1971

VICTORIA

REPORT

OF

ROYAL COMMISSION

INTO THE

FAILURE OF WEST GATE BRIDGE

PRESENTED TO BOTH HOUSES OF PARLIAMENT PURSUANT TO SECTION 7 OF THE WEST GATE BRIDGE ROYAL COMMISSION ACT 1970, No. 7989

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ROYAL COMMISSION OF INQUIRY INTO THE FAILURE OF WEST GATE BRIDGE.

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

YOUR EXCELLENCY,

By virtue of the Authority of the *West Gate Bridge Royal Commission Act* 1970, (No. 7989 of 1970), and in pursuance and execution of Letters Patent dated the 21st day of October, 1970, under the Seal of the State of Victoria, whereby 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—

> The Honourable Edward Hamilton Esler Barber, a Judge of the Supreme Court of Victoria ; Professor Frank Bertram Bull, M.A., B.Sc. (Eng)., Professor of Civil Engineering in the University of Adelaide ; Sir Hurbert Shirley-Smith, C.B.E., B.Sc. (Eng)., F.I.C.E., F.C.G.I., F.I.C.—

a Commission authorizing and appointing us to inquire into and report to Your Excellency, upon—

> the circumstances surrounding and the cause or causes direct and indirect of the failure on the 15th day of October, 1970, of the steel span between piers 10 and 11 of the bridge known as " West Gate Bridge " being constructed for the Lower Yarra Crossing Authority over the Lower Yarra River at Spotswood ;

and it was directed and appointed that the Honourable, Mr. Justice Edward Hamilton Esler Barber should be Chairman of the said Commission.

And whereas on the 23rd February, 1971, the Governor in Council, under the powers conferred by Section 2 (3) of the aforesaid *West Gate Bridge Royal Commission Act* 1970, and upon a request of the Chairman under the said Section 2 (3)—extended the terms of reference of the said Commission referred to in Section 2 (1) of the said Act, by adding thereto the following:—

> " to inquire into and report upon whether any aspect of the design of the steel span between piers 10 and 11 is inadequate or undesirable ".

We, the undersigned, Chairman and members of the Commission, having duly inquired 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 the preliminary sitting of the Commission was published in the " Herald ", the ' Age ", the " Australian ", and " Sun " newspapers on the 24th October, 1970, and advice of the sittings of the Commission for taking of evidence was published in the " Herald ", the " Age ", the " Australian ", and " Sun " newspapers on the 30th day of October, 1970, and the 27th day of January, 1971.

2. SITTINGS OF THE COMMISSION.

The Commission held a Preliminary Hearing on the 28th day of October, 1970, and thereafter between the 5th November, 1970 and the 1st day of December, 1970, and the 3rd day of February, 1971, and the 15th day of June, 1971, sittings upon 80 days. The Commission heard evidence viva voce of 52 witnesses, a list of whom is contained in Appendix A of this Report. Three hundred and nineteen exhibits were received in evidence as set out in Appendix B.

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. B. L. MURRAY, Q.C., (Solicitor-General), with
Mr. J. G. GORMAN, Q.C., (Mr. Gorman withdrew on the 3rd day of February, 1971), and
Mr. J. W. J. MORNANE, appeared to assist the Commission.

The following counsel were granted leave to appear :—

Mr. K. A. AICKIN, Q.C., with Mr. S. CHARLES, instructed by Mallesons, appeared on behalf of the Lower Yarra Crossing Authority.

Mr. W. KAYE, Q.C., with Mr. S. E. K. HULME, Q.C., and Mr. R. SEARBY, instructed by Blake and Riggall, appeared on behalf of John Holland (Constructions) Pty. Ltd.

Mr. B. W. BEACH, Q.C., and Mr. J. E. BARNARD, instructed by Clarke, Rowan and Richards, appeared on behalf of Freeman, Fox and Partners.

Mr. J. McI. YOUNG, Q.C., with Mr. R. C. TADGELL, instructed by Arthur Robinson and Co., appeared on behalf of Maunsell and Partners.

Mr. P. MURPHY, Q.C., with Mr. J. A. GOBBO, and Mr. A. GOLDBERG, instructed by Ellison, Hewison and Whitehead, appeared on behalf of World Services and Construction Pty. Ltd.

Mr. P. A. COLDHAM, Q.C., with Mr. D. DAWSON and Mr. J. S. WINNEKE, instructed by Aitken, Walker and Strachan, appeared on behalf of Broken Hill Proprietary Company Limited, until, on the 25th February, 1971, counsel sought and were granted leave to withdraw.

Mr. R. K. FULLAGAR, Q.C., and Mr. K. F. COLEMAN, instructed by Smith and Emmerton, appeared on behalf of McPherson's Limited, until, on the 3rd February, 1971, counsel sought and were granted leave to withdraw.

