex most of the technical difficulties were subsequently solved.

(*c*) Inherently, however, it never became a competent fabricator of low-alloy steel because of the failure to appreciate that the troubles encountered were largely metallurgical in origin.

(*d*) Within its competence, and working under adverse conditions created by C.R.B. inspection and Utah production pressure, it did its best.

(*e*) These adverse conditions arose from lack of knowledge of the steel being fabricated, but they were not helped in their adversity by the similar lack of knowledge of the C.R.B. officers who were in effect their task masters.

(*f*) Had it not been for the close C.R.B. inspection the bridge would have been much worse than it was.

The question of the inspection of the girders by C.R.B. is discussed in Section 2.10.6 below and we now turn to the attitude of Utah to the difficulties encountered by their sub-contractor. We are not concerned here with the question of legal liability, but simply with whether Utah should have intervened or could have done so to good effect.

In actual fact they did practically nothing beyond transmitting letters from C.R.B. to J. & W. and vice versa. They did take action, on occasions, to try to iron out differences but they were deliberately careful not to become closely involved themselves. Having awarded the sub-contract to J. & W. who were, so far as anyone knew at the time, competent and experienced fabricators they left them to get on with it.

The Utah engineers concerned were Fink and Bonwick, whose work in this context was described as follows: "The most important thing as far as we were concerned was to administer the sub-contract in respect of payment, measurement of work, timing of when girders were going to arrive, scheduling, programming, and so on, and give them any technical assistance we could give them if they asked for it".

It is convenient at this stage to list a number of significant dates :—

13.8.57	..	Contract signed between C.R.B. and Utah for bridge construction.
4.9.57	..	J. & W. given contract for fabrication by Utah.
19.9.57	..	Work began.
11.11.57	..	J. & W. approved by C.R.B. as sub-contractors for fabrication.
20.5.58	..	First steel ordered by J. & W. from B.H.P.
—.8.58	..	First steel delivered at J. & W. (earliest test certificate is dated 9th July, 1958).
15.8.58	..	Scarlett nominated as Utah's representative at J. & W.
5.9.58	..	Letter J. & W. to Utah saying (erroneously) that B.H.P. would do the Izod tests in future.
2.3.59	..	Unsuccessful demonstration of automatic welding.
10.3.59	..	Letter J. & W. to Utah undertaking to do Izod tests.
24.3.59	..	Acrimonious conference at J. & W.
14.4.59	..	Conference between Utah and J. & W. Difficulties reviewed and action agreed.
7.8.59	..	Letter Utah to J. & W. saying that sub-structure was ready for 105 girders but that none had been delivered.
7.9.59	..	First girder erected.
29.11.59	..	Letter J. & W. to Utah asking for £334,000 extra in view of high standard of work and excessive inspection required by C.R.B.
30.11.59	..	Letter Utah to J. & W. rejecting claim for extra payment.

A perusal of the correspondence makes it abundantly clear that throughout 1959 J. & W. were in great difficulties to get the job going. By about July, 1959, Stocker was advising J. & W. to stop the contract but the principals of the firm honourably decided that having set its hand to the contract, it would see the matter through.

The pathetic aspect of this correspondence is that J. & W. were continually saying that all would be well if only the C.R.B. would relax their control over welding procedures and the severity of their inspection; it never seems to have occurred to J. & W. that this control and inspection was essential to the satisfactory execution of the job.

It will be clear from the above dates that long before the first girder was completed Utah was very well aware that its sub-contractor was in very serious trouble.

It transpired at the Enquiry that in April, 1959, Fink wrote to a friend of his, Mr. C. C. Winter, of the California Department of Public Works, to ask about welding high-tensile steel; he received the following suggestions :—

1. Set up rigid tolerances and procedures and follow them.
2. Moisture is critical.
3. Surges in power are critical.
4. Rigid inspection.

As a further suggestion: "Have someone from your company visit and talk to the American Bridge Company or the U.S. Steel Company". This was indeed good advice but neither C.R.B. nor J. & W., to whom it was shown, nor Utah themselves did anything about it and it was not until the end of January, 1960, that Bonwick was sent to the U.S.A. and the U.K. to "obtain answers to the following three main questions :—

(*a*) What methods are used for the fabrication of welded low-alloy steel plate girder bridges?

(*b*) What are typical costs and man-hour production rates for this type of work?

(*c*) What standards of inspection are adopted by the different authorities for this work?"

By the time he got back in early March about 90 girders out of a total of 277 had been erected so that the valuable information he obtained was really too late to be effective.

Utah representatives at the Enquiry more than once said that in view of the close supervision of the work by C.R.B. and the fact that J. & W., whatever their deficiencies, knew more about welding than they did, any intervention by Utah would have confused an already difficult situation. We agree; the only effective action Utah could have taken

would have been to bring over from America, early in 1959, a thoroughly experienced welding engineer who had been used to working to the requirements of the California Division of Highways or some other similar State authority. This man would have had to be put in charge of the whole operations at J. & W.

We do not know whether action of this kind was considered by Utah at the time although Fink said in evidence that he did not think it a good idea; he did say, however, that Utah had power under the contract to intervene and supervise J. & W.'s work.

We are convinced that nothing less would have sufficed to bring J. & W. up to the required standard in time. As it was, by the time Bonwick got back from abroad the combined efforts of Ward, the C.R.B., and J. & W. themselves seemed to have been successful and completed girders were moving steadily out of the shop.

There was, however, one occasion when Utah could possibly have obtained some useful information and failed to do so. In July, 1959, it came to their notice that a bridge had been constructed of B.S. 968 steel at Glen Quoich in Scotland and on the 14th a cable was sent to Sir Wm. Halcrow & Partners of London asking for information. The reply referred Utah to Sir Wm. Arrol & Co. of Glasgow, who designed and constructed the bridge, but no further action was taken.

Our conclusions about Utah are, therefore, as follows:—

(*a*) They did not come up to the expectations of the C.R.B. by bringing American experience to bear on the fabrication of the steel work.

(*b*) They did not realize that J. & W. were too inexperienced in this class of work to be given the sub-contract. (Their appointment of Scarlett as their representative is clear evidence of this.)

(*c*) They could only have improved the performance of J. & W. by virtually taking over control of the J. & W. shop by importing expert staff.

(*d*) Their attitude to the whole question of fabrication was disappointing.

2.10.6. Inspection of the welding and fabrication.

The correspondence referred to in the previous section makes it abundantly clear that C.R.B. not only inspected the finished work but also insisted on their right to approve every detail of the fabrication and welding procedure. They were certainly entitled to do so but they were no more competent than J. & W. in the special problems of handling B.S. 968 steel because their only prior contact with it was when Jackson went to D.S.L. for training (Section 2.4.3).

One can readily imagine, therefore, the state of mind engendered in the J. & W. personnel when these C.R.B. men, themselves without appreciable knowledge or experience of the steel, set themselves up as the final arbiters in all welding procedures.

The inspectors were, we believe, expert welders but they had had no experience of inspection; nevertheless they had the final authority even though, like J. & W., they were learning on the project.

To add to J. & W.'s difficulties, Utah was continually pressing for delivery of the girders, but not otherwise contributing to the progress of the project. The fabricator's staff must have been a frustrated group of men; they were trying to handle a steel which was variable and not always in conformity with the steel-works certificates; the controlling authority was applying very high standards; and the contractor was wanting his girders. This was not a congenial atmosphere for a great experiment in a new material.

Inspection is an art which involves knowledge, skill, and personality. There is nothing more calculated to rouse hostility amongst workmen than an inspector who is so unsure of his own knowledge that he interprets his instructions literally.

From the evidence it would seem that the C.R.B. staff virtually took over the details of girder fabrication in J. & W. shops. They themselves had no knowledge of the fabrication difficulties with this steel and they seem to have "stood over" J. & W. personnel, demanding the submission of every detail of procedure, A typical case arose in connection with the use of automatic welding. This was considered by C.R.B. personnel to be a departure from standard practice, which to them meant manual welding. Certainly,

early efforts to produce a satisfactory automatic weld were not successful but C.R.B. officers appear to have been critical rather than helpful. When it seemed that success was in sight, Eastick adopted the attitude of a disciplinarian regarding stops and starts of the welding machine.

From an early stage in the contract C.R.B. officers appear to have doubted the technical competence of J. & W. to construct girders in this steel and, having no knowledge of their own, they adopted the unhelpful attitude of criticism. This naturally created a feeling of resentment in the J. & W. organization and the mutually uncooperative attitude seems to have been a feature throughout the contract.

For instance, there was a suggestion that J. & W. introduced night work at short notice in order to get on with some welding when the inspectors were not present. We think that the exigencies of the production schedule probably demanded night work, or at least overtime, quite often but the fact that such action could be interpreted as an attempt to outwit the inspectors is evidence of an unsatisfactory relationship.

C.R.B. seems to have lost sight of the possibility of brittle fracture at an early stage. In fact there is no evidence that anybody in the C.R.B. organization gave this a second thought. Consequently, even C.R.B. officers went through the motions of Izod testing quite mechanically and with no thought that it was of critical importance. In this they were at one with J. & W.

We now consider in turn several specific issues: the first is the question of the weldability tests. The specification for these was based on B.S. 2549 : 1954, which deals with electrodes for welding steel to B.S. 968 : 1941. In this specification the method of preparing test pieces described requires the temperature of the parent plate to be between 50° F. and 85° F. when the test welds are started.

For reasons that we were not told but which, for all we know to the contrary, may have been entirely sound, the tests of B.S. 2549 were used by the C.R.B. to examine, not the electrodes but the steel. Unfortunately, there was no prescription of the initial temperature of the parent plate and, in consequence, this became a matter of dispute between C.R.B. and J. & W. The weldability test specimens were prepared by J. & W., welded by Murex and finally tested by E.T.R.S. or the Melbourne Technical College; on 24th November, 1958 the Melbourne office of Murex wrote to the research department at Hobart as follows:—

> "In most cases the tests were very satisfactory, but you will notice that the Izod values are low on some of the tests, so therefore we have to carry out a retest on this particular plate." The reply from Hobart included the following: " . . . it would appear that the trouble is confined to the Izod specimens when the notch is located in the heat-affected zone. As we have pointed out before, this is entirely a thermal effect and can be expected with the heavier section plate material.
>
> We understand that pre-heating is not allowed by the C.R.B. inspectors. There is little else than can be done to slow down cooling right in the heat-affected zone . . . ".

We never did discover with any precision what was done about pre-heating the weldability test specimens although Scarlett, in evidence, implied that the C.R.B. inspector present at the welding fixed the pre-heat. We are, however, satisfied that it occurred neither to C.R.B. nor to J. & W. that these tests might have been used to help to determine the welding procedure.

There is, however, plenty of evidence that Clarke, and the inspectors under him, gave a great deal of attention to such matters as the use of thermocrayons and the general "housekeeping" at J. & W. and they certainly looked for, and found, large numbers of weld defects which were subsequently repaired, re-inspected and passed. This is the basis of J. & W.'s claim that every defect that was found was repaired to the satisfaction of the C.R.B.

The records of work of the inspectors are contained in the three volumes of Ex. 103 (Fillet weld survey sheets), in Ex. 98 (File of official memoranda from Clarke) and are summarized in Ex. 222. This evidence certainly gives the impression, at first glance, of a very strict and meticulous inspection. However, the fact remains that a number of things got past the inspectors that have subsequently come to light.

The design specifications prohibited transverse welds in tension flanges except in regions where the stress was at a sufficiently low value. Accordingly the gusset plates to which the cross-bracing was attached were shown on the drawings to be welded to the flanges by longitudinal welds only. Nevertheless it was found later that transverse welds had also been used on the western girders over City-road, and Hudson, in his examination of the bridge after the collapse of span W.14, found that a number of these welds gave indications of cracks. He also found indications of cracking at various locations on the 85 girder ends that he examined.

The incidence of cracks at the cover plate ends is described in Section 2.9.6.1 *et seq.* but the question of whether this location, which is now known to have been of critical importance, should have been the subject of a special examination remains to be discussed.

The C.R.B. inspectors clearly had no instructions to give it particular attention and regarded it in the same way as any other fillet weld. The butt welds on the tension flanges were very properly given a very close examination and it is surprising that similar reasoning was not applied to the cover plate end welds.

We think that they should have been picked out for special examination by the C.R.B. when the intensity of inspection to be imposed was being settled.

We have commented, in Section 2.6.3, on the designer's role in specifying welding details and we believe that similar arguments apply to the interest he should take in inspection.

The counter argument is that defects, sometimes resulting in serious consequences, are not infrequently found in comparatively unimportant details, the attachment of lugs for supporting pressure vessels, for instance, and tack welds have been mentioned to us as features which are sometimes too lightly regarded because of their seeming unimportance. From this standpoint the C.R.B. attitude of regarding all welds as of equal significance was correct.

We do not agree with this view. Any system of inspection that is not 100 per cent. complete involves selection and among the criteria for selection must surely be the possible consequences of a crack remaining undetected. As soon as this has been said it becomes obvious that transverse cracks in tension flanges must be sought with special care and eliminated.

The final inspection apparently often took place with workmen swarming over the girder putting the finishing touches to the work. It seems that J. & W. insisted that production should not be delayed because inspection and testing were incomplete and maintained that any defects in the girders would eventually be corrected; they regarded it as merely a commercial risk that this might involve removing completed girders from the bridge. As a consequence the final inspection was carried out in very unsatisfactory circumstances and it is quite likely that the cracks which eventually caused the bridge to fail were missed for this reason.

We hardly know whom to blame the more, J. & W. for its cavalier attitude or C.R.B. for putting up with it.

Our conclusions about the inspection of the welding are as follows:—

(*a*) C.R.B. inspectors were inexperienced in the inspection of this class of welding and therefore adopted what they regarded as safe criteria.

(*b*) J. & W. did not realize that, with this material, meticulous inspection was essential.

(*c*) In consequence there was continual bickering about the standards of inspection.

(*d*) The C.R.B. inspectors missed a great many cracks that have since been found.

(*e*) Many cracks were found and repaired.

(*f*) The standard of inspection was high but not high enough.

PART 3.—THE COLLAPSE OF THE W.14 SPAN OF THE BRIDGE.

In the preceding narrative we have attempted to present a picture of the relationships between the parties concerned and in particular of the conditions prevailing in the fabricator's organization and workshops.

We now propose to consider in detail the nature of the failure, using the information acquired during the Enquiry, including the results of the former Committee of Investigation and the investigations which have been made for us by the Defence Standards Laboratories and the various parties concerned.

3.1. Characteristics of the Failed Girder W.14–2.

This was the second girder from the west side of the elevated carriageway and its general characteristics can be seen in the accompanying photograph, Fig. 2, which is taken looking south. The cover plate end is therefore not seen. To complete the picture and to show the relationship with the cover plate, reference is made to Fig. 3 which is a photograph of the southern end fracture (looking north) of girder W.14–1, and to photograph Fig. 4 of girder W.14–4 (north). These two show typical cover plate endings and the association of the fracture with them. These three photographs are typical of all the fractures in a general way. They differ in detail.

Fig. 2 shows the following characteristics :—

(*a*) A brittle (i.e., non ductile) fracture of the lower flange and the web.

(*b*) A dark area (A.) in the centre of the flange, in the neighbourhood of the web/flange fillet welds at the toe of the cover plate end weld.

(*c*) A dark area (B.) in the web near the top ot the photograph. This is a fatigue fracture which serves to establish the condition of this girder prior to the final collapse on 10th July, 1962.

Fig. 3 shows a typical cover plate end transverse weld. It can be seen that the fracture in the flange starts at the toe of this transverse weld, of which two of the three weld beads can be seen. The root bead is of course covered.

Figure 4 shows the longitudinal weld of the cover plate to the flange and the welding at the end of the cover plate. It can be seen that the taper weld started after gouging back 3 or 4 inches into the longitudinal weld. Two weld beads are evident.

Figure 5 shows in more detail the nature of the fracture in W.14–2 at the south end. From the bottom is seen—

(*a*) the cover plate and two of the transverse weld beads.

(*b*) a dark area which in effect is composite. First the toe crack about $\frac{1}{8}$ inch deep, then a primary brittle fracture, rusted, and terminating at the two web/flange fillet welds showing that these welds arrested the first fracture.

(*c*) The web/flange welds.

(*d*) The web and flange secondary brittle fracture (rusted).

We consider this girder fractured through the flange to point B. (i.e., 44 inches up the web) in Fig. 2, months before the final collapse, because it was the only fracture which was rusted.

In trying to fix the time for this fracture we have several significant features :—

(*a*) There is no indication of the final coat of paint which was applied during January, 1961. This coat, having an aluminimum base was readily distinguishable, and was in fact found in at least one fracture (W.14–3 south end). There is little doubt also that the fracture would have been so obvious that the painter would have noticed it. Nobody has mentioned this to us.

(*b*) The surface of the fracture is rusty. It is considered that this would have required several months to reach the condition noted at the time.

(*c*) The fatigue fracture at B. (Fig. 2) indicates a response to traffic loads because of the presence of characteristic markings showing the progress of fatigue.

(*d*) The temperature at the time of the failure (11 a.m. 10th July, 1962) was about 45° F. following a minimum of 36° F. during the previous night. The only lower temperature to which the bridge was subjected was in 1961, when a minimum of 33° F. was recorded in July.

The only definite limits for the time of fracture of W.14–2 are therefore January, 1961, and July, 1962, with a possibility, judging by the rust and the low temperature that it occurred in July, 1961. It is possible, therefore, that this fracture was present on 18th October, 1961, when the bridge was taken over by M. & M.B.W. If there had been an inspection this fracture, if present, would have been noticed.

When the actual fracture is examined it is evident that the progress of the fracture up the web took place in several stages.

3.2. The Condition of the W.14 Span just prior to the Collapse.

Before considering the failure in detail, and trying to trace the ultimate causes, it is appropriate to describe the condition of the failed span, and in fact of the whole bridge as we now know it to be. This is illustrated by reference to Fig. 6. The following features are noted :—

(*a*) At all seven cover plate transverse welds where the main fractures were found there was a toe crack about $\frac{1}{8}$ inch deep x 3 inches long.

(*b*) Red primer paint had penetrated six of these toe cracks.

(*c*) In the following positions the toe cracks had developed into primary brittle fractures which had passed through the flange plate : W.14–1 (south), W.14–2 (north), W.14–3 (south), whilst at W.14–2 (south), a similar fracture had developed but had been arrested by the fillet welds at the web.

(*d*) Red primer paint had penetrated three of these brittle fractures from the top side of the flanges : W.14–1 (south), W.14–2 (north) and W.14–3 (south).

(*e*) The final coat of aluminimum paint penetrated also the primary brittle fracture in W.14–3 (south) from the top side of the flange, showing that this fracture had broken through the primer coat, and the second coat of paint which was applied after the girder had been erected. This means that this fracture was opening up under traffic loads before January, 1961.

(*f*) Sometime after January, 1961, i.e., after the final coat of paint was applied, the primary brittle fracture at the south end of W.14–2 passed through the two fillet welds which had first arrested it and stopped 44 inches up the web, leaving only a few inches of the web intact. The primary brittle fracture at the south end of W.14–3 about the same time spread 4 inches up the web. It is suggested that these extensions occurred during the winter of 1961.

(*g*) During the succeeding year the south ends of W.14–2 and W.14–3 were flexing under traffic loads to an extent which resulted in fatigue fracture developing for about $\frac{1}{2}$ inch further up the web in each case.

In addition the north end of W.14–2 had a fracture 1 inch up the web as seen in Fig. 7.

It can be seen, therefore, that during the twelve months from about July, 1961, to July, 1962, these two girders (W.14–2 and W.14–3) were carrying no appreciable part of the traffic load.

The ultimate collapse of this span was therefore inevitable for the condition was getting continually worse due to the presence of fatigue fractures in these two girders.

At this stage it is appropriate to comment on the situation in the remainder of the bridge as it has been disclosed during the Enquiry.

There are at least two other locations where the flange plates are fractured through at the transverse weld, though as yet these are partial fractures similar to those already described (See (*c*) above). These locations are span E. 11/S4 over City-road and E. 6/S2 over Whiteman-street.

In the course of reconstruction many cover plate ends have been milled away and the locations examined for cracks similar to those which caused the failure. Without going into detail it can be said that toe cracks at this weld were frequently found and a few of them had evidently developed into partial brittle fractures which were $\frac{3}{8}$ inch to $\frac{5}{8}$ inch deep in the flange plate which was only $\frac{3}{4}$ inch thick.

These points are mentioned, not because they had any direct connection with the failure, but in order to indicate that the condition which caused span W.14 to collapse is not by any means unique in the structure as a whole.

3.3. What is to Be Learned from the Fractures.

3.3.1. The significance of the toe cracks at the transverse welds.

It is known in welding circles that this kind of crack is more liable to occur in low-alloy steel than in mild steel. This is one of the features included in the term " weldability ", and means that to avoid defects of this kind it is necessary to take certain precautions when welding low-alloy steel. The higher the carbon, manganese and chromium in any given plate of steel the more careful must the welder be.

The factors a competent fabricator would take into account are :—

(*a*) The use of the correct temperature for pre-heating the location of the weld, taking into account the gauge of electrode to be used, the composition of the steel and the geometry of the joint.

(*b*) The type of electrode to be used, including its composition and the composition of the flux covering.

(*c*) The drying of the electrode immediately prior to use.

(*d*) The fit of the two plates to be welded, any gap between the two being detrimental.

(*e*) The restraint placed on the weld during cooling, this being largely determined by the geometry of the particular joint and the welding sequence used.

It will therefore be seen that a complex set of circumstances was involved and we have no means of sorting out the part played by these factors other than by trying to interpret the evidence.

We have reason to think that at the time these W.14 girders were fabricated there had been a reduction of pre-heating temperatures. J. & W. was never impressed with the need for pre-heating and at the end of September, 1960, it issued instructions for the weldability index to be lowered. This meant a lower pre-heating temperature. As previously mentioned (Sec. 2.7.3.) the steel used for these girder flanges was out of specification—being high in carbon. The pre-heat temperature should therefore have been raised, rather than lowered.

The electrode used for the manual welding of the end of the cover plate was a low-hydrogen electrode Fortrex 35 designed for welding low-alloy steel. It has been established that the welds were actually made from such electrodes. The maker, however, recommended that such electrodes be " stored in a warm place and re-dried at 150° C. for 30 minutes immediately before use ". The object of this was to remove any moisture which may have been absorbed from the atmosphere.

There is evidence that about the time the W.14 girders were fabricated there was some slackness regarding the shop practices relating to electrodes. This is one of the features of low-alloy steel welding which J. & W. had to take into account for good fabrication. It had not been used to this refinement but it was introduced by Ward of Murex in 1959. Our reasons for thinking that this may have had a bearing on the failure are as follows :—

(*a*) Clarke (C.R.B.) in his diary mentioned several occasions in September and October, 1960, when he issued warnings about electrodes not being tidied up and welders using cold electrodes.

(*b*) At no time during the Enquiry has J. & W. mentioned for how long the electrodes were heated. We presume therefore that this feature of the welding was left to the judgment of individual welders.

(*c*) With a view to assisting the Commission, A.I.S. carried out some experiments in which it was indicated that all moisture was not driven off from the electrodes until a temperature of 350° C. was used.

The importance of this drying is that moisture from any source decomposes in the welding arc and releases hydrogen which is absorbed by the weld metal. This is one of the most important causes of the type of cracking found at the toes of the transverse welds.

It is in this connection also that the " fit " of the plates being welded is of importance. The gas used for the pre-heating burners was either acetylene or propane. Each of these produces water which at first condenses on the cold metal, but evaporates as the temperature rises. If there is a bad fit, water can condense between the two plates. This could be a source of hydrogen. We considered this as a possible cause of the failure, but finally rejected it for the following reasons :—

(*a*) This source of hydrogen could only affect the root weld bead whereas the toe cracks causing the fractures occur at the second bead run, i.e., after all access to any cavity between the plates had been closed.

(*b*) The fit of the plates at W.14–2 (south end), where the first failure occurred, was particularly close.

Cracks are caused by local stresses acting on material which is too brittle to yield, or which is restrained from yielding because stresses are acting in several different directions. These stresses are due to the contraction during cooling after welding and are said to be " internal stresses " because they develop inside the weldment without any external load being applied.

These W.14 girders were fabricated in such a way that there was a maximum of restraint on this transverse weld. It was, in fact, the final weld done on the girders. It was restrained by the two taper welds at the end of the cover plate and by previous welds running at right angles to it on the opposite side of the flange plate, i.e., the web/flange welds. No record is available for many of the girders, particularly those constructed in the early stages of the contract, but at least half were welded in the manner of the W.14 girders. Clarke said this was done because manual welders were not available at the time when the automatic cover plate flange longitudinal welds were completed. In other words there was no rational or pre-determined sequence for this weld—to which nobody attached any special importance—it was just a matter of shop expediency.

From these considerations we conclude that the following factors were concerned in causing the toe cracks which, being stress raisers, set off the train of events which led to the failure of span W.14 :—

(*a*) The pre-heating of the weld location was insufficient because the carbon content of the steel was higher than was known at the time, and the pre-heat had been lowered by decision.

(*b*) The electrodes may have been insufficiently dry due to slackness in shop practice.

(*c*) The restraint on the transverse weld caused by the sequence of welding used.

We cannot, however, say whether one of these factors was more important than the others.

3.3.2. The cause of the primary or partial brittle fractures.

Any sharp re-entrant angle in the steel causes a sharp concentration of stress at the leading edge whether the stress is of internal or external origin. The toe cracks mentioned constitute a defect of this type. The ability of steel to resist such stress concentration is known as " notch ductility " or " notch toughness " (See Sec. 2.7.1.2.). It is an intrinsic property of a given piece of steel, and involves considerations of micro-structure, hardness

and chemical composition, including nitrogen and hydrogen content. The precise degree of notch ductility revealed is, however, also a function of the geometry of the part in the neighbourhood of the notch, of the size of the weldment and of the rate of application of the external load. The essential characteristic is the presence of a notch—in this case the toe cracks.

In our thinking on this feature of the failure we have been influenced by the appearance of the partial brittle fractures in the failed girders. They are always discoloured and generally rusty. This discolouration we think was caused by the phosphoric acid used for cleaning the weld prior to painting and if this is so the fractures must have been present at this stage. These W.14 girders were painted in the fabrication bay (Hardcastle Tr. 2070) and from the description we have had of this operation it is evident that little time would elapse between the finish of manual welding at the end of the cover plate and the application of acid for cleaning. This latter was left on for about half an hour.

These partial brittle fractures occurred therefore almost certainly within two hours of the welds being made. There is evidence also that internal cracks were formed in the fillet welds connecting the tapered part of the cover plate end to the flange as well as the transverse weld itself. The former cracks have not propagated by brittle fracture so far as we know and this inclines us to the view that local internal stresses, probably aided by handling stresses as the girders were turned over for painting, were most severe at the transverse end and caused the cracks there to develop. Scarlett (Tr. 1480) said there was much flexing of girders during handling.

We have asked ourselves the question, " Why did these partial brittle fractures occur at five of the eight welds and not at the other three ? " No satisfactory answer has been found. Four of them, including three of the worst fractures at this stage, involved heat 55 which had a bad record for notch ductility when tested. The fourth occurred in heat 56 which has given mixed notched bar results on the whole, but never so bad as those from heat 55. It is possible that these erratic results in heat 56 are due to strain ageing but we are not satisfied that this is the answer. We note that heats 55 and 56 were rather higher in nitrogen (·008 per cent. to ·011 per cent.) than is fixed as an upper limit in the 1962 amendment of B.S. 968. We cannot rule out the possibility that some mixing of these two heats has occurred during fabrication.

We have also considered the possibility of hydrogen embrittlement arising from the pickling process to which all plates were subjected. We are aware that some authorities consider that there is a permanent effect resulting from the absorption of hydrogen. We have, however, rejected this possibility because the condition would be found in all plates, and because at the location of the fractures the steel had been heated by the welding operation. Hydrogen in this sense must be distinguished from hydrogen introduced by the use of undried electrodes. This latter hydrogen is confined very locally to the heat-affected zone of the weld and may be partially responsible for the toe cracks.

We conclude that the primary brittle fractures in these girders were due to the brittle nature of the steel. They were " triggered " by the cracks present at the toes of the transverse weld at the cover plate end. The stress needed to cause the brittle fractures was partly internal (resulting from the close restraint of this weld) and partly external due to the handling of the girders for painting. This latter feature was probably the variable which decided whether partial brittle fractures occurred or not, coupled with the notch ductility of individual plates of steel.

Whilst we cannot satisfy ourselves that it had any part in the failure we would draw attention to what we consider a bad practice, and one fraught with danger. This is, the method used for cleaning welds prior to painting, by swabbing with either 15 per cent. hydrochloric acid or 15 per cent. phosphoric acid. The practice was to leave this acid in contact for half an hour or so. Under such circumstances the acid would penetrate into any cracks which had remained undetected and would generate hydrogen which would be absorbed by the steel in the immediate neighbourhood. Thus the cracks would be surrounded by embrittled steel, and this might lead to brittle fracture due to handling stresses. Another bad feature, particularly if hydrochloric acid is used, is that the corrosion products formed would be a continuing source of corrosion even under the paint.

3.3.3. The presence of paint in the fractures.

At the time of the collapse there were rumours that some of the fractures had been filled in with red lead which had then been painted over. The implication was that cracks had been found in the finished girders by the fabricator and there had been an attempt to conceal them. Examination of the fractures shows that nothing of the kind could have happened. We have no evidence that anyone noticed these cracks which must have been present before painting. They were sufficiently wide for the priming coat of paint to penetrate into some of the toe cracks and partial brittle fractures. They could not have escaped detection if the penetrant dye method of inspection had been used. As shown elsewhere, the painters were usually on the heels of the inspectors and any final inspection was done under very adverse conditions.

The W.14–3 south fracture is particularly interesting. It is shown in Fig. 8. The priming coat of paint penetrated from the upper surface of the flange, indicating that at this stage a brittle fracture had developed completely through the flange and fillet welds (web/flange) and was in fact wider than the 3-in. transverse weld. This fracture was, at this stage, more advanced than that in girder W.14–2 (south) which ultimately was the first to fail completely.

The W.14–3 south fracture also contains the final coat of aluminium paint which penetrated from both sides of the flange. This means that owing to the flexing which occurred during transportation and erection this crack had opened again through the priming paint and was visible. It must also have been flexing under traffic loads between November, 1960 and January, 1961, because the second coat of paint applied after erection must also have cracked through.

The evidence of Clarke (Tr. 957–8) was that if these end welds were done last it was not the general practice to examine them for cracks by the penetrant dye method. Such an inspection should have been made. We have been told by witnesses from K.S.B.D., C.R.B. and J. & W. that nobody gave a second thought to this weld as being of any special importance. We cannot agree with that view—(*a*) because a sharp change of section in a flange creates a position of stress concentration, and (*b*) because it is a transverse fillet weld on a tension flange. We think, therefore, that special attention should have been drawn to it.

We conclude that the transverse welds of the W.14 girders were not adequately inspected for cracks and that this omission is a direct cause of the failure of the W.14 span of the bridge. We find that failure to inspect these welds was due to a combination of circumstances :—

(*a*) No particular attention had been drawn to the importance of these cover plate end welds.

(*b*) The inspection of final details on the girders was carried out in such an atmosphere of hurry and bustle that no inspectors could be expected to do this important work satisfactorily.

(*c*) For the same reason, that is, speeding to complete the painting operation, a sufficient period was not allowed for cracks to develop. It is accepted that cracks of the kind found in the failed girders can occur up to two or three days after the welding operation. We consider therefore that painting should not have started until this lapse of time had occurred. The fact that, in the circumstances, the cracking of these welds occurred within an hour or two of the end of welding is not relevant to this argument.

In our opinion the parties responsible for the lack of inspection which should have discovered these cracks are—

(*a*) K.S.B.D. for not realizing and drawing attention to the importance of this weld.

(*b*) J. & W. for creating the circumstances which made adequate inspection impossible.

(*c*) The C.R.B. for not insisting on adequate time elapsing between the final weld and the painting operation, and on adequate facilities for final inspection being provided by the fabricator.

3.4. The Chemical Composition and Physical Properties of the Steel in the W.14 Girders.

Since the collapse numerous samples have been taken from the failed girders in order to find any anomalies which would help to decide the cause of the failure. To clarify the situation at this stage we repeat that the immediate cause of the collapse was—(*a*) the presence of toe cracks at the transverse weld at the end of the cover plates; (*b*) the development from these of partial brittle fractures. It is our opinion that the toe cracks should not have been there if suitable welding practices had been employed, and should not have developed into brittle fracture if suitable steel had been used. In other words we agree with the opinion expressed in American Welding Research Council Bulletin No, 16, of November, 1953 (p. 41) :—" The arresting of a crack is primarily the responsibility of the material; the prevention of crack initiation on the other hand depends on a combination of geometry, fabrication and material ".

It will be noticed that the steel enters into both aspects. This is because the chemical composition of the steel determines the welding characteristics and response to stress is determined by the physical characteristics.

We now proceed to examine the material of the failed girders from these points of view.

3.4.1. The chemical composition.

As previously explained the steelmaker defines the chemical composition of a heat by the analysis of a test ingot poured part way through the teeming of the steel into ingot moulds. To simplify the consideration and in our opinion without in any way detracting from the treatment of the evidence, we will confine our attention to the material of which the lower (tension) flanges of the collapsed span were fabricated. We make this statement because, in our view, once these flanges had fractured through, the ultimate collapse was inevitable.

Only two heats of steel are concerned in the tension flanges of the four girders. These are the fully-killed electric furnace heats 7890 (C.R.B. No. 55) and 7892 (C.R.B. No. 56) made at the Port Kembla plant of A.I.S. For convenience these will be designated by the C.R.B. Nos. 55 and 56.

The cast analyses of these two heats are as follows :—

Heat	Carbon (C.)	Phosphorus (P.)	Manganese (Mn.)	Silicon (Si.)	Sulphur (S.)	Chromium (Cr.)
55	·21	·025	1·70	·30	·022	·23
56	·23	·017	1·58	·19	·026	·24

From the point of view of weldability the significant elements are carbon, manganese and chromium. Attention will be paid later to some more detailed analyses which we had made to help solve some of the problems. The B.S. 968 specification defines the composition of this steel as follows :—

Carbon (C.)	·23	per cent. maximum
Manganese (Mn.)	1·8	,, ,, ,,
Chromium (Cr.)	1·0	,, ,, ,,
Manganese + Chromium	2·0	,, ,, ,,

From this it is seen that on the cast or ladle analysis heats 55 and 56 fall within the specification. Heat 55 is on the upper limit of manganese plus chromium and heat 56 on the upper limit of carbon. For reasons already discussed (Section 2.7.2.1) analysis of the plates delivered to the fabricator may vary appreciably from that of the ladle sample shown on the analysis certificate. The reasons for this are well known. Some depend on

the skill of the steelmaker, others are just natural phenomena to be coped with. It is, however, essential for the fabricator to understand these matters and for the steelmaker by his skill to minimize the difference between what he purports to supply and what he actually does supply.

Since we have to deal with the facts of the case analyses have been made on material taken from various parts of the failed girders. The results of these investigations are summarized in Tables 5 and 6 and in Appendix 5.

It will be recalled that heats 55 and 56 were two of the " submitted heats " about which B.H.P. wrote to J. & W. on the 27th February, 1959 (Ex. 76), as follows:—

D.16/8—B.S.S. 968 QUALITY STEEL.

> " In the latest rolling of B.S.S. 968 girder plate for your good selves, test failures occurred in a number of fully-killed electric furnace and open-hearth heats.
>
> Our metallurgical investigation of the heats in question indicated that the steel quality is good as regards soundness and freedom from banding and lamination, also, despite the high tensile, the steel would be weldable.
>
> In the case of heat 243241, the mechanical tests were satisfactory for $\frac{3}{4}$-in. girder plate, but due to the reduction in tensile strength from 35/41 t.s.i. to 33/39 t.s.i. in the heavier sections, the 1-in. girder plate failed to meet the specification.
>
> In view of the foregoing, we are submitting in the attached sheets, details of the heats for your approval.
>
> We have asked our works to forward samples from each heat involved as we understand it will be necessary to conduct weld tests on this material before any decision can be reached.
>
> If you are involved in any additional tests for submission to the Country Roads Board, we would be prepared to meet reasonable costs.
>
> Assuring you of our best attention."