Mr. F. BROOKING, Q.C., and Mr. H. STOREY, instructed by Corr and Corr, appeared on behalf of Pioneer Concrete (Vic.) Pty. Ltd., until, on the 9th November, 1970, counsel sought and were granted leave to withdraw.

Mr. K. F. MARKS, Q.C., and Mr. P. HASE, instructed by Morris Blackburn and Company, appeared on behalf of the Victorian Trades Hall Council, (although granted leave to appear, in fact, counsel did not appear after 28th October, 1970).

A number of corporations, companies and firms are of necessity referred to with great frequency in this Report. We have adopted the general practice of using the full name where first mentioned, and thereafter have used initials or a short form of the name, unless there is some particular reason for using the full name. The abbreviations adopted are as follows :—

The Country Roads Board (of Victoria)	CRB
The Broken Hill Proprietary Company Limited	BHP
The Lower Yarra Crossing Authority	The Authority
John Holland (Constructions) Pty. Ltd.	JHC
Freeman Fox and Partners	FF & P
World Services and Constructions Co. Ltd.	WSC
Maunsell and Partners	Maunsell
G. Maunsell and Partners (London)	Maunsell London
Redpath Dorman Long and Company Ltd.	RDL
Werkspoor Utrecht N.V.	Werkspoor
Wescon N.V. Utrecht	Wescon

It will also be noticed that in the text of the Report, for convenience of expression, we have treated companies as if they were firms, and used the plural form of pronoun.

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 :—

BARMBY, B. D.	B.C.E., F.I.E. Aust., General Manager, Engineering Division, JHC.
BAXTER, J. W.	B.Sc., F.I.C.E., F.I.E. Aust, Managing Partner of Maunsell, London, Partner in Maunsell.
Birkett, E. M.	B.Sc. (Eng)., F.I.C.E., F.I.E. Aust., Managing Partner of Maunsell, Partner of Maunsell, London.
BROWN, Dr. W. C.	O.B.E., B.Sc., D.I.C., Ph.D., M.I.C.E., Partner, FF & P since January, 1970.
BURBURY, T. V.	B.E., M.I.E. Aust., Section Engineer, JHC (East Side).
*CROSSLEY, P. J. F.	B.A., M.I.C.E., Site Engineer, FF & P.
ENNESS, E.	Senior Inspector of Steel Work for FF & P.
FERNIE, G. N.	B.C.E., M.I.E. Aust., Partner of Maunsell.
FREEMAN, Sir R.	C.V.O., C.B.E., M.A., F.I.C.E., Senior Partner, FF & P since 1963.
GRASSL, J. J.	Dip. Eng. (Vienna), Consulting Engineer.
GRIEVE, R. K.	B.E., M.I.E. Aust., Engineer, Maunsell.
HALSALL, E. C.	Rigger, WSC (East Side), JHC (West Side).
HARDENBERG, G.	M.C.E., Senior Representative of Werkspoor-Utrecht, Wescon and WSC in Melbourne.
HART, K.	Jacking Technician for WSC and JHC.
HAYLOCK, N.	B.C.E., M.I.E. Aust., Engineer representing Royal Commission during demolition of span 10–11.
*HINDSHAW, J.	M.I.C.E., Resident Engineer, FF & P for project.
HOLLAND, C. V.	B.C.E., F.I.E. Aust., Chairman and Managing Director, JHC.
JAMES, H. B.	B.C.E., F.I.C.E., Associate Partner, Maunsell.
KERENSKY, Dr. O. A.	C.B.E., B.Sc., D.Sc., F.R.S., F.I.C.E., F.I. Struct. E., Partner, FF & P.
MCINTOSH, D. F.	M.I., Struct. E., Site Engineer, FF & P.
*MILLER, I. H.	B.C.E., M.I.E. Aust., Construction Manager, JHC, Assistant Project Manager.
MURRAY, Prof. N. W.	B.E., Ph.D., M.I.E. Aust., Professor of Civil Engineering, Monash University.
NIXON, T. R.	B.C.E., M.I.E. Aust., General Manager, Southern Zone, JHC, Project Manager.
RICHMOND, Dr. B.	B.Sc. (Eng)., Ph.D., M.I.C.E., Senior Engineer, Maunsell, London.
RIGGALL, J. T.	Dip. C.E., Section Engineer, JHC.
ROBERTS, Sir G.	B.Sc., F.R.S., F.I.C.E., F.I., Struct. E., Partner, FF & P. (retired June 30th, 1969, although continued in charge of West Gate Bridge Project until December 31st, 1969).
RODERICK, Prof. J. W.	M.A., Ph.D., F.I.C.E., F.I.E. Aust., Professor of Civil Engineering, Sydney University.
RUGLESS, R.	B.Sc., (Eng)., Planning Engineer, JHC.
SEWELL, A. P.	M.A., M.I.C.E., M.I.E. Aust., Senior Design Engineer, JHC.
SCHUT, J. H.	Managing Director, WSC.
SCHOT, R.	Area Supervisor, WSC, Adviser, JHC (West Side).
SIMPSON, C. V. J.	B.Eng., M.I.C.E., Section Engineer, FF & P (East Side).

SCHROEDER, J.	M.E., Managing Director, Wescon, Chairman, WSC.
SPEE, J.	Area Supervisor, WSC, Adviser, JHC (East Side).
STEVENS, Prof. L. K.	B.C.E., Ph.D., M.Eng.Sc., M.I.E. Aust., Professor of Civil Engineering, University of Melbourne.
*TRACY, W. F.	B.C.E., Section Engineer, JHC (West Side).
VAN VELDHUIZEN A.	C.E. (Dip)., Project Manager, WSC.
WALLACE, A.	B.Sc. (Eng.), Senior Engineer, Maunsell, London, seconded to WSC, April–November, 1969, Maunsell, Melbourne, November, 1969–March, 1970, and to JHC, March–June, 1970.
WARD, D.	M.I.C.E., (1971), Section Engineer, FF & P (West Side).
WILSON, C. V.	B.C.E., M.I.E. Aust., General Manager, The Authority.

*Died as a result of the collapse of span 10–11.

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THE REPORT

PART 1. INTRODUCTION.

1.1.1. THE DISASTER.

On the 15th October, 1970, at 11.50 a.m., the 367-ft. span of the West Gate Bridge, known as span 10–11, being one of the spans on the western side of the River Yarra, suddenly collapsed.

There can be no doubt that the particular action which precipitated the collapse of span 10–11 was the removal of a number of bolts from a transverse splice in the upper flange plating near to mid-span. These bolts were removed in an attempt to straighten out a buckle which had occurred in one of the eight panels which constitute the upper flange. The buckle, in turn, had been caused by the application of kentledge in an attempt to overcome difficulties caused by errors in camber.

To attribute the failure of the bridge to this single action of removing bolts would be entirely misleading. In our opinion, the sources of the failure lie much further back ; they arise from two main causes.

Primarily the designers of this major bridge, FF & P failed altogether to give a proper and careful regard to the process of structural design. They failed also to give a proper check to the safety of the erection proposals put forward by the original contractors, WSC. In consequence, the margins of safety for the bridge were inadequate during erection ; they would also have been inadequate in the service condition had the bridge been completed.

A secondary cause leading to the disaster was the unusual method proposed by WSC for the erection of spans 10–11 and 14–15. This erection method, if it was to be successful, required more than usual care on the part of the contractor and a consequential responsibility on the consultants to ensure that such care was indeed exercised. Neither contractor, WSC nor later JHC, appears to have appreciated this need for great care, while the consultants, FF & P, failed in their duty to prevent the contractor from using procedures liable to be dangerous.

In July, 1970, following the collapse of the Milford Haven Bridge in Wales, a substantial programme of strengthening was put in hand on the partly built West Gate Bridge. Despite this extra strengthening we are not satisfied beyond all reasonable doubt that the stresses at all points of the steel bridge, as now designed, will be within safe limits, unless further modifications are made. (See Section 5.1.1.) We recommend that before construction is recommenced a thorough check be made of the whole design by an independent authority.

At the time of the collapse, men were working on the span in various capacities, and others were in, or near, some lightly constructed huts which had been placed immediately beneath the span, and on to which the span fell. Of the men on the bridge, or beneath it, 35 were killed outright, or died as a result of their injuries. Many others were injured in various degrees of severity.

The disaster is probably the most tragic industrial accident in the history of Victoria. The names of those who lost their lives will be found in Appendix G.

Throughout this Commission we have been very conscious of the personal tragedies caused by the disaster. We extend our deepest sympathy to the wives and families of those who perished. It is our earnest hope that the findings of this Commission may go some way towards ensuring that such a calamity will never again occur.

1.1.2. RESCUE WORK.

Following the collapse, those near at hand immediately set about the task of giving emergency aid to the injured. Stage 2 of the " State Disaster Plan " was brought into force, and a massive rescue operation was mounted. Within minutes, dozens of police, firemen, doctors, nurses, first-aid staff and volunteers attended the injured.