The letter implied that a complete metallurgical investigation had been made to determine why these heats were high in tensile. From Ex. 186 (check tests) it appears that no check analyses were made on plates of some of these heats prior to despatch. It is difficult to understand, therefore, the basis of the confidence expressed in the letter that " despite the high tensile, the steel would be weldable ". At this stage, it was essential that A.I.S. should have made check analyses of plates. Since the failure, many such analyses have been made on heats 55 and 56, and these are summarized in Tables 5 and 6. From these we see that the average of sixteen analyses of heat 55 is carbon ·25 per cent. and manganese plus chromium 2·0 per cent. and of fifteen analyses of heat 56 is carbon ·26 per cent. and manganese plus chromium 2.0 per cent.

It is evident, therefore, that in these two cases, the pit sample was not representative of the heat, and had A.I.S. made the thorough investigation necessary it would have been disclosed that these heats were out of specification as regards composition.

This feature was of much greater importance than the slightly high-tensile strength to which attention was drawn. It raised the weldability index (as calculated from the composition) to D and, therefore, made the task of the fabricator still more difficult. It reduced the notch ductility and thereby weakened the second line of defence against brittle fracture.

This is not the case which is confused by the claim of the steelmaker to a tolerance on a ladle analysis. In our opinion the ladle analyses were not representative of heats 55 and 56 and the bulk of the steel of these heats was clearly out of specification.

TABLE 5.—CHEMICAL ANALYSES OF STEEL FROM FAILED GIRDERS

Girder No. W.14/S	Location	Sample No.	Composition				
			C.	Mn.	Cr.	Mn.+Cr.	N.
	Heat 55—D.S.L. Analyses						
1	North end of centre plate	C.R.B./40	·24	1·77	·24	2·0	..
	South end of „ „	C.R.B./41	·24	1·76	·24	2·0	..
	North end of south plate	C.R.B./42	·25	1·78	·24	2·0	..
	South end of south fracture	C.R.B./43	·26	1·80	·25	2·0	·012
	North of south fracture	S1/F	·25	1·76	·25	2·0	..
2	North end	C.R.B./44	·25	1·78	·24	2·0	..
	At end of north weld	N2/F	·27	1·77	·25	2·0	·011
3	North end of centre plate	C.R.B./45	·28	1·80	·25	2·0	..
	South of south fracture	C.R.B./46	·26	1·80	·25	2·0	·011
4	North end of centre plate	C.R.B./47	·22	1·73	·24	2·0	..
	North of south fracture	S4/F	·22	1·73	·24	2·0	·008
	South of „ „	C.R.B./48	·23	1·77	·24	2·0	..
	Heat 55—A.I.S. Analyses						
1	North of south fracture	W.14/1S	·24	1·73	·27	2·0	..
2	North of north fracture	2N	·25	1·72	·29	2·0	·008
3	North of south fracture	S3/1	·26	1·67	·25	1·9	..
4	North of „ „	W.14/4S	·23	1·72	·26	2·0	..
	D.S.L. mean	..	·25	1·77	·24	2·0	·010
	A.I.S. mean	..	·25	1·71	·27	2·0	·008
	A.I.S. certificate	..	·21	1·70	·23	1·9	..

TABLE 6.—CHEMICAL ANALYSES OF STEEL FROM FAILED GIRDERS

Girder No. W.14/S	Location	Sample No.	Composition				
			C.	Mn.	Cr.	Mn.+Cr.	N.
	Heat 56—D.S.L. Analyses						
1	Near north end	C.R.B./50	·25	1·73	·24	2·0	·010
	„ „	N.I.F.	·26	1·70	·25	2·0	..
	North end south of fracture	C.R.B./51	·26	1·75	·24	2·0	..
2	Centre plate north end	C.R.B./52	·26	1·72	·24	2·0	..
	Centre plate south end	C.R.B./53	·27	1·72	·25	2·0	..
	North end of south plate	C.R.B./54	·26	1·73	·24	2·0	..
	Near south fracture	C.R.B./55	·25	1·72	·24	2·0	·009
3	Near north end	C.R.B./56	·25	1·73	·24	2·0	..
	South of north fracture	N3/F	·26	1·73	·23	2·0	·008
4	Near north end	C.R.B./57	·25	1·73	·24	2·0	..
	North of north fracture	C.R.B./58	·26	1·74	·24	2·0	·009
	Heat 56—A.I.S. Analyses						
1	North of north fracture	1N	·27	1·71	·29	2·0	·008
2	North of south fracture	S2/2	·26	1·68	·21	1·9	..
3	North of north fracture	3N	·25	1·64	·26	1·9	..
4	North of „ „	4N	·26	1·64	·26	1·9	·008
	D.S.L. mean	..	·26	1·73	·24	2·0	·009
	A.I.S. mean	..	·26	1·67	·26	1·9	·008
	A.I.S. certificate	..	·23	1·58	·24	1·8	..

3.4.2. The presence of trace elements in the steel.

In view of the unexpected brittleness of some of the steel used in the bridge project we had exhaustive tests made to see whether there was any detrimental impurity present in trace amounts.

Samples of steel from the failed girders and from a U.K. source have been analysed spectrographically in three laboratories namely:—D.S.L., B.W.R.A. and A.I.S. The results are given in Table 7 for heats 55 and 56 which showed no differences.

TABLE 7.—SPECTROGRAPHIC ANALYSIS OF GIRDER FLANGE PLATE

Element	D.S.L.	A.I.S.	B.W.R.A. Aust.*	B.W.R.A. U.K.
	%	%	%	%
Aluminium	·01—·02	Less than ·005	Less than ·005	ND
Arsenic	NS	·003	ND	
Antimony	NS	·003	NS	
Copper	Less than ·05	·030/·045	·1/·2	·1/·2
Cobalt	NS	·005	ND	ND
Lead	NS	Less than ·002	About ·01	Less than ·01
Molybdenum	ND	·01	·01/·02	About ·01
Nickel	Less than ·05	·030/·045	About ·1	About ·2
Niobium	NS	Less than ·005	ND	ND
Nitrogen	·008/·012	·008	NS	NS
Tin	ND	·005	ND	ND
Titanium	ND	Less than ·005	ND	ND
Vanadium	ND	Less than ·005	ND	Less than ·01
Zirconium	ND	Less than ·005	ND	ND

ND: Not detected. NS: Not sought. * Not identified by heat number but probably heat 55.

The only trace element found in significant concentration is nitrogen, which will be considered in the section on strain ageing. (3.4.4).

From this enquiry it can be stated that no element has been discovered to be present in the Australian steel which is not also present in the sample of steel made in U.K. This result, with the exception of the nitrogen value, is completely negative in helping us to solve the problem of the brittleness of heats 55 and 56.

3.4.3. Notch ductility of the steel.

As previously explained, this property is measured by either the Charpy or the Izod test. To be useful, the results must show a series of values of energy absorbed in breaking the notched bar at different temperatures. It is well known that there is no clearly defined relationship between the values obtained on the two machines. We have therefore, wherever possible, collected values which would help us to correlate the two tests. The Charpy test has become accepted internationally, but it was the Izod test which was specified by the C.R.B. We have also thought it advisable to try to establish whether the Izod values adopted by the C.R.B. were adequate for the protection of the bridge from failure by brittle fracture, in the light of the most recent knowledge. Unfortunately, the results are so variable from individual pieces of steel that no satisfactory solution has been found. Tables 8 and 9 show the Charpy and Izod values obtained on heats 55 and 56 at D.S.L. and A.I.S. during the course of this Enquiry. The variability of the steel is indicated by the following results taken from Exhibit 46 (D.S.L. report).—

			Ft./lb. (Charpy)
Heat 55	Sample S4/F	0°C.	6, 7, 8, 9, 9, 10
,,	,, ,,	21°C.	17, 21, 28
Heat 56	Sample 14/2	0°C.	5, 8
,,	,, ,,	21°C.	13, 31, 34

The accepted standard for protection from brittle fracture in the case of mild steel ship plates is 15ft./lb. Charpy at the lowest operating temperature. Whether we accept 32° F. (0° C.) as proposed by the C.R.B. or 22° F. (–5° C.) as more realistically based on meteorological observation in Melbourne it is evident that neither of the two heats concerned approaches the required Charpy values. Based on the Izod test and using the C.R.B. criterion, heat 55 still shows poor results and although on the average heat 56 just fails, it will be seen from Table 9 that four of the six samples fail to reach the specified value, one of them failing badly. In reverting again to the variability of results obtained in these investigations it should be pointed out that B.H.P., during the Enquiry, maintained that the evidence of this variability as shown by the acceptance tests was dubious because of lack of knowledge of the preparation of the test pieces. We accept that criticism but this can not apply to the values quoted in Table 9, the preparation of the specimens in this case being done under laboratory control for the purposes of this Enquiry.

We conclude that the steel in the flanges of the failed girders was of low and variable notch ductility and that as a result the steel was not able to resist the propagation of brittle fracture from the toe cracks at the transverse weld at the end of the cover plates.

We have plotted all the results available to us relating to Charpy and Izod values taken for comparison from the same plates of steel to B.S. 968 of $\frac{5}{8}$ inch and $\frac{3}{4}$ inch thickness supplied to this contract. There is a wide scatter as has been indicated, and only twenty observations. Assuming that zero energy is the same point for both machines, the best relationship is

$$\text{Izod} = 2\ \text{Charpy}$$

but the values scatter between Izod = 4 Charpy and Izod = 4/3 Charpy. The range of values included was 5 to 55 Izod and 4 to 30 Charpy.

If we accept the above relationship we conclude that the 20 ft./lb. Izod specified by the C.R.B. was approximately equivalent to 10 ft./lb. Charpy, and was therefore not equivalent to the value accepted as protecting mild steel, ship plate, i.e., 15 ft./lb. Charpy. This would be equivalent to 30 ft./lb. Izod. If we look at Table 9 it is seen that this value was recorded in only one case, at S.2/1 (heat 56). This was the location of the first major brittle fracture.

Although the evidence is slender, we conclude that a value higher than 30 ft./lb. Izod (15 Charpy) was necessary with this steel to protect the bridge from brittle fracture, and tentatively suggest that 40 ft./lb. Izod (20 ft./lb. Charpy) would have been necessary, though this can only be an informed guess. However the information on which this view is based was not available when the C.R.B. specifications were written.

The question then arises whether in 1956, steelmakers would have accepted such a value in a specification, using B.S. 968 as a basis. We have seen the reaction of B.H.P. to the value of 20 ft./lb. at 32° F. Yet, when we look at the plate acceptance tests which were carried out on 61 heats of steel to this contract we find that 40 of them would have satisfied the 40 ft./lb. Izod criterion. Of these, 34 were semi-killed basic open-hearth steel and six fully-killed electric furnace steel.

We have for comparison some results from bridge steel made in U.K. at about the same time (1956–57) as the C.R.B. specifications were being written (Ex. 93). For the four thicknesses of steel below 1 inch the values at 0° C. are all above 20 ft./lb. Charpy. Since this is the one valid comparison between B.S. 968 made contemporaneously in U.K. and Australia respectively we have reproduced the results in Figs. 9 and 10. It is at once evident that the transition temperature of the U.K. steel is appreciably lower than that of Port Kembla steel, and the liability to brittle fracture is correspondingly less.

It would appear therefore that the 20 ft./lb. Charpy criterion at 0° C. is not an impossible one to attain.

TABLE 8.—CHARPY VALUES ON HEATS 55 AND 56.

¾-in. Plate.

Ft./lb. at Temperature °C.

Heat	Laboratory	Sample	−20	0	5	10	20	30	40	50	60	80
55	D.S.L. ..	S.4F ..	..	8	..	..	22	..	43	..	57	..
	A.I.S. ..	2N ..	..	..	..	5	10	13	..	22	..	..
	A.I.S. ..	S.3/1 ..	..	4	4	..	10	10	..	..	..	..
	Average ..	.. ..	..	6	..	..	14	..	..	..	..	..
56	D.S.L. ..	14/2 ..	4	8	..	..	25	..	42	..	68	72
	A.I.S. ..	1N ..	..	..	16	27	30	41	..	..	..	..
	A.I.S. ..	3N ..	..	..	..	8	19	30	..	35	..	..
	A.I.S. ..	4N ..	..	..	..	13	17	38	..	47	..	..
	A.I.S. ..	S.2/1 ..	..	6	8	..	26	44	..	..	..	..
	A.I.S. ..	S.2/2 ..	..	5	8	..	28	28	..	..	..	..
	Average ..	.. ..	..	6	..	..	24	..	..	..	..	..

TABLE 9.—IZOD VALUES ON HEATS 55 AND 56.

¾-in. Plate.

Ft./lb. at Temperature °C.

Heat	Laboratory	Sample	−20	0	5	10	20	30	40	50	60
55	D.S.L. ..	S.4F	12	25	..	..	36	..	53	..	..
	A.I.S.	2N	..	8	..	..	23	..	..	..	..
	A.I.S.	S.3/1 ..	..	4	..	..	8	..	..	..	..
	Average ..	.. ..	..	12	..	..	22	..	..	..	..
56	D.S.L. ..	14/2	..	17	..	..	41	..	55	..	65
	A.I.S.	1N	..	25	..	..	33	..	..	..	..
	A.I.S.	3N	..	10	..	..	14	..	..	..	..
	A.I.S.	4N	..	17	..	..	39	..	..	..	..
	A.I.S.	S.2/1 ..	..	30	..	..	33	..	..	..	..
	A.I.S.	S.2/2 ..	..	16	..	..	17	..	..	..	..
	Average ..	.. ..	..	19	..	..	30	..	..	..	..

3.4.4. Strain ageing tests.

It has already been mentioned (Section 2.7.1.2) that the notch ductility may change during the fabricating process. When certain heats of steel are strained slightly so that permanent deformation occurs, and this is followed by heating to about 250° C., the notch ductility decreases because of the phenomenon of strain ageing. Straining for straightening and heating for and by welding are inherent in the fabrication processes. Strain ageing is associated with the presence of nitrogen in the steel. This element forms a nitride which can dissolve in the ferrite (the relatively pure iron part of the steel structure). Whilst it is in solution it is relatively harmless but when such steels are plastically deformed by stress the nitride will precipitate from the solid solution on heating. There is an intermediate stage just prior to precipitation at which the nitrogen atoms form clusters and this appears to be the condition which produces pronounced notch brittleness. It is in this respect that the nitrogen content of the steel is important. Heats 55 and 56 contained ·012 per cent. and ·009 per cent. nitrogen respectively and this is rather above the normally accepted limit. The electric furnace heats showed almost double the nitrogen content of the open-hearth heats (Ex. 184—Section 2).

Steel supplied for the contract was tested for strain ageing characteristics both by D.S.L. and A.I.S. laboratories. A synopsis of the results is given.

D.S.L. confined their experiments to a flange plate from heat 55. The treatment and the results are shown below. (Ex. 194, App. 9).

D.S.L. strain ageing tests on heat 55.

	Average of Three Tests (Ambient temp.)
As received	7·5 ft./lb. Charpy
Heated to 250° C. 30 minute air-cooled	5·5 ,, ,,
1 per cent. plastic strain/heated 250° C. 30 minute air-cooled	4·0 ,, ,,
5 per cent. plastic strain/heated 250° C. 30 minute air-cooled	3·5 ,, ,,

This was brittle material as received but the ageing treatment increased the brittleness.

Another sample was taken from within 2 feet of the last ones and was subjected to a normalizing treatment with the following result :—

	Average of Three Tests
As received	13 ft./lb. Charpy
Normalized by heat 900° C. and furnace cooling ..	32 ,, ,,

This result suggests that the material as received from the girder was already in a strain-hardened condition—possibly from fabrication effects. There was a large amount of pulling and pushing during the fabrication of a girder. This may account for some of the variability which characterises these tests, for example the differences between the two "as received" samples above taken from plates 2 feet apart. It would not account for the variability in samples from one plate such as those mentioned in Section 3.4.2.

The A.I.S. results on strain ageing were made on material from J. & W. stock and are recorded in Table 10.

The results are not sufficiently extensive to draw firm conclusions but it would seem that strain ageing is a general characteristic of A.I.S. steel made to B.S. 968.

Other steels investigated at the same time show that imported B.S. 968 and other heats made by A.I.S. are liable to the same characteristic.

We conclude that the steelmaker should give special consideration to this problem particularly as the addition of "scavenger" elements in the ladle would probably reduce its incidence.

TABLE 10.—STRAIN AGEING RESULTS ON B.S. 968, KINGS BRIDGE CONTRACT. (EXHIBIT 184, SECTION 4).

Heat No.	Type	Izod ft./lb.			
		Tested 0° C.		Tested 20° C.	
		As Received	Strain Aged	As Received	Strain Aged
4	O.H.	9 (23)	. .	25 (19)	6
4	,,	6	. .	8	7
4	,,	11	. .	16	12
20	,,	50 (27)	25	52 (30)	41
20	,,	43	19	50	26
27	,,	48 (42)	15	55 (51)	41
47	E.F.	19	13	27	22
53	,,	11	. .	26	10
65	,,	36	34	46	45
84	,,	43	22	47	58
84	,,	41	14	46	26
84	,,	34	16	49	29
105	,,	27	9	41	18
109	,,	43	36	67	38

The values in brackets refer to acceptance tests as shown in Exhibit 72.

3.5. Summary of the Metallurgical Causes of the Failure.

In this section we eliminate the human element in so far as it relates to matters of opinion and decision with regard to the conduct of the project, and confine ourselves to the facts as they have been established since the failure. In this sense we confine our opinions to the properties of the steel supplied and the properties of the welds made. We further restrict ourselves to heats 55 and 56 as these were the two heats of steel immediately associated with the failure.

1. The heats were out of specification regarding carbon and on the maximum limit with respect to manganese plus chromium.
2. The steel was susceptible to strain ageing, probably due to its high nitrogen content.
3. The steel was of low notch ductility.
4. The pre-heat used in making the transverse weld was probably too low, though there is no direct evidence relating to this.
5. The electrodes may not have been dried sufficiently immediately prior to welding thereby introducing hydrogen into the welds.
6. The thermal severity characteristics of this weld had been underestimated.
7. Acid had been used to clean the welds prior to painting.
8. Because of (1), (4), (5) and (6) cracks developed at the toes of the transverse welds and these were the primary cause of the failure.
9. There was a high degree of restraint on the transverse weld due to the fact that it was the last weld made on the girder. The restraint was due to the two taper welds and the web/flange fillet welds. The result of this restraint was a high-longitudinal tensile stress operating at right angles to the transverse weld.
10. It is possible also that during the handling of the girders, after the final welding and before painting, tensile stresses were set up which would tend to open up the cracks. We have no means of estimating these stresses.
11. Because of (2), (3), (7) and (8) the welds developed partial brittle fractures from the extremities of the toe cracks. These brittle fractures reduced the strength of the flanges in W.14–2 and W.14–3 so much as to eliminate them as load-bearing features. W.14–1 was similarly weakened at the south end.