The Melbourne Harbor Trust Emergency Service and the State Electricity Commission also did valuable work. The Salvation Army and clergymen from various denominations were there to give aid and comfort to the injured and dying. Rescuers worked all afternoon and far into the night always in horrifying conditions, and often in peril of death or injury themselves. A fire broke out as a result of spilled diesel oil igniting ; while quickly extinguished, the fire added to the difficulties of rescue work. A number of cranes were brought into use in attempts to free those trapped under the fallen span. Some bodies were not recovered for many days after the collapse. A fleet of ambulances took the injured to four of the metropolitan hospitals, where special arrangements had been made to treat them. All that was humanly possible to save life, and mitigate the suffering of the injured, was undoubtedly done.

1.1.3. The Coroner's Committee and the Police.

We were fortunate that the City Coroner, Mr. H. W. Pascoe, S.M., within only hours of the collapse, gathered around him a number of technical experts who were able to give advice during rescue work and to make observations on the wreck before it had been altered or changed in the interests of rescue.

The unofficially constituted body of men formed themselves into a committee and obtained very valuable information in the days immediately following the collapse, and before the Commission itself could begin its task.

The membership of the committee was as follows :—

Prof. L. K. Stevens (Co-ordinator)	Department of Civil Engineering, University of Melbourne.
Prof. N. W. Murray	Department of Civil Engineering, Monash University.
Prof. H. Worner	Department of Metallurgy, University of Melbourne.
Dr. F. A. Blakey	C.S.I.R.O., Division of Building Research.
Dr. G. Wilms (and later Dr. M. de Morton)	Defence Standards Laboratory.

The committee members made a report to the Coroner in which they set out the result of their inquiries, which were fully recorded both in documentary form, and by photograph, and also included records of interviews with eye-witnesses and other persons with knowledge of events which concerned us. This report was tendered in evidence. We are greatly indebted to this committee, especially for the speed with which they obtained and recorded information which would otherwise have been lost.

We are also indebted to the members of the Victoria Police, some of whom were on the scene of the tragedy within minutes. A special unit of the Homicide Squad, under Superintendent Holland, assisted by Sergeant Russell (now Inspector), Senior Detective Coates (now Sergeant) and Sergeant Morrison, co-operated with the Coroner's committee. They worked assiduously for many days obtaining statements from those involved in the accident, and from eye-witnesses, many of whom were traced with some difficulty, and most of whom were eventually called before us. An extensive set of highly important photographs of great technical excellence was also obtained by police photographers and made available to us.

1.2.1. History of the West Gate Bridge Project.

As Melbourne developed it became increasingly apparent that a crossing of the Lower Yarra somewhere between Port Melbourne and Williamstown was a necessity to enable traffic to move across the river, other than by the somewhat limited facilities provided by the Williamstown ferry, and for many years it has been obvious that such a crossing would be greatly to the benefit of Melbourne and suburbs, and to western Victoria generally.

In 1957, there was formed an association of industries engaged in their various occupations in Williamstown, Spotswood, Altona, Footscray and adjacent areas—called the Western Industries Association. This association was most concerned that a crossing of some description should be built across the Lower Yarra. In 1958 discussions were carried out between the Government, the association and interested municipalities. The Minister for Public Works intimated that at that time there was no money available to build this crossing, and suggested that it might be financed by private enterprise.

In 1961, a company called the Lower Yarra Crossing Company Limited was incorporated, which took up further negotiations with the Government, and in 1962, the Government acting through the Country Roads Board, carried out quite considerable sub-surface investigations.

In 1964, a committee was appointed to examine the question of whether a crossing should be made by way of a bridge, which would be sufficienty high to keep the river open to shipping, or whether it would be more appropriate to construct a tunnel. The committee failed to agree on this question, but the Government eventually decided that the crossing be by way of a high level bridge rather than by way of a tunnel. The reasons for that decision are irrelevant to this inquiry.

In 1965, the Lower Yarra Crossing Company went into voluntary liquidation and a company was formed called the Lower Yarra Crossing Authority Limited, which was later given a licence under the Companies Act by the Attorney-General to discard the word " Limited ", in its name, and consequently, the name became simply " The Lower Yarra Crossing Authority ".

Although that name may give the impression that the body is a Government instrumentality of some sort, it is in fact, a company limited by guarantee, and entirely comprised of representatives of private enterprise.

By the *Lower Yarra Crossing Authority Act* 1965 (No. 7365), the Authority was vested with certain powers, the general scheme of the Act being that it would be able to borrow money on debentures, to finance the construction of the crossing. It was given the necessary powers for the compulsory acquisition of land, and it was also given power to raise tolls on the bridge, so that, when completed, the bridge would be open to the public, a toll would be charged, and the funds received would gradually pay off the debenture debt which the company had raised. In the final result the bridge would be paid for by those using it, and at the stage when all loans had been discharged, the bridge would become the property of the Crown.

The Authority