PART 4.—SUMMARY AND CONCLUSIONS.

4.1. General Observations.

Before presenting our final assessment of the responsibility for the failure of the bridge, it is desirable to make some general observations.

The basic fact of the whole Enquiry is that a large and important public structure, which cost the community overall some £4,000,000 failed dramatically and embarrassingly within fifteen months of its coming into service, and failed moreover, by reason of inherent defect, and not from the action of some natural force of unforseen magnitude, or other external factor.

Such a failure should not have occurred. It cried out for explanation, and for the assessment of responsibility among those whose duty it was to create a safe and satisfactory structure, and we reject the theory, somewhat diffidently suggested to us at one stage, that the failure was due to sheer misfortune occurring in such circumstances that no blame can be attached to any of those associated with the project.

We accept the argument addressed to us by Counsel, that our task, so far as it relates to fixing blame and responsibility, is to do so as a matter of fact simply, and not to concern ourselves with legal liability, direct or vicarious. We have made every endeavour to avoid any findings outside our proper sphere, particularly having in mind the existence of litigation currently proceeding between some of the parties, and the possibility of future litigation.

We refer to some minor matters of controversy which arose during the sittings of the Enquiry, and which are to be found scattered over the 4,000 pages of transcript. These matters were relevant, and at the time appeared important, but are not vital to our findings, and reference to them is omitted in the interest of avoiding intolerable length in this Report.

We have also asked ourselves whether we have been judging these incidents by standards that are too high for ordinary mortals to reach and whether we are demanding higher standards of competence from engineers than we would from doctors, lawyers or other professional men. It is, of course, undeniable that a doctor or lawyer who makes a mistake which causes his client to lose life or liberty normally escapes public censure although he may have to answer a charge of negligence in the courts. But engineers generally—and in this case certainly—do not work as individuals in a consultant–client relationship, they work as a team. This certainly brings with it problems of communication and organization but it also means that individual engineers are supported by others who can help and check their work.

From this standpoint we do not think that we have been too harsh on individuals. It is the various organizations within which they were working that we regard as being collectively responsible for the unhappy state of affairs that we have been investigating.

4.2. Findings Pursuant to Terms of Reference.

We now present, in summary form, our major conclusions and findings arising directly under the terms of reference. It will be appreciated that throughout the body of this Report we have made a great many findings of fact and expressed a number of opinions which we have not attempted to summarize, but which we, nevertheless, consider important. In the following pages will be found only the fundamental and essential findings.

1. The major causes of failure were as follows :—

(*a*) Cracks were present at the toe of the welds terminating the cover plates on the tension (lower) flanges of the W.14 span which collapsed.

(*b*) The cracks were caused by the unfamiliarity of the fabricator, Johns and Waygood Limited, with the problems of welding low-alloy steel, and the quality of the steel supplied by Broken Hill Proprietary Company Limited much of which was so high in carbon, and so unexpectedly variable, that even an experienced fabricator would have had difficulty in welding it.

(c) The cracks remained in the girders because they were not discovered either by Johns and Waygood Limited or by the Country Roads Board inspectors.

(d) The steel supplied by the Broken Hill Proprietary Company Limited for the tension flange plates on the span concerned, accepted by the Country Roads Board, and used by Johns and Waygood Limited without adequate examination, was low in notch ductility and was thus unable to resist the propagation of brittle fractures from the toe cracks.

2. Under the specific terms of reference we find as follows :—

Term I.—The terms, conditions, specifications, and drawings in accordance with which tenders for design and construction of the bridge were invited by the Country Roads Board, and whether the same were adequate and reasonable for the purpose.

(a) The form of contract which was entered into, as required by the terms, conditions, and specifications, called a "design and construct" contract was in fact a modification of the contract usually so described. It was unsuitable in that it failed to provide the necessary over-all supervision of the various aspects of the work and was to some degree responsible for the absence of a proper co-ordination of the project.

(b) The terms and conditions in accordance with which tenders were invited were inadequate in that the clause dealing with the assessment of tenders should have provided that one of the factors to be considered was "the ability and competence of the tenderer to perform the contract and, in any case in which it is proposed to sub-contract a substantial part of the work, the ability and competence of the sub-contractor."

(c) The specifications and drawings were in general satisfactory and reasonable, but were unsuitable to the following extent :—

(1) In the sections relating to the manufacture, testing, and inspection of steel, the specifications lacked precision and contained ambiguities.

(2) A number of important problems only arose because the specifications included B.S. 968 : 1941 steel with added clauses. It would have been much better if the C.R.B. had set out fully the specification of the steel required without using the B.S. 968 label at all.

Term II.—The tenders received, the action taken to investigate the same, the circumstances surrounding the acceptance of the tender submitted by Utah Australia Limited, and whether the acceptance thereof was reasonable and proper and justified in the circumstances.

The number of the tenders received was reasonable, but we feel that the C.R.B. must have been disappointed that the names of some bridge builders of world reputation were conspicuous by their absence.

The action taken to investigate the tenders was reasonable and proper from the point of view of business propriety. We, however, make some criticism of the C.R.B. in Section 2.2. of our Report : in particular we feel that the use of high-tensile steel in the Utah tender should have been specifically drawn to the attention of the Government before the decision to use this material was finally made.

The acceptance of the Utah tender was reasonable from the point of view of business propriety. However, it should not have been accepted without far more enquiry and investigation, because the material to be used in the superstructure was one of which neither the C.R.B. nor Utah had any experience. We found no evidence or any circumstances giving rise to any suspicion of impropriety in relation to these matters.

Term III.—The design submitted and adopted for the bridge, and whether the same was adequate and suitable or was in any and what respects defective or inappropriate or deficient.

(*a*) The general design was adequate and suitable. We are of the opinion, however, that the choice by Utah of high-tensile steel as specified was unfortunate for the reasons stated hereafter.

(*b*) The use of cover plates, and the design of the end detail of cover plates was criticized in the report of the Committee of Investigation (Appendix 3) and a great deal of evidence and discussion was directed to this matter. We are not prepared to find that, in 1957 this feature was undesirable, but on the basis of knowledge now available, we are satisfied that any feature involving transverse fillet welds in tension flanges is undesirable.

Term IV.—The materials and processes and workmanship used in the construction and erection of the bridge, the standard and suitability thereof for the purposes for which they were used, whether they were in accordance with the contract specifications and whether they were in any and what respects defective or inadequate.

(1) *Materials.*

Steel.

(*a*) At the stage at which the specifications were drawn we are agreed that on balance the C.R.B. was justified in including high-tensile steel as an optional material for the superstructure. However, we are quite satisfied that this material should not have been used for the whole of the superstructure in a project of such magnitude, in the circumstances existing in 1957, where neither Utah, J. & W., nor the C.R.B. itself, had any experience in the use of the material for welded bridge building.

(*b*) Some of the steel used in the bridge and particularly in the girders of the W.14 span did not comply with the C.R.B. specifications as regards composition and notch ductility and should have been rejected by J. & W. and the C.R.B.

(*c*) J. & W. while bound to supply steel to the specifications (B.S. 968 with additional clauses) ordered the steel from B.H.P. as B.S. 968 : 1941 simply without the additional tests. J. & W. with the knowledge of the C.R.B. failed to carry out sufficient tests to ensure the notch ductility of some of the steel used, and failed to make check analyses of the steel supplied to ensure its correct chemical composition.

(*d*) J. & W. and the C.R.B. accepted delivery of certain " submitted heats " to which their attention had been drawn by B.H.P. No proper consideration was given to these heats before acceptance, and some of them should have been rejected, in particular heat 55, which was used in the failed girders.

(*e*) All these parties failed to hold sufficient discussion on the subject of the quality of the steel.

Electrodes.

The electrodes used—Fortrex 35 for manual welding and Murawire and Muraflux for automatic welding—supplied by Murex were suitable, in accordance with specifications, and in no way defective or inadequate.

(2) *Processes and Workmanship.*

There was no evidence to suggest any defect or departure from proper standard in any of the processes or workmanship involved except in the matter of *welding*.

In relation to welding we reach the following conclusions :—

(*a*) J. & W. did not adequately prepare its staff and workmen for welding low-alloy steel.

(*b*) No pre-determined sequence was set down for laying the cover plate end weld, the importance of which was not realized.

(*c*) At the time when the W.14 girders were fabricated, adequate pre-heat was not used and it is possible that the electrodes were not properly dried.

(*d*) Whilst the high carbon and manganese content of some heats increased the difficulty of producing crack-free welds, the cracking found is so general, irrespective of the heat of steel or the time of fabrication, that we consider the welding techniques are also responsible for it.

(*e*) We condemn the practice of using acid for the final cleaning of welds for if there is any undetected crack the presence of acid will create a brittle condition in the surrounding steel.

(*f*) Insufficient time was allowed between the last welding operation and the final inspection and the conditions during that inspection were such as made adequate inspection difficult.

Term V.—The nature, extent, and standard of supervision exercised over the construction and erection of the bridge, and whether the same was reasonable and adequate or was in any and what respects inadequate or defective.

Utah.

Utah, the head contractor, which might have been expected to undertake considerable supervision of the work of J. & W., deliberately refrained, as a matter of policy, from any such supervision or inspection, on the basis that doing so would only duplicate and interfere with the work of C.R.B. inspectors. Utah did appoint Scarlett of J. & W. as its representative on matters of testing and inspection or acceptance of steel, but did little else. We do not make any adverse comment on this attitude, as it was accepted by C.R.B.

C.R.B.

The system of acceptance testing of the steel was deficient in several respects and whilst never utilizing its full right of testing, C.R.B. ultimately gave way to pressure exerted by J. & W. and Utah and relaxed its testing requirements without adequate consideration. The C.R.B. also set up an inspection system designed to effect a satisfactorily high standard of workmanship. However, it was carried out by people who lacked any experience in the technique of inspection or in the welding problems associated with the steel being fabricated. As a result, the control was in some regards too rigorous, so that the personnel at J. & W. were made resentful at what they regarded as " pin pricking ". On the other hand, while many defects were discovered and remedied, the system failed lamentably to uncover some crucial defects including the cracks associated with the W.14 girders which failed. For reasons given in Section 2.10.6, these defects should have been discovered.

J. & W.

The fabricator never understood the need for meticulous inspection of welding when the material being fabricated was high-tensile steel. This attitude, combined with the suspicion that the inspectors knew no more than they did, engendered a strong feeling of resentment, which led to constant arguments over standards of workmanship. Harassed by the inspectors on the one hand, and on the other pressed by Utah for production on schedule, the J. & W. management continually sought some relaxation of inspection. All this militated against the efficiency of the system. It is fair to say, therefore, that J. & W.'s attitude contributed to the failure by the C.R.B. to discover important defects which passed into the bridge structure.

Term VI.—Whether any and what negligent, culpable, or improper act or omission directly or indirectly caused or contributed to the failure of the bridge, and if so the party or parties responsible therefor.

The determination of the questions arising under the first head of this term of reference, required us to undertake the difficult task of fixing some standard of negligence which can fairly be applied to the matters which we have found necessary to criticize in the conduct of many of those associated

with the Kings Bridge project. We accept the argument that not every act or omission should be held to be " negligent " within the meaning of that word in this context, even though they might be negligent from the standpoint of legal liability. For the purpose of this term of reference we have, therefore, adopted the standard that any course of action, resulting from a considered judgment exercised after proper enquiry, is not negligent even though in the event it proved to be wrong. Nor do we class as negligent some instances of lack of competence arising simply from inexperience, under circumstances in which the party concerned has acted to the best of its skill and ability. On the other hand, we treat as negligent such instances of error as we are satisfied occurred because those responsible entirely failed to give reasonable consideration to the particular decision.

Applying this somewhat restricted test we are of the opinion that the following matters of importance do constitute negligence :—

1. The failure by J. & W. to carry out its clear obligation to test and select, from the material supplied by B.H.P., steel which would have come within the full requirements of the C.R.B. specification.
2. The decision by C.R.B. to abandon the full programme of acceptance testing and to carry out testing on a random basis.
3. The acceptance by the C.R.B. and J. & W. of the " submitted heats " which B.H.P. had given them the opportunity to reject or accept.
4. The offer by B.H.P. to J. & W. of the " submitted heats " by means of a letter (Ex. 76) which declared that " our metallurgical investigation . . . indicated that the steel quality is good . . . also . . . the steel would be weldable ". A proper investigation could not have supported such a statement (See Sec. 3.4.1.).

We found no evidence of any act or omission which we are prepared to find was either culpable or improper. Specifically we refer to one suggestion of gross impropriety which was given some publicity shortly before the Enquiry started. This allegation was to the effect that someone, presumably in the employ of J. & W., had intentionally concealed known cracks in the girders by filling them with red lead and painting over them. We are satisfied that there is no truth in this suggestion. It is true that the inspection methods failed to reveal certain cracks which should have been found and that paint was found to have penetrated into some of them, which was a significant matter from several points of view. However, the suggestion of deliberate concealment is unfounded.

Term VII.—Whether the construction and erection of the bridge in accordance with the tender submitted by Utah Australia Limited was reasonable having regard to the known state of engineering and scientific knowledge and experience subsisting at the time the tender was accepted.

The general knowledge in the engineering profession of the quality and behaviour of B.S. 968 : 1941 steel has greatly increased since January, 1957. Nevertheless, there was ample information in the literature generally available at that date, and in the minds of metallurgical engineers like Ferris, to have provided the parties involved in the bridge project with a clear warning that this steel required very special care and skill in manufacture and fabrication.

All the parties except Ferris and Murex failed to realize that in essaying the construction of a large and important welded structure in this material, they were taking a very long step from the familiar processes of mild-steel construction into an unfamiliar situation where more elaborate techniques were required. Because of this lack of appreciation, the proper programme of test and experiment which alone would have justified the use of this steel was not undertaken.

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4.3. Condition of the Superstructure.

Although it is not within our terms of reference, we cannot conclude our Report without recording our very grave concern about the future of the steel work of the bridge. We know that it contained a great many cracks that have been found and removed but from all we have learnt, we are certain that there must be many more. Some of these may be in critical and highly-stressed regions. Such cracks must be a continuing source of danger, either of brittle fracture or fatigue, unless some means is found to reduce the tensile stress concentrations to negligible amounts. We wish to point out that we do not know of any determination of the fatigue properties of this steel even at this late stage.

We desire to record our thanks to those who have rendered us very great assistance during the course of this Enquiry.

Counsel assisting the Commission, Mr. S. T. Frost, Q.C., and Mr. Gordon Just, displayed great industry and capacity.

Their skill in presenting the evidence, and in advising us on procedural and other matters, was of the greatest possible assistance.

We also wish to thank the other Counsel appearing at the Enquiry, who, while never losing sight of their paramount duty to advance and protect their clients' interests, so conducted their cases as to greatly assist our investigation.

Many of the organizations directly and indirectly concerned with the Enquiry provided us with valuable information, and some of them devoted much time and expense to scientific experiment and the compilation of extensive records.

In particular, we thank the Defence Standards Laboratories, and the Melbourne and Metropolitan Board of Works. Two organizations directly concerned with the Enquiry—the Country Roads Board and the Broken Hill Proprietary Company Ltd.—each provided us with much material of great importance beyond that which was necessary for their own purposes.

Engineering, Testing, and Research Services Pty. Ltd. carried out certain tests and investigations for our information, and its Manager and Metallurgist, Mr. Hudson, rendered us extremely valuable assistance.

Finally, we thank Mr. C. A. Mitchell, the Secretary to the Commission, who discharged his duties with unfailing good will, and whose long experience of Enquiries of this nature was most useful to us.

E. H. E. BARBER, Chairman.

J. NEILL GREENWOOD, Member.

J. A. L. MATHESON, Member.

Melbourne,

3rd June, 1963.

APPENDICES.

APPENDIX 1.

ALPHABETICAL LIST OF WITNESSES.

Name of Witness	*Pages of Transcript*
BAKER, Geoffrey William	3351–3356A
BONWICK, John Edwin	1063–1204 3321–3326A
BOOTH, Reuben Joseph	1649A–1651
BOX, George Radburn	3307–3320
BULL, Professor Frank Bertram	2802–2879
BUTLER, Leicester Travers	1849–1856
CAMPBELL, Richard	1573–1628 3159–3163
CLARKE, Norman Victor	943–1003 1030–1063 3159–3160
DARWIN, Donald Victor	170–225
EASTICK, Robert Frank	555–942 In Camera, 1–7
FARRAR, William Crisp	2881–2908 3062–3141
FERRIS, Irwin James	2616–2776 3348–3350 3622–3
FINK, George William	1216–1257
FISHER, Eugene Peter	2343–2373
FOX, Edward William	1651–1683
FRANCIS, Professor Arthur James	1881–2008
GARDEN, Kenneth Alexander	2073–2085 2146–2147
GRAHAM, David Gordon	1683–1703
HARDCASTLE, Phillip Alfred	2053–2073
HARDCASTLE, Roy Thomas Andrew	423–509 2012–2037
HOFFMANN, Siegfried	1628–1649
HUDSON, Robert Frederick	515–551 3218–3230
HUNTER, Joseph	2777–2787 2791–2801 3039–3058
JAEGER, Ernest	3212–3217 3243–3251
MAIN, Allen Bennie	1780–1844
MASTERTON, Clifford Alexander	2085–2130
MATHIESON, John	1845–1849
MOLL, Victor Raymond	551–555
MORRIS, David Owen	3252–3259 In Camera, 8–42

APPENDIX 1—*continued.*

Name of Witness	*Pages of Transcript*
MUIR, Professor Hugh	2911–3037
NOBLE, Raymond	255–259
RALSTON, Owen Bruce	2133–2253 3142–3158
REEDY, Lorne	3199–3211
ROBB, George	3338–3344
SCARLETT, Kenneth Frederick Alexander	1472–1572 3265–3299
SIDEBOTTOM, William Logan	1012–1023 2333–2342
SLATTERY, Francis James	3164–3174
SLAVIN, Benjamin	1004–1012 2325–2333
STOCKER, William Reginald	1205–1214 1257–1456
SWANSON, Victor George	225–229
THOMPSON, John Ward	2255–2324 2473–2603 3175–3194
WARD, Frank Alexander	1705–1780
WILLIAMS, Frederick Vincent	3327–3337
WILLIAMS, Ronald Trethewey	2373–2400
WILSON, Cecil Alexander	229–254 259–423 1857–1881

APPENDIX 2.

LIST OF EXHIBITS.

Exhibit Number	Description	Page at which Tendered
1	Report of Committee of Investigation	47A
2	Photographs numbered 1 to 16 (being the photographs referred to in Ex. 1)	49
3	Plans numbered R23, 00120 and R23, 00116	49
4	Colour slides of fractures (Nos. 1 and 2, showing failure of girder W.14 (2), Nos. 3 and 4, showing failure of girder W.14 (3), Nos. 5 and 6, showing failure of girder W.14 (1) and Nos. 7 and 8, showing failure of girder W.14 (4). Each of the failures shown is at the southern end of the girders)	53
5	Two photographs of crack on girder E.11	55
6	Specification—five books	97
7	Contract (being contract between C.R.B. and Utah, including annexures except for specification and drawings)	97
8	Book of Drawings (made by tenderer and forming part of tender and ultimately of the Contract)	97
9	Schedule of tenderers (annexed to Exhibit 11)	98
10	Letter dated 12th March, 1957, from Sir Herbert Hyland to the Premier, and the Premier's reply dated the 27th March, 1957	103–4
11	Letter dated 27th May, 1957, from Mr. Darwin to the Minister for Public Works	103–4
12	Memorandum dated 20th June, 1957, from Mr. Swanson to the Minister for Public Works	111
13	"Answers to Minister's Questions" in a document dated 27th July, 1962, from Mr. Roberts, the Chairman of the C.R.B. to the Minister for Public Works	117
14	Book of drawings..	125
15	Correspondence passing between J. & W. and Utah (the contract between these parties is contained herein)	131
16	Bundle of letters by various people concerning quality of steel (this exhibit was later absorbed in Ex. 51)	135
17	Copy of Order from J. & W. to B.H.P. dated 20th May, 1958 (the first order herein)	138
18	A.I.S. acceptance of order (the acceptance of the order referred to in Exhibit 17)	139
19	Utah Construction Co. brochure (submitted by Utah with its tender) ..	144
20	List of heat numbers and chemical compositions (showing the C.R.B. numbers applied to B.H.P. heat numbers and the ladle analysis of each heat; this document compiled by C.R.B.)	147
21	Undated letter (about February, 1958) from Dr. Fox to C.R.B. ..	149
22	Paper "Development of Project" by Mr. D. V. Darwin, and paper "Development of Electric Welding", by Mr. C. A. Masterton	153
23	Book "Arc Welding of Low Alloy Steels" (usually referred to in the hearing as "Bradstreet" or as "B.W.R.A. Booklet")	154A
24	Report on overseas plate girder practice by Mr. J. E. Bonwick ..	157
25	Report by Dr. Weck	169A
26	Surface treatment of steelwork—Supplementary specification, clause 4-8-5	169A
27	Summary of tests specified in specifications for high-tensile steel ..	235
28	Document *re* Bailey bridges	248
29	Article "The Selection of Steels for Welded Structures", by Hugh Muir and J. S. Hoggart	320
30	Report by Professor Francis	355
31	British Standard 2642 : 1955	394
32	Memorandum dated 29th November, 1957, by Mr. Butler to C.R.B. (as to inspection of steel in the mill)	394
33	Schedule of bridges built in welded medium and high-tensile steel and photographs of some of these bridges	402 and 419
34	Summary of steel specification requirements (refers to U.K., France, Germany, and U.S.A.)	403
35	Extract from "Public Roads", Volume 30, No. 4, October, 1958, "Current Structural Bridge Steels : A Survey of Usage and Economy", by Nathan W. Morgan	403
36	Standard Composite I-Beam Bridges, spans 50 feet to 100 feet (U.S. Department of Commerce, Bureau of Public Roads, May, 1953)	405
37	Letters and enclosures .. .. 1. Dated 12th November, 1957, C.R.B. to Utah 2. Dated 6th December, 1957, Utah to C.R.B. 3. Dated 14th February, 1958, C.R.B. to Utah	408

APPENDIX 2—*continued.*

LIST OF EXHIBITS—*continued.*

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38	Letters and enclosures .. .. 1. Dated 18th February, 1958, Utah to C.R.B. 2. Dated 13th March, 1958, C.R.B. to Utah 3. Dated 25th March, 1958, Utah to C.R.B.	412–413
39	Letter dated 25th March, 1957, Utah to K.S.B.D., and agreement dated 19th January, 1959, between Utah and K.S.B.D. Ltd.	426
40	Computations of girders W.14, 1–4 together with transmittal notices ..	435A
41	Approved drawings of girders which failed	437–438
42	Report dated 10th March, 1960, from Department of Civil Engineering of the University of Melbourne on eight pieces of steel (sent to it by C.R.B. for testing) together with diagrams (the pieces of steel are further described at page 1657 of the transcript)	461
43	Sheet of Civil Engineering lecture notes for 1961 of the University of Melbourne (identified by Professor Francis at page 1889 of the transcript)	468
44	Paper "Transfer of Stresses in Welded Cover Plates", by A. M. Ozell and A. L. Conyers, published in Welding Research Council (U.S.A.), Bulletin No. 63, August, 1960	502
45	Paper "Fatigue in Welded Beams and Girders", by W. H. Munse and J. E. Stallmeyer, published in Highway Research Board, Bulletin No. 315, March, 1962	504
46	Report dated 3rd August, 1962 (with annexures) by Mr. Ferris ..	511
47	Piece of cover plate from girder E.14–3 at the north end	531
48	Kings Bridge steel girders, key drawings (being a plan presented by Mr. Hudson showing the results of the examination of the transverse welds at the ends of the cover plates)	532
49	Photostat copy of Mr. Hudson's findings	534
50	Drawing No. JW746A (a shop drawing used for fabricating; it refers to girder W.11S1 and gusset plates)	555
51	C.R.B. and Utah correspondence, &c. (This includes the letters referred to by Mr. Eastick in his evidence in chief)	557–558
52	Comparative table showing pre-heating requirements (compiled by Mr. Eastick)	572
53	Report by Mr. Jackson "Investigation into the Heat-affected Zone Cracking of B.S. 968 Steel Using the Controlled Thermal Severity Test"	577
54	Report by Mr. Jackson "Investigation into Weldability and Physical Properties of B.S. 968 Steel Plate"	578
55	Part 1.—Typed copy of relevant extracts from Mr. Wilson's diary .. Part 2.—Mr. Wilson's diary	578 688
56	Pamphlet describing National Association of Testing Authorities ..	595
57	Paper "Radiographic Inspection of Welded Highway Bridges", by John L. Beaton, presented at the 37th Annual Meeting of the Highway Research Board, Washington D.C., 6th to 10th January, 1958	597
58	Sheet showing modified radiographic standard	599
59	Circular dated 20th August, 1958, prepared by Mr. Jackson, being "Inspection Procedure List for Welding Supervisors Engaged on the King-street Bridge Project"	602
60	*Pro forma* procedure sheet	607
61	*Pro forma* visual inspection results	607
62	Typical trace of radiographic examination	607
63	Three files of test certificates produced by Utah (the files include certificates of physical tests on the steel supplied given by the steelmaker by E.T.R.S. and by Royal Melbourne Technical College, and certificates of analysis given by the steelmaker and also letters to and from Utah transmitting to and from C.R.B. and J. & W. the certificates and approvals thereof)	644
64	Procedure qualification fillet weld test, automatic and semi-automatic (an extract from "Inspection and Tests of Welding of Highway Bridges", by John L. Beaton, December, 1958)	688
65	Letter dated 8th October, 1962, from Mr. Wilson to Colvilles Ltd. and reply dated 18th October, 1962, thereto	703
66	List showing dates of erection of girders, King-street Bridge project (prepared by Utah)	704
67	Certificates of mechanical tests with diagram and photograph referring to letter of 22nd July, 1958 (the certificates were given by Murex)	710
68	The comparison of bending stresses enclosed with letter of 22nd April, 1958 (the comparison being made by K.S.B.D.)	710
69	Utah file on induced stresses	731
70	File in support of J. & W. claim for extra payment	737
71	Part 1.—Copy of wall charts supplied to C.R.B. by J. & W. .. Part 2.—Original of the said wall charts	738 931

APPENDIX 2—*continued.*

LIST OF EXHIBITS—*continued.*

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72	Chart compiled from C.R.B. records relating to heats (amendments to it are in transcript pages 740–741)	740
73	Details of heat numbers of steel in main girders	742
74	Correspondence tendered by Utah	743
75	Copy of Mr. Scarlett's notes of conferences 23rd June, 1959; 25th November, 1959; 14th December, 1959; 16th December, 1959; and 17th December, 1959	797A and 1658
76	Letter dated 27th February, 1959, with enclosures from B.H.P. to J. & W.	825
77	Examples of completed copies of *pro forma* as in Exhibit 60	828A
78	British Standard Specification No. 2549 of 1954	848–849
79	Copies of all orders for steel placed by J. & W. with B.H.P. ..	881
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125	Letter dated 10th February, 1960, from British Iron and Steel Corporation Ltd. to Norman W. Hutchinson and Sons Pty. Ltd., with sketch drawings and letter dated 16th February, 1960, from the latter to Johns and Waygood	1311
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APPENDIX 3.

RELEVANT EXTRACT FROM REPORT OF COMMITTEE OF INVESTIGATION.

The Failure.

On 10th July, 1962, at a time between 11 a.m. and 11.30 a.m. one span close to the southern end of the western half-section of the High Level Bridge collapsed suddenly under a load carried by a semi-trailer vehicle having a total length of about 49 feet. The total weight was approximately 47 tons. The stresses in the bridge as designed due to this load were within permissible limits.

The span concerned has four (4) suspended girders 100 feet long carried by cantilevers extending from adjoining spans. Each of the four girders was fractured at a point approximately 16 feet from the southern end, and three at points approximately 16 feet from the northern end. The structure sagged a maximum distance of about 1 foot but did not completely collapse, the fractured girders being supported by the concrete deck and to some degree by vertical concrete wall slabs enclosing the space beneath the bridge.

Each girder consists of steel flange plates welded to a steel web plate. The forces sustained by the flanges in girders of this type vary from a minimum at the ends to a maximum at the centre and the thickness of the flange is commonly proportioned accordingly. In these girders the variation was obtained by constructing the flange of two plates, the lower of which was welded to the upper but did not extend over the whole length. The lower plate is described as the cover plate.

In the four girders there are thus eight (8) points of termination of the cover plates and the seven (7) fractures occurred at these points.

As a precautionary measure, all parts of the bridge were closed to traffic later in the same day.

There was no evidence of settlement or other movement of the foundation sub-structure or the supporting piers and the investigation has been directed to the superstructure.

The Investigation.

The matters examined by the Committee are related to the known or probable factors concerned. Responsibilities have not been considered by the Committee.

The items of the investigation were :—

1. The cause of the failure.
2. Tests and examinations to ascertain, as far as possible, the presence of any other points or potential points of weakness.
3. The degree of risk in re-opening parts of the bridge to traffic and restrictions desirable.
4. Remedial measures.

1. *The Cause.*

Examination and inspection of the fractures show that the failure was of a type known as " brittle fracture " in the steel. This is a condition brought about by the combination of several principal factors, namely :—

(*a*) The nature of the steel ;

(*b*) low temperatures ;

(*c*) the process of fabrication, in this case, welding ;

(*d*) design details.

Brittle fracture is a mode of failure which may occur in any steel if a critical combination of condition exists. These are principally :—

(1) The presence of a notch or indentation, crack, or other flaw ;

(2) temperature in the range critical for the particular steel concerned ;

(3) a stress sufficient to cause a crack or flaw to extend.

Under the most unfavourable combination of these factors, a collapse may occur whilst the structure has no load on it other than that due to its own weight.

The critical temperature in the steel used in the construction of this bridge is relatively high and brittle fracture could, therefore, occur at temperatures commonly experienced in Melbourne, particularly in the winter. Minimum night temperatures at the time of the failure were in the order of 35° F. to 40° F. and this is considered to have been a factor contributing to the failure.

The steel used in these girders was of a kind commonly known as high-tensile steel. The specification required that it should conform to British Standard No. 968 : 1941, and contained additional conditional conditions relating to impact properties.

It is known that steel of this description presents certain difficulties in welding and that this was understood at the time of the preparation of the specification. Extensive precautions were taken to ensure a satisfactory structure; however, it is the opinion of the Committee that the nature of the steel used, although so far as is known at present, conforming in general to the specification, was such that even these precautions proved inadequate.

Careful and extensive records were kept by the Country Roads Board of certificates supplied by the sub-contractors fabricating the girders certifying the quality of much of the steel.

There are gaps, however, in the information relating to some of the "heats" or batches of steel included in the girders which failed.

The fractures occurred in each case at the short tranverse weld at the end of the cover plates and the examinations and tests made show that an excessively hardened zone was created in the flange plate at this weld so that "locked-up" stresses caused cracks to occur in these zones. In the girders which failed, it is clear that the cracks led to the fractures and failure.

Four of the seven fractures have been examined closely by the removal of parts of the girders and there is little doubt that at least one girder was fractured over a considerable portion of its depth not less than three months and possibly much longer, prior to the failure. Two other fractures show that some cracking existed before the failure. These fractures also show traces of red lead paint indicating that some cracking was present either before the girders left the fabrication shop or before a second coat of similar paint was applied after erection.

The failures are associated with a specific design feature, namely the stopping of cover plates. This is known to produce high local concentration of stress and is considered an undesirable feature.

Whilst the Committee has not been able to ascertain whether the fabrication procedures actually employed were in accordance with those laid down in the specification, the existence of heat-affected zone cracks suggests that these may have been insufficient. It may have been that no pre-heat was used in making the end welds of the cover plates or that there may have been excess hydrogen present either from a pickling process or electrode coatings; further that the rate of heat input to the joint in making the manual welds was insufficient.

In general, automatic or semi-automatic welding methods were used but at certain points, including the ends of cover plates, manual welding was employed.

2. *Tests and Examination.*

The character of the observed fractures indicated that similar flaws might exist elsewhere in the bridge. A programme of examination and testing has been carried out and a considerable number of flaws have been detected.

The following methods were used:—

1. Visual inspection. This has detected two cracks.
2. Magnetic particle inspection.
3. Ultrasonic inspection.
4. Radiography.

Any one of these tests may not be reliable but a positive indication by two can be taken as being of fair order of reliability.

Visual inspection has been made throughout the bridge but instrument examination has been confined to sections where a high degree of public risk existed or as necessary elsewhere to obtain information.

The coverage of tests and the information gained has been sufficient to justify the conclusion that defects probably exist in all girders.

APPENDIX 4.

The following extracts are taken from the paper " How to use high-strength steel effectively " by A. L. Elliott, Bridge Engineer in Planning, Bridge Department of the California Division of Highways, which appeared in the " Engineering News Record " for 18th February, 1960.

It was included in Bonwick's report to Utah (Ex. 24) :—

" Progress in use of high-strength steels has accompanied advances in connecting methods. But recently, metallurgists have pushed ahead with still stronger steels ; progress in applying them to structures awaits development of suitable welding techniques. Strength is not the only consideration in selecting a steel for structural application. Other physical characteristics, such as toughness, ductility and weldability, are equally important, and these as well as strength are influenced by addition of small amounts of alloying elements to steel.

Adding carbon to steel, for example, inereases strength, but as more and more carbon is added, the steel becomes hard and brittle. Soon a point is reached where the steel is virtually useless for structural applications.

This carbon limit is rather low. Exceeding it may lead to disaster : steel might crack in cold weather or be shattered by vibration.

To be weldable without brittleness, steel must have a carbon content less than 0·25 per cent. Manganese also is critical and should be below 1·25 per cent.

Adoption of welding caused some major changes in steel use in California. Before 1951, use of rolled beams had been increasing steadily. For long spans, cover plates were riveted on, and sometimes the webs were split and plates inserted to increase the depth of the beams. But these were expensive solutions to the long-span problem. After a short period of working with welded beams, California began to utilize welding to the fullest extent in 1954. Since then, use of rolled beams has tapered off to a mere 21 tons in 1958.

Meanwhile, use of welded beams has been increasing steadily (18,500 tons in 1958). With this background of experience in welding, California was well prepared to make effective use of the high-strength steels when they became available.

When alloy structural steels were introduced about 20 years ago, California started to use them in bridges. In 1939, two large truss bridges were designed with over 2,200 tons of high-strength steel. Two years later, this new steel was covered by an ASTM tentative specification, A242 for alloy steel. After that, California used it extensively.

Well over 100,000 tons of A242 steel have been used in the past ten years in welded California highway bridges alone. This experience prepared the State's engineers for the next step to a higher stress grade.

When the early designs for the second Carquinez Straits Bridge were beginning to take shape about 1953, the new 90,000 lb./sq. in. yield-point steel had not yet been used in large quantity in any structure. Since engineers are traditionally conservative, the normal procedure would have been to wait ten or fifteen years until the new steel was well tested before specifying it for a bridge. But the potential economies, the saving in weight and the reduction in size of members that this higher-strength steel could effect, were too attractive to permit its adoption to be postponed.

So California engineers undertook a thorough investigation of this steel, which the maker, U.S. Steel Corp., called T-1. After testing more than 3,000 lb. of it, they decided to use it in a substantial portion of the Carquinez bridge. During fabrication of the trusses, some defects in welding were discovered, and during erection, some transverse cracks were found in longitudinal welds (E.N.R. Sept., 4, 1958, p. 42). The cracks frequently were close together, but in widely spaced groups.

All welds were rechecked and those that had cracks were removed and repaired. Less than 60 members were found to have cracks.

Since then, much time and money has been spent to duplicate the cracks deliberately and discover the cause. But duplication has not been possible. The cause can be deduced only from circumstantial evidence.

Part of the trouble seems to be attributable to the difficulty of getting welding rods that match the strength of the alloy steel. If this is true, special care should be taken to insure quality. A tack weld should be treated as meticulously as a major weld.

Use, care, and handling of low-hydrogen rods and proper selection of combinations of wire and flux become very important with higher-strength steels. So does general house-keeping in the shop ; dirt and moisture must be excluded. Welding high-strength steel is a precision operation, not to be undertaken casually or haphazardly. With proper care and attention to detail, successful welds are assured ".

APPENDIX 5.

Results of Tests on Samples from W.14 Girders, Extracted from D.S.L. Report, Ex. 194.

SUMMARY OF HARDNESS SURVEYS OF WELD HEAT-AFFECTED ZONES.

These surveys were made using an adaption of the B.W.R.A. method with a 5 kg. indenting load. A number of traverses from parent plate to weld pool were made through the H.A.Z. of representative welds. Peak hardness values of each traverse were noted and from these average peak Hv. for each weld was determined. The maximum Hv. obtained in each H.A.Z. is also quoted.

Girder	Sample	Zone	Hardness			
			Av. Peak	Max.	Parent Plate	Weld Deposit
W.14–S1–North end	Automatic Weld Web to Flange	Flange H.A.Z. (Western Side)	370	385	205	210
		Web H.A.Z. (Western Side)	300	300	..	210
		Flange H.A.Z. (Eastern Side)	310	315	200	205
	Manual Transverse Weld Cover Plate to Flange (Central Section)	Flange H.A.Z. (2nd Fillet)	345	360	..	..
		Cover Plate H.A.Z. (3rd Fillet)	340	340	..	..
	Manual Taper Weld Cover Plate to Flange (Eastern Side)	Flange H.A.Z. (3rd Fillet)	445	460	200	240
		Cover Plate H.A.Z.	..	325	190	..
	Manual Taper Weld Cover Plate to Flange (Western Side)	Flange H.A.Z. (3rd Fillet)	420	435	195	220
		Cover Plate H.A.Z. (2nd Fillet)	320	320	180	..
W.14–S3–North end	Automatic Weld Cover Plate to Flange	Flange H.A.Z.	410	425	195	235
		Cover Plate H.A.Z. ..	..	400	205	235
	Manual Transverse Weld Cover Plate to Flange	Flange H.A.Z. (2nd Fillet)	..	370	195	230
		Cover Plate H.A.Z. (3rd Fillet)	365	385	205	230
	Manual Taper Weld Cover Plate to Flange (Eastern Side)	Flange H.A.Z. (3rd Fillet)	475	485	180	230
		Cover Plate H.A.Z. (2nd Fillet)	..	325	205	230
	Manual Taper Weld Cover Plate to Flange (Western Side)	Flange H.A.Z. (3rd Fillet)	470	485	190	235
		Cover Plate H.A.Z. (2nd Fillet)	..	345	205	235
W.14–S2–South end	Manual Transverse Weld Cover Plate to Flange	Flange H.A.Z. (2nd Fillet)	..	390	185	235
		Cover Plate H.A.Z. (3rd Fillet)	..	350	185	240
	Manual Taper Weld Cover Plate to Flange (Eastern Side)	Flange H.A.Z. (3rd Fillet)	..	435	210	240
		Cover Plate H.A.Z. (2nd Fillet)	..	255	190	240

Appendix 5—*continued.*

KINGS BRIDGE—SUMMARY OF CHEMICAL ANALYSES.

Girder Component	Heat No.		Girder W.14–	Location in Girder (From North End)	Sample No.	Composition						
	B.H.P.	C.R.B.				C.	Mn.	Cr.	Si.	S.	P.	N.
Web ..	243,237	40	S.1†	Near splice with Heat 38, N. end	N.1W ..	·21	1·68	·15	·03	·022	·037	·005
				Adjacent Fracture, S. end	W.14–S.1, S. Fracture N. Surface	·22	1·67	·15				
			S.2	Near splice with Heat 38, S. end	C.R.B./16..	·26	2·03	·04				
				Near splice with Heat 38, S. end	C.R.B./17..	·26	2·01	·04				
				Near splice with Heat 38, S. end	C.R.B./18..	·25	2·01	·04				
			S.3	Near splice with Heat 38, S. end	C.R.B./25..	·24	1·95	·04				
				Near splice with Heat 38, S. end	C.R.B./26..	·24	1·94	·04				
				Near splice with Heat 38, S. end	C.R.B./27..	·25	1·97	·04				
			S.4	Near splice with Heat 38, S. end	C.R.B./34..	·25	1·96	·04				
				Near splice with Heat 38, S. end	C.R.B./35..	·25	1·95	·04				
				Near splice with Heat 38, S. end	C.R.B./36..	·25	1·96	·04				
	D.S.L. Mean (Neglecting Girder S.1)					·25	1·97	·04				
	B.H.P. Certificate					·23	1·75	·050*	·06	·021	·015	..

* Analysed for Chromium but not reported in Test Certificate.

† This is almost certainly Heat 38, not Heat 40.—Commissioners.

Girder Component	Heat No.		Girder W.14–	Location in Girder (from N. end)	Sample No.	Composition						
	B.H.P.	C.R.B.				C	Mn	Cr	Si	S	P	N
Flange (Bottom)	7,890	55	S 1	Plate 2, North end ..	CRB/40	·24	1·77	·24				..
				„ „ South end ..	CRB/41	·24	1·76	·24				..
				Plate 3, North end ..	CRB/42	·25	1·78	·24				..
				„ „ South of S. Fracture	CRB/43	·26	1·80	·25				·012
				„ „ North of S. Fracture	S1F ..	·25	1·76	·25	·34	·031	·020	..
			S 2	Plate 1, North end ..	CRB/44	·25	1·78	·24				
				Plate 1, above North end of Cover	N2F	·27	1·77	·25	·33	·034	·021	·011
			S 3	Plate 2, North end ..	CRB/45	·28	1·80	·25				
				Plate 3, South end ..	CRB/46	·26	1·80	·25	·32			·011
			S 4	Plate 2, North end ..	CRB/47	·22	1·73	·24				
				Plate 3, North of S. Fracture	S4F	·22	1·72	·24	·30	·027	·018	·008
				„ „ South of S. Fracture	CRB/48	·23	1·77	·24				
	D.S.L. Mean					·25	1·77	·24	·32	·030	·020	·011
	B.H.P. Certificate					·21	1·70	·23*	·30	·022	·025	

* Analysed for Chromium but not reported in Test Certificate

APPENDIX 5—*continued.*

Girder Component	Heat No.		Girder W.14	Location in Girder (from N. end)	Sample No.	Composition						
	B.H.P.	C.R.B.				C	Mn	Cr	Si	S	P	N
Flange (Bottom)	7,892	56	S 1	Plate 1, near North end ..	CRB/50	·25	1·73	·24				·010
				,, ,, near North end ..	N1F	·26	1·70	·25				
				,, ,, South end ..	CRB/51	·26	1·75	·24				
			S 2	Plate 2, North end ..	CRB/52	·26	1·73	·24				
				,, ,, South End ..	CRB/53	·27	1·72	·25				
				Plate 3, North end ..	CRB/54	·26	1·73	·24				
				,, ,, South of S. Fracture	CRB/55	·25	1·72	·24	·22			·009
			S 3	Plate 1, near North end ..	CRB/56	·25	1·73	·24				
				,, ,, South of N. Fracture	N3F	·26	1·73	·23	·22	·031	·018	·008
			S 4	Plate 1, near North end ..	CRB/57	·25	1·73	·24				
				,, ,, North of N. Fracture	CRB/58	·26	1·74	·24				·009
	D.S.L. Mean					·26	1·73	·24				
	B.H.P. Certificate					·23	1·58	·24*	·195	·026	·017	

* Analysed for Chromium but not reported in Test Certificate

Girder Component	Heat No.		Girder W.14	Location in Girder (from N. end)	Sample No.	Composition						
	B.H.P.	C.R.B.				C	Mn	Cr	Si	S	P	N
Cover Plate	242,503	4	S 4	Plate 3, South end (near S. Fracture)	S4C ..	·24	1·84	·06	·06	·038	·039	·005
	B.H.P. Certificate					·21	1·72	·070*	·070	·029	·042	
Cover Plate	267,356	16	S 1	Plate 1, North end ..	N1C ..	·24	1·71	·24	·21	·026	·019	
	B.H.P. Certificate					·21	1·70	·10*	·145	·021	·010	
Cover Plate	7,878	52	S 2	Plate 1, North end ..	N2C ..	·19	1·87	·25	·31	·028	·016	·009
	B.H.P. Certificate					·20	1·77	·25*	·30	·023	·021	
Cover Plate	7,886	53	S 3	Plate 1, North end ..	N3C ..	·26	1·78	·17	·31	·032	·017	·008
	B.H.P. Certificate					·21	1·69	·18*	·28	·024	·024	
Cover Plate	243,241	58	S 1	Plate 3, South end (near S. Fracture)	S1C ..	·21	1·92	·06	·04	·021	·012	·004
	B.H.P. Certificate					·22	1·69	·25*	·26	·024	·020	

* Analysed for Chromium but not reported in Test Certificate

6352/63—8

APPENDIX 5—*continued.*

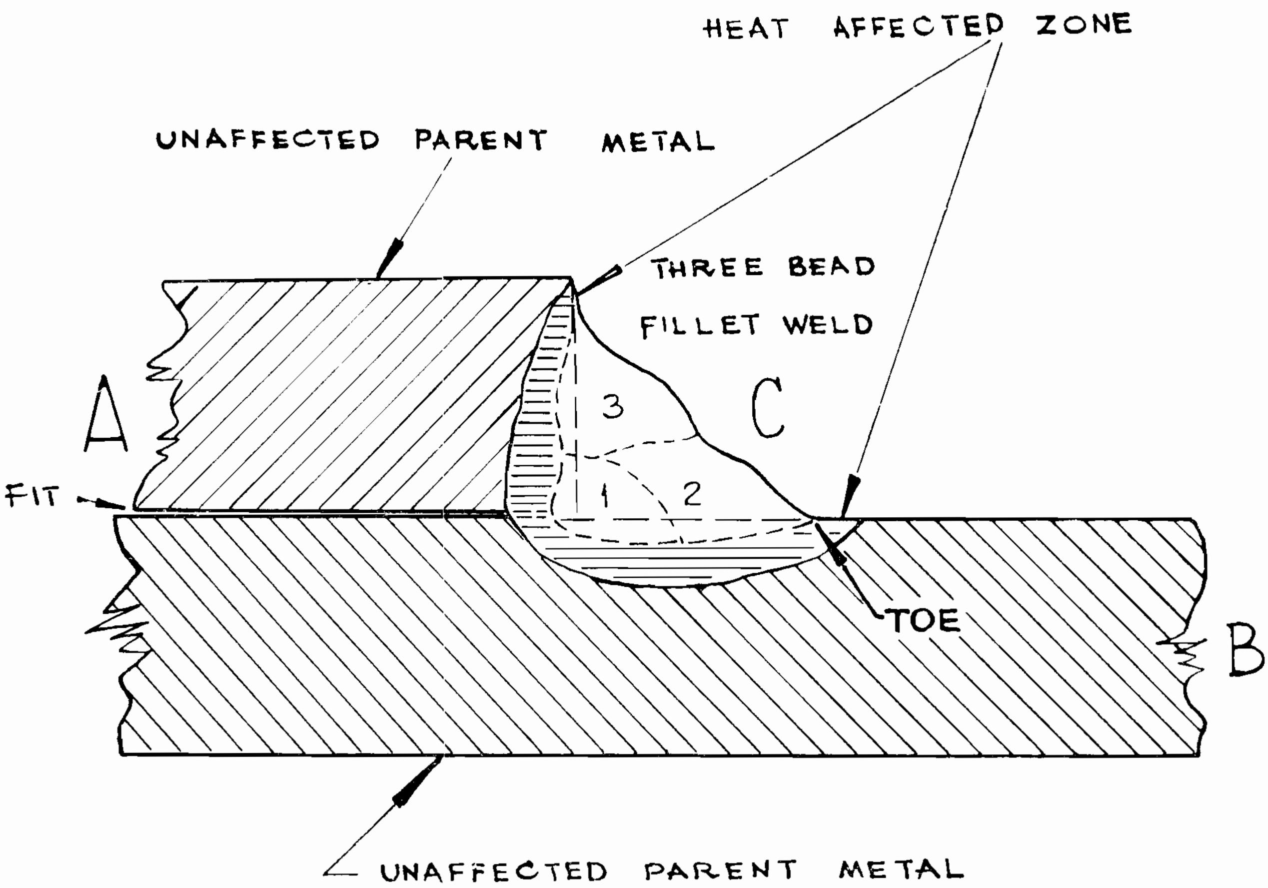
MANGANESE PLUS CHROMIUM TOTALS, CALCULATED CARBON EQUIVALENTS (C.E.)* AND DERIVED WELDABILITY INDICES (W.I.)*

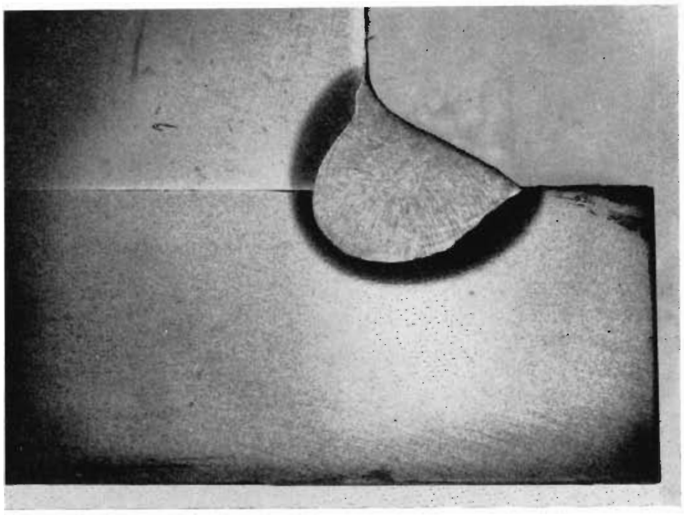
Sample No.	C.R.B., Heat No.	Mn. + Cr.	C.E.	W.I.
N.1W	40	1·83	0·309	C
W.14–1, S.Fr., N. Surface	,,	1·82	0·319	C
C.R.B./16	,,	2·07	0·366	D
C.R.B./17	,,	2·05	0·365	D
C.R.B./18	,,	2·05	0·355	D
C.R.B./25	,,	1·99	0·342	C
C.R.B./26	,,	1·98	0·341	C
C.R.B./27	,,	2·01	0·353	C
C.R.B./34	,,	2·00	0·352	C
C.R.B./35	,,	1·99	0·352	C
C.R.B./36	,,	2·00	0·352	C
C.R.B./40	55	2·01	0·353	C
C.R.B./41	,,	2·00	0·352	C
C.R.B./42	,,	2·02	0·363	D
C.R.B./43	,,	2·05	0·375	D
S.1F	,,	2·01	0·363	D
C.R.B./44	,,	2·02	0·363	D
N.2F	,,	2·02	0·384	D
C.R.B./45	,,	2·05	0·395	D
C.R.B./46	,,	2·05	0·375	D
C.R.B./47	,,	1·97	0·331	C
S.4F	,,	1·96	0·330	C
C.R.B./48	,,	2·01	0·343	C
C.R.B./50	56	1·97	0·361	D
N.1F	,,	1·95	0·370	D
C.R.B./51	,,	1·99	0·372	D
C.R.B./52	,,	1·97	0·371	D
C.R.B./53	,,	1·97	0·381	D
C.R.B./54	,,	1·97	0·371	D
C.R.B./55	,,	1·96	0·360	D
C.R.B./56	,,	1·97	0·361	D
N.3F	,,	1·96	0·370	D
C.R.B./57	,,	1·97	0·361	D
C.R.B./58	,,	1·98	0·371	D
S.4C	4	1·90	0·338	C
N.1C	16	1·95	0·350	C
N.2C	52	2·11	0·309	C
N.3C	53	1·95	0·366	D
S.1C	58	1·98	0·312	C

NOTE :—* Weldability Index derived from carbon equivalent calculated from formula given in B.W.R.A. booklet "Arc-Welding of Low Alloy Steels" by B. J. Bradstreet—for Class 6 (low hydrogen) electrodes.

APPENDIX 6.

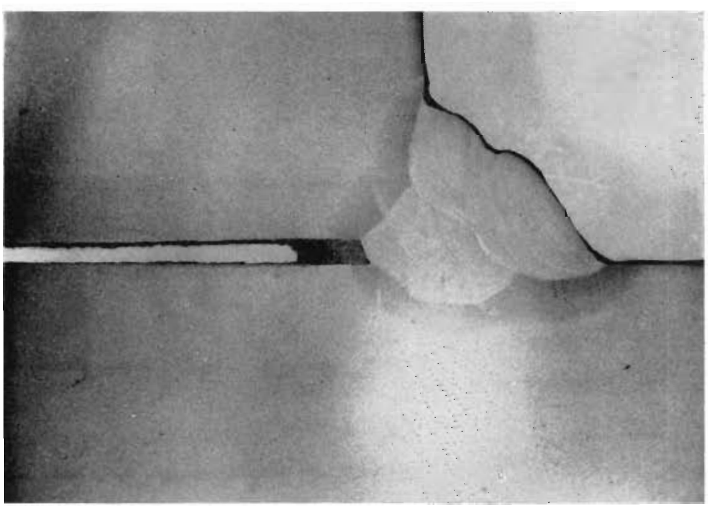
FIGURES REFERRED TO IN THE TEXT OF THE REPORT.


HEAT AFFECTED ZONE
UNAFFECTED PARENT METAL
THREE BEAD
FILLET WELD
A
3
C
FIT
1
2
TOE
B
UNAFFECTED PARENT METAL
Fig. 1.



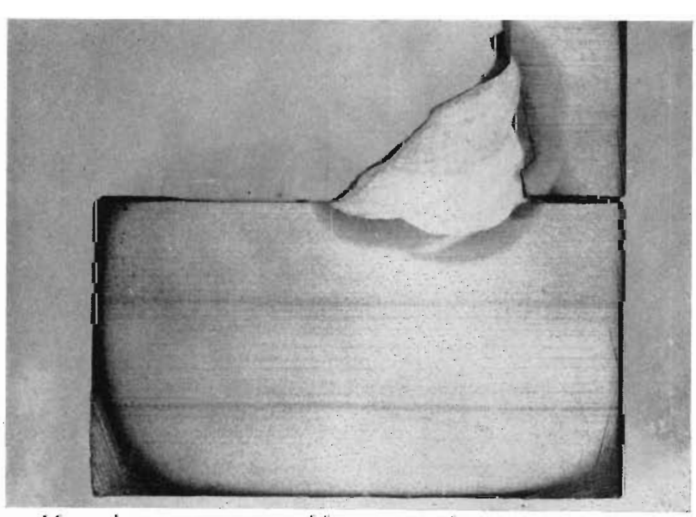
Automatic longitudinal weld—cover plate to flange.
(Girder W.14–S.4, south end.)

× 2



Manual taper weld—cover plate to flange.
The gap between cover plate and flange was greater in this section than in any of the others examined to date.
(Girder W.14–S.1, north end.)

× 2

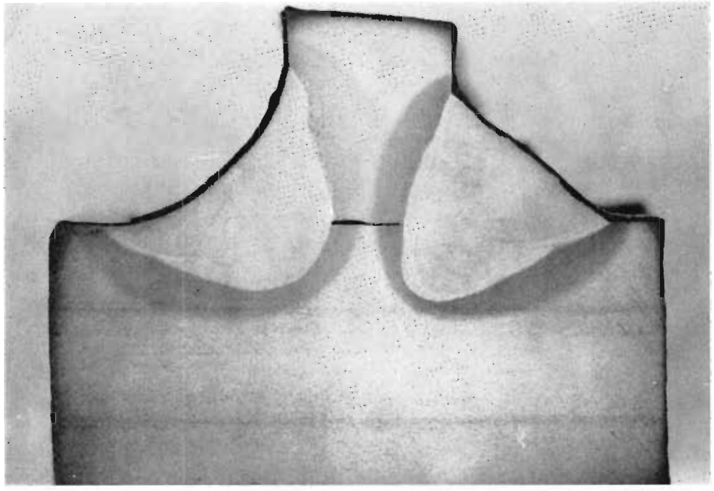


Manual transverse weld—cover plate to flange.
(Girder W.14–S.1, north end.)

× 2

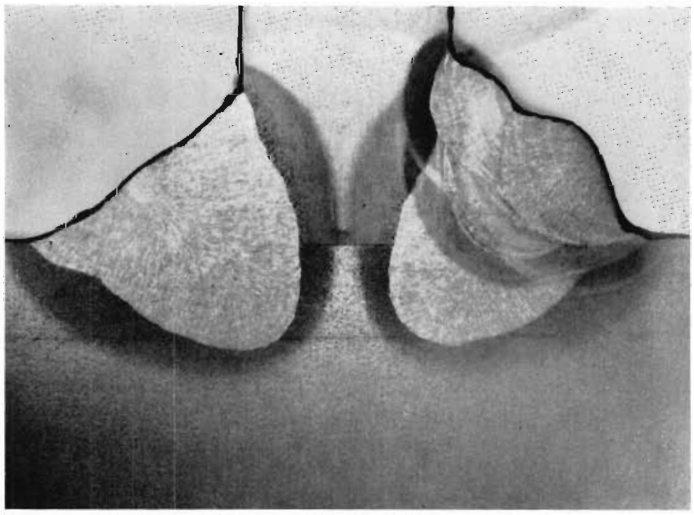
Fig. 1 – *(continued).*

TYPICAL WELD STRUCTURES



Automatic longitudinal weld—web to flange.
(Girder W.14–S.1, north end.)

× 2



Web to flange weld, repaired by manual welding.
(Girder W.14–S.3, north end.)

× 2

Fig. 1 – *(continued)*.

Fig. 2.
Fracture in girder W.14–2, southern end, looking south.

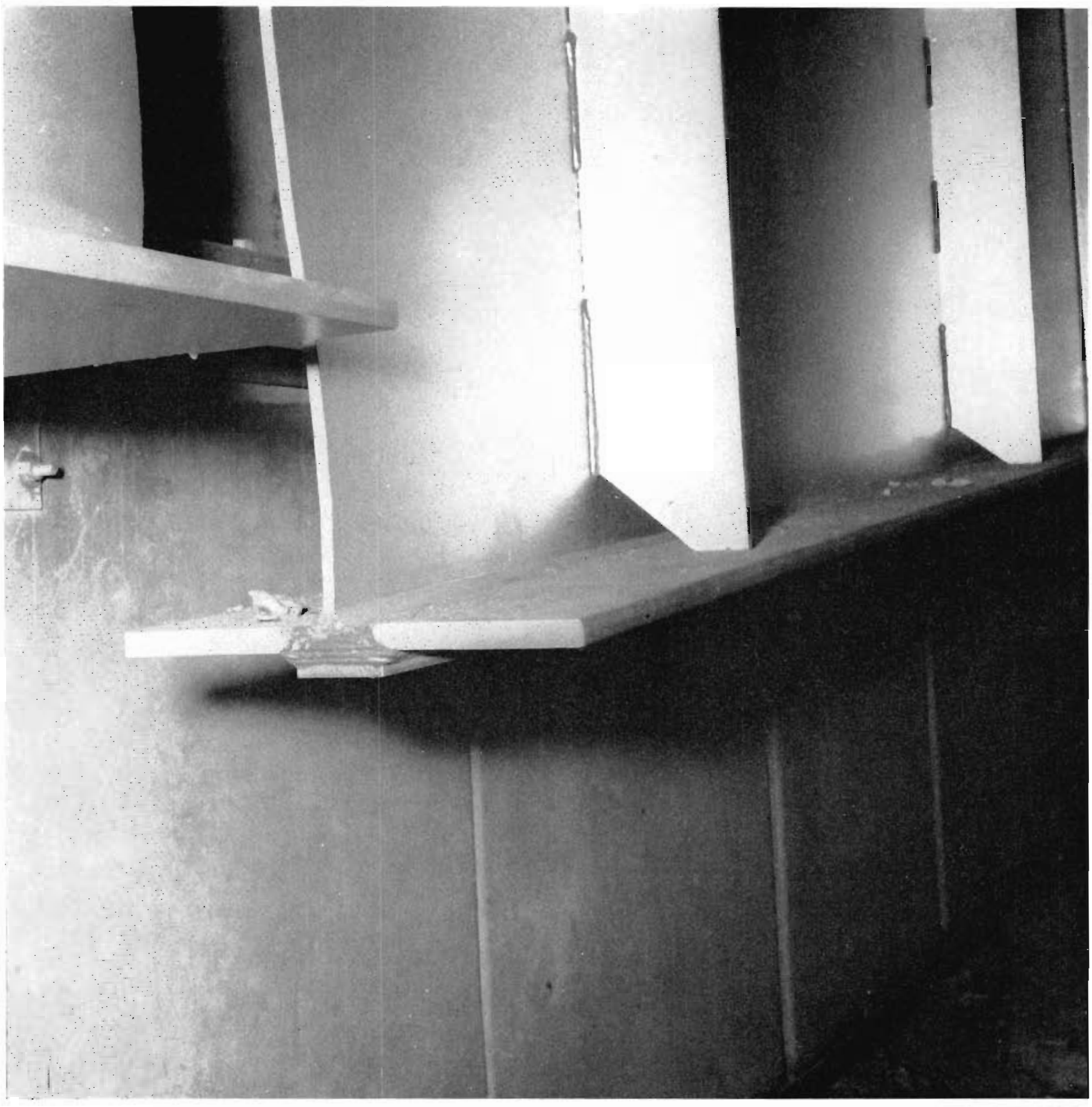


Fig. 3.

Fracture in girder W.14–1, southern end, looking north.

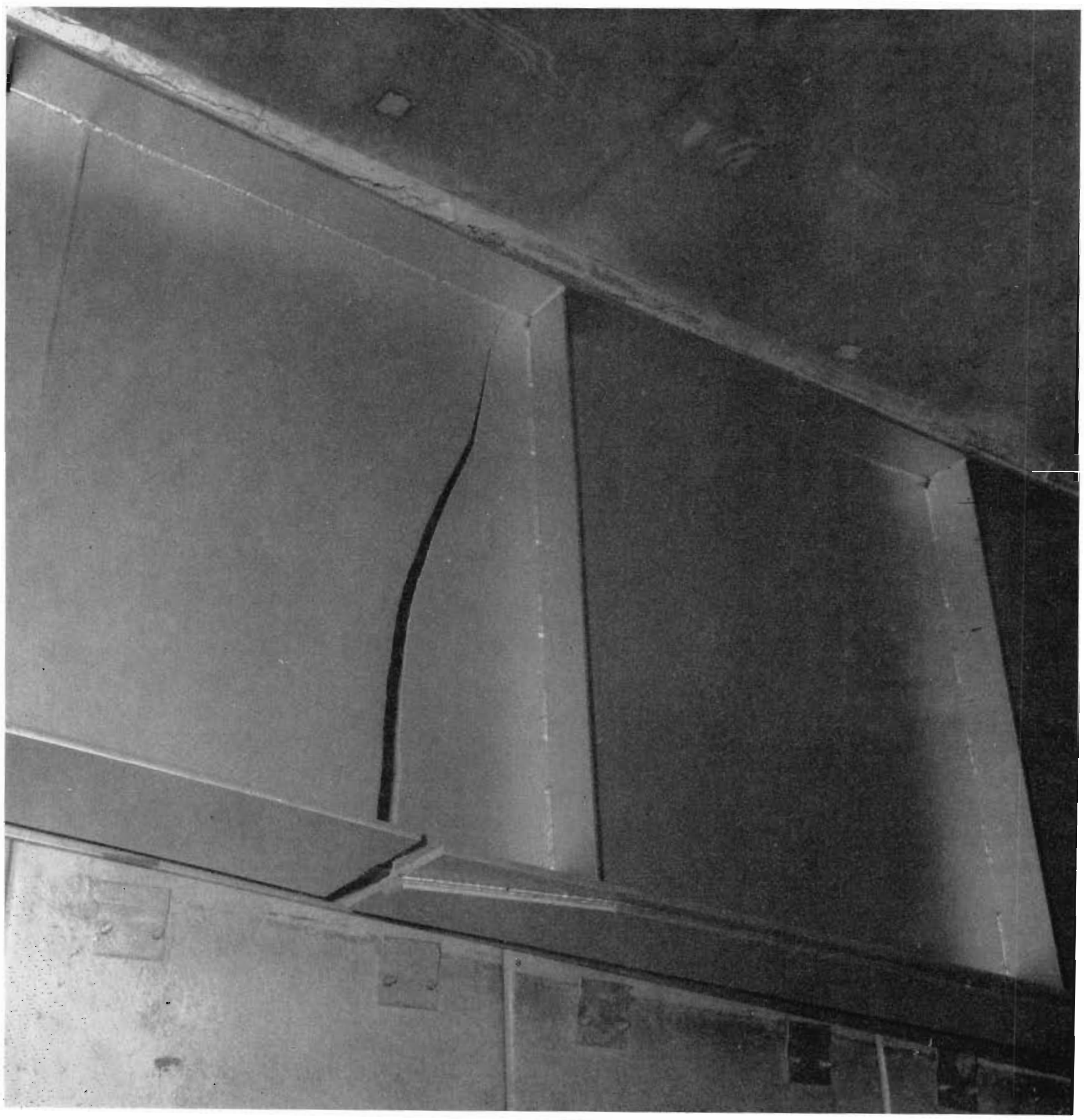
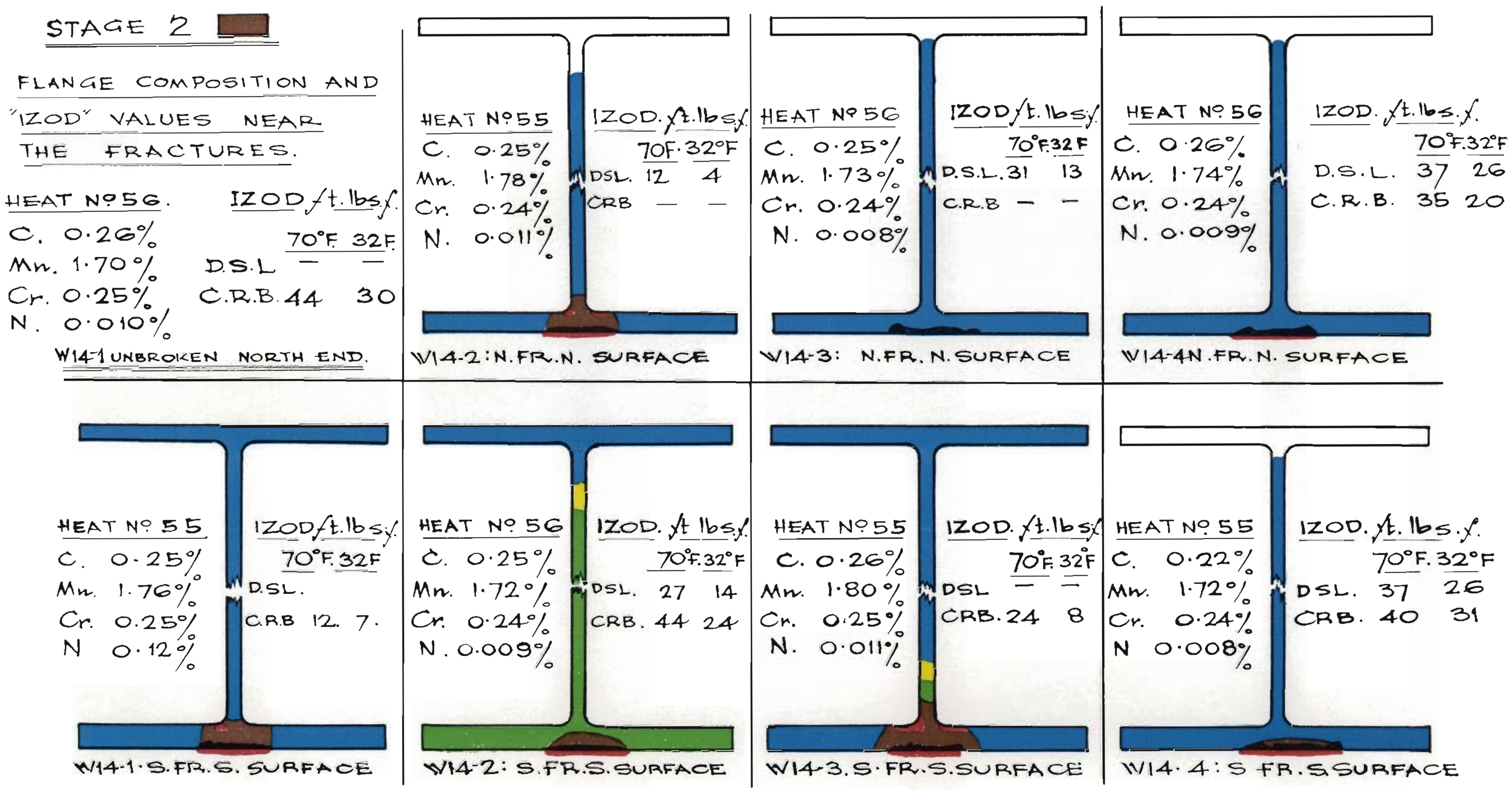


Fig. 4.
Fracture of girder W.14–4, northern end, showing cover plate end.

Fig. 6.—" Features of the seven fractures in the failed W.14 span ".

The legend applicable thereto is :—

Black	Toe crack.
Red	Priming paint.
Brown	Primary brittle fracture.
Green	Extension of primary brittle fracture prior to 10th July, 1962.
Yellow	Fatigue fracture.
Blue	Final fracture associated with the collapse.



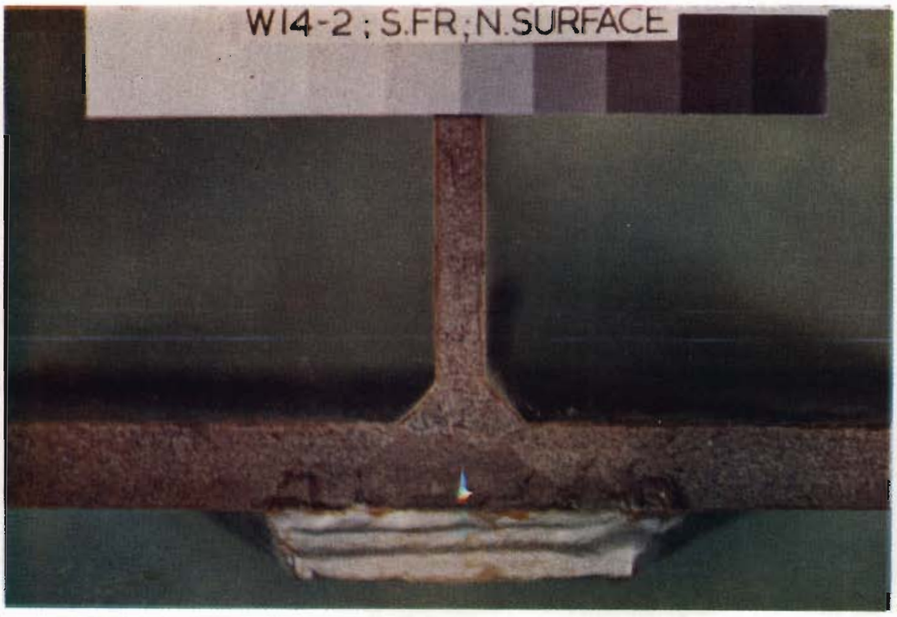


Fig. 5

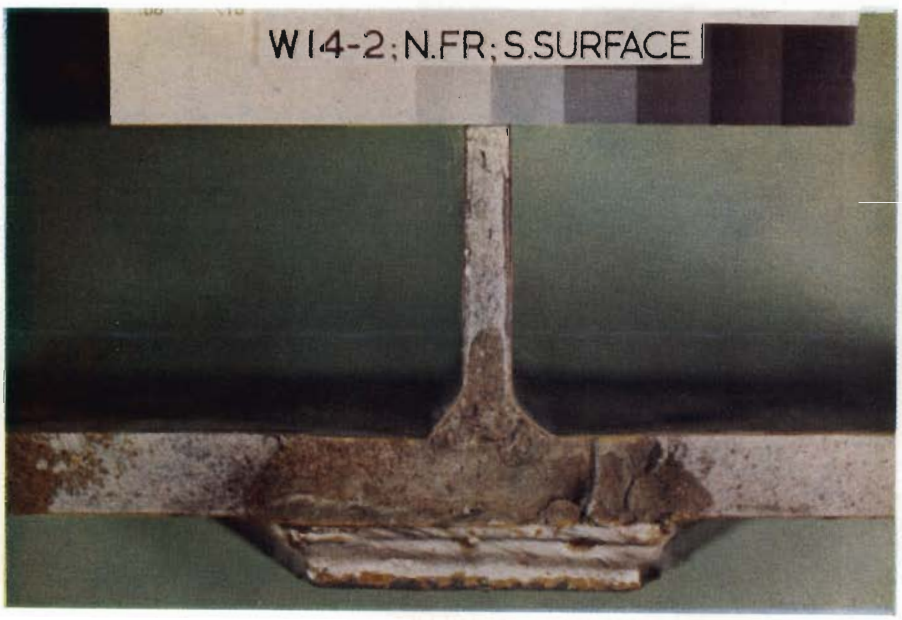


Fig. 7

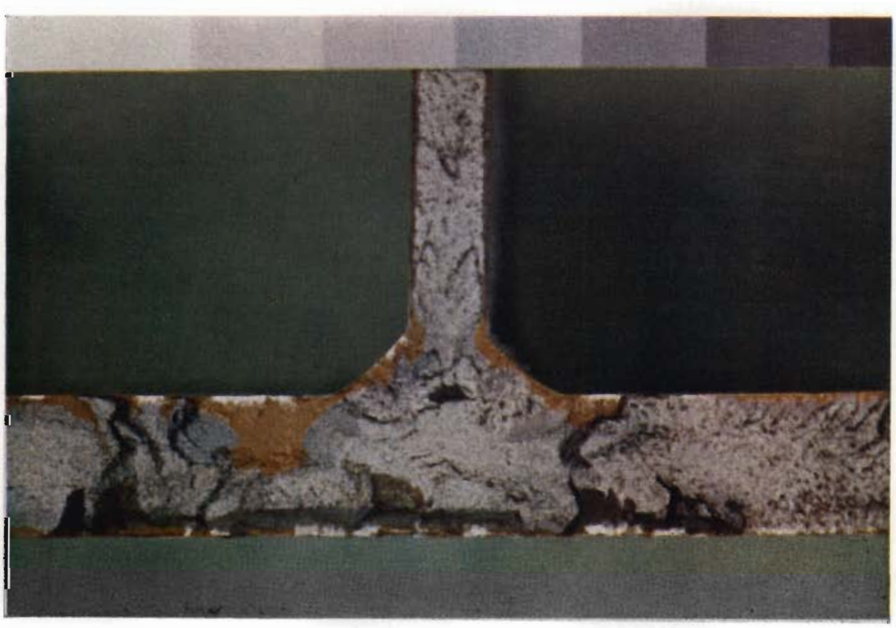


Fig. 8

W14-3 S.Fracture ; South Surface

Red=Priming Paint. White=Final Aluminium Paint.

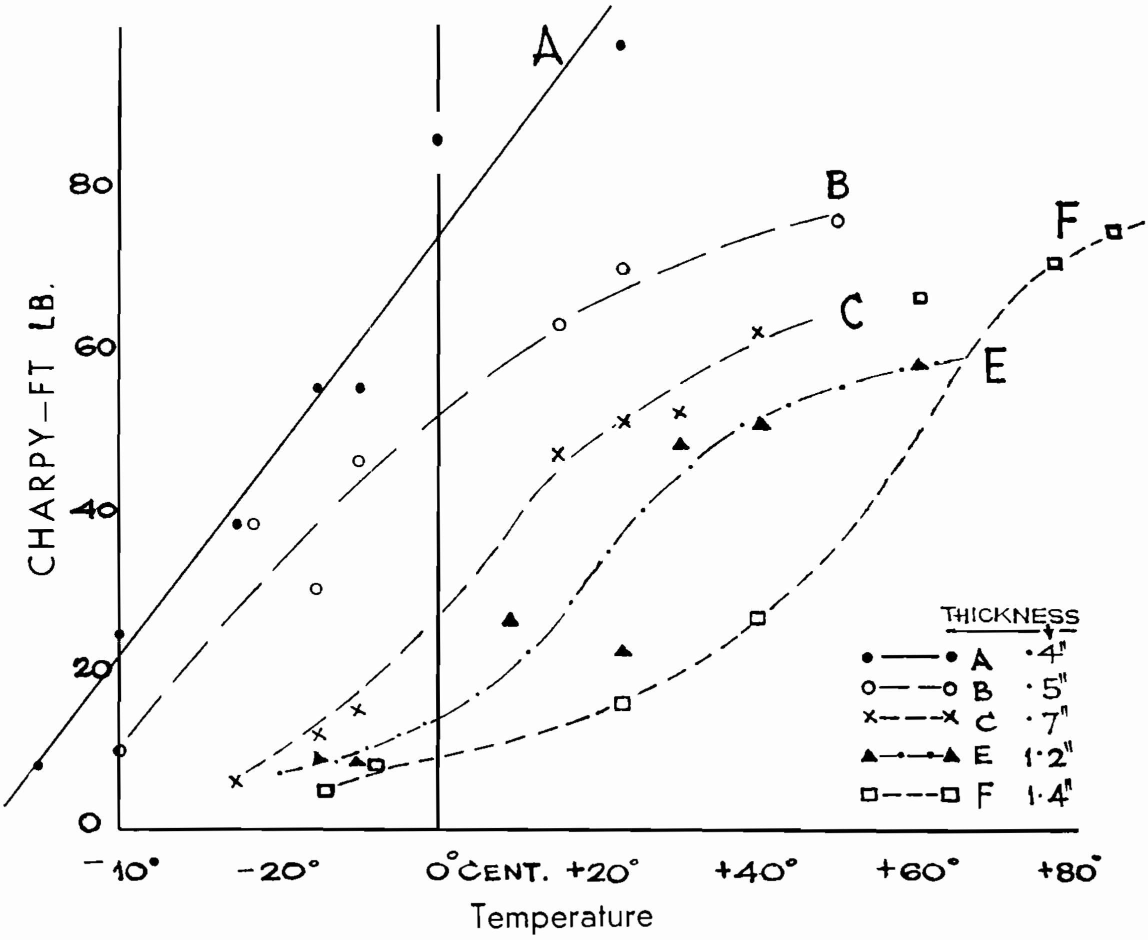

A
B
C
E
F
CHARPY–FT. LB.
80
60
40
20
0
THICKNESS
A ·4"
B ·5"
C ·7"
E 1·2"
F 1·4"
−10°
−20°
0° CENT.
+20°
+40°
+60°
+80°
Temperature

Fig. 9.

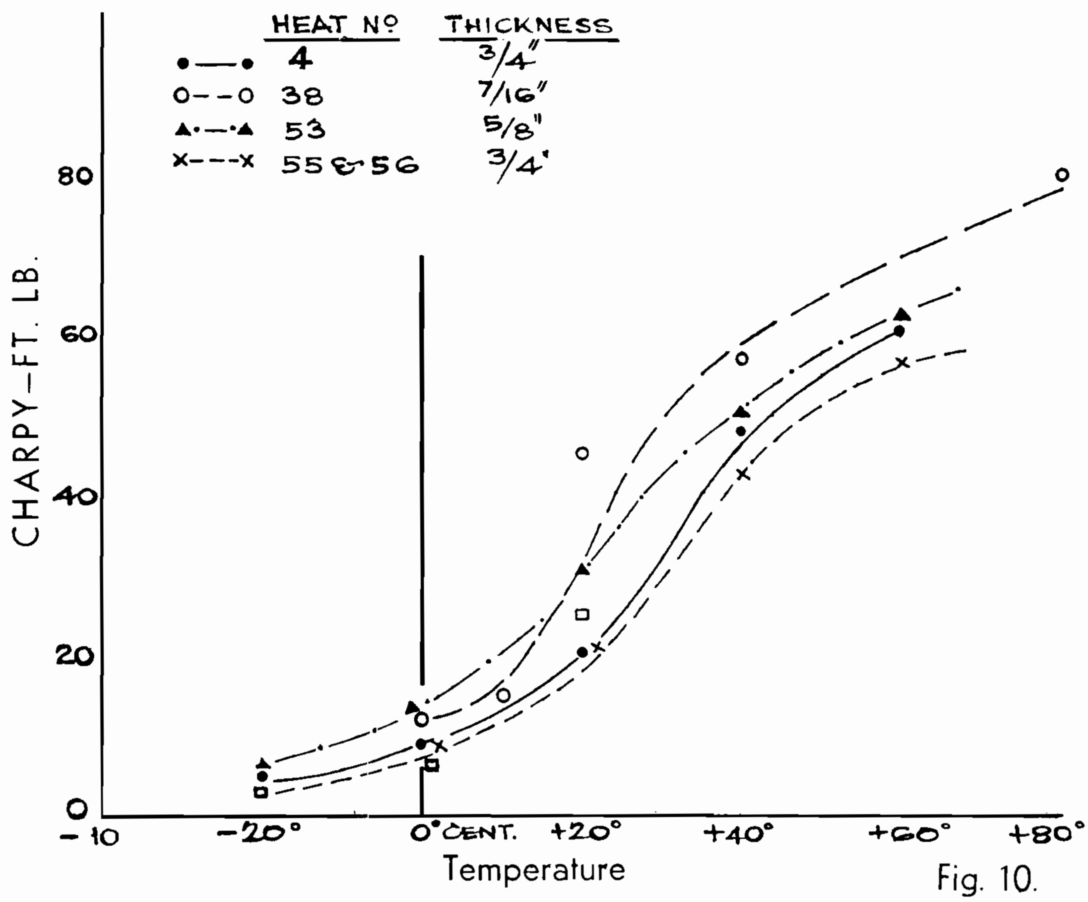

HEAT Nº
THICKNESS
4
3/4"
38
7/16"
53
5/8"
55 & 56
3/4"
CHARPY–FT. LB.
80
60
40
20
0
−10
−20°
0° CENT.
+20°
+40°
+60°
+80°
Temperature

Fig. 10.

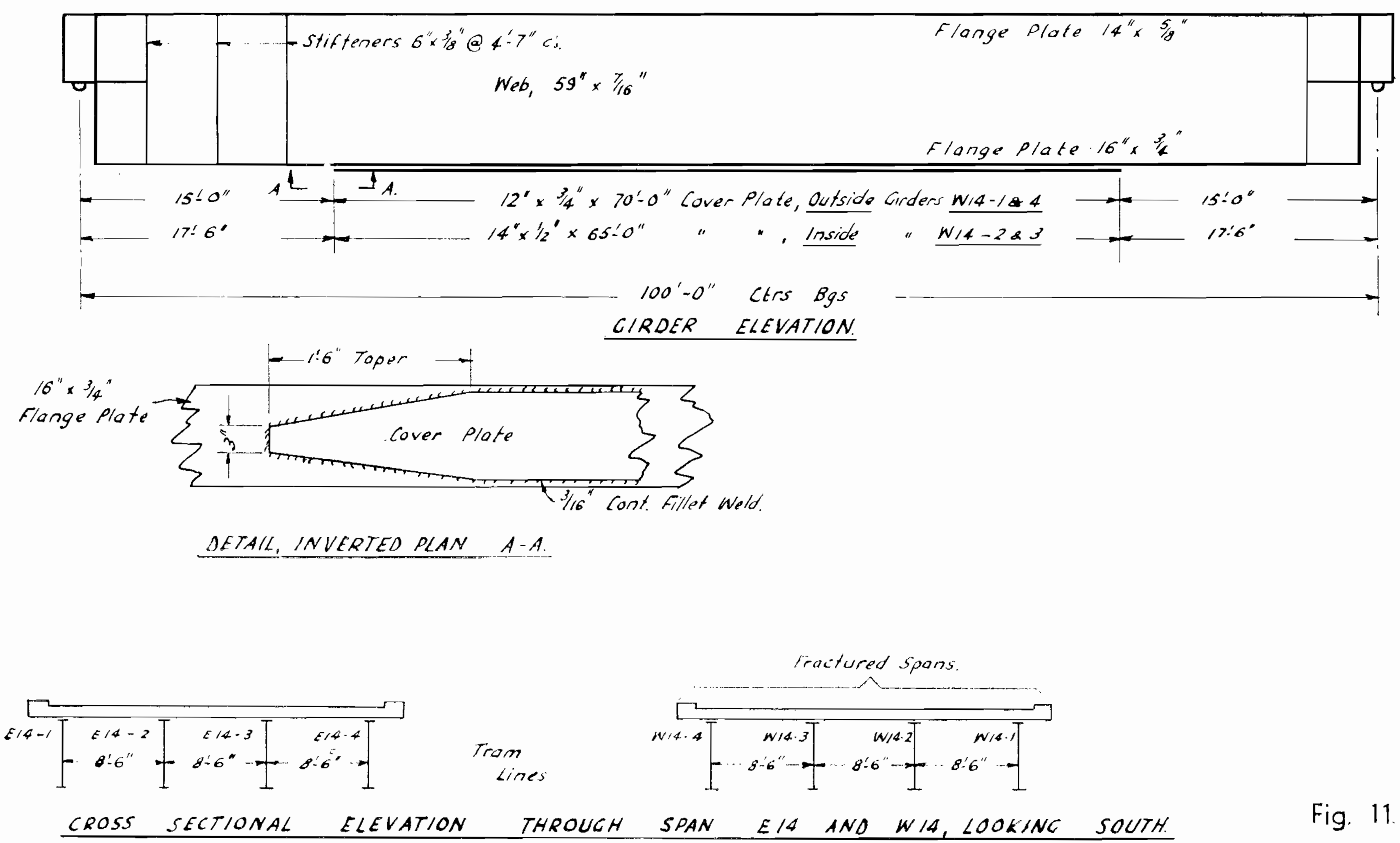
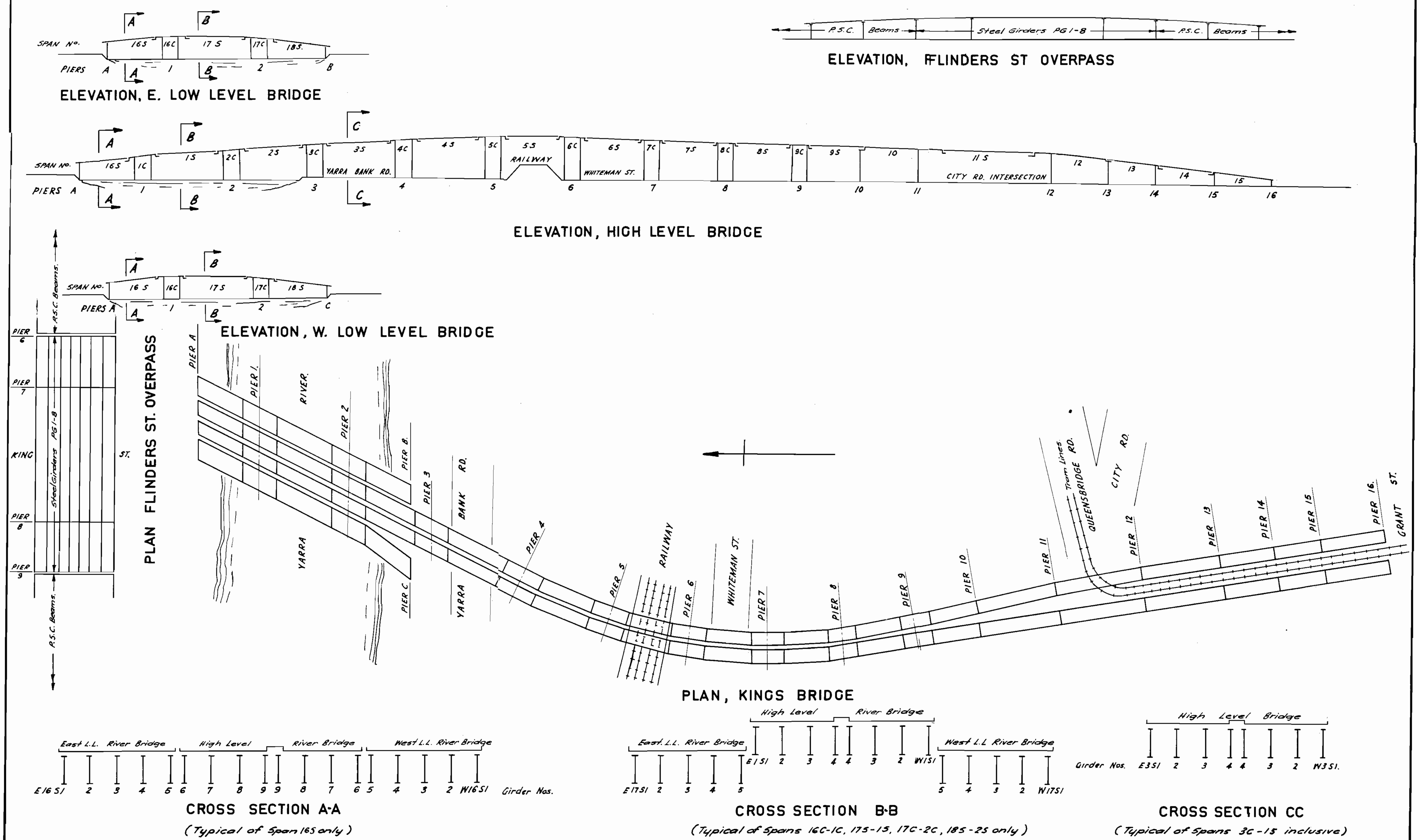


Fig. 11.

DESIGNED		M.M.B.W. PLANNING AND HIGHWAYS BRANCH	SCALE Not to Scale
DRAWN B.S.S. 27/7/62	CHIEF PLANNER	KINGS BRIDGE FAILURE 1962	DRAWING NO
TRACED B.S.S.	FILE NO.	FRACTURED GIRDERS	R23-00-116
CHECKED		W14-1,2,3,4.	
	F.B L.B.		

Fig. 12.



INDEX	DATE	REVISION	APPD.
E			
D			
C			
B			
A			

GENERAL NOTES

ENGINEER METROPOLITAN HIGHWAYS / /

CHIEF PLANNER / /

DESIGNED / /		FILE NO:
DRAWN	B.S.S. 3/8/62	JOB NO:
TRACED	B.S.S. 3/8/62	CONTRACT NO:
CHECKED / /		
/ /		800' PLAN NO:
SUPERVISING ENGINEER		F.B. L.B.

M.M.B.W. PLANNING AND HIGHWAYS BRANCH

KINGS BRIDGE — INVESTIGATION

KEY PLAN

SCALE: Not To Scale.
SHEET OF
DRAWING NO.
R 23-00-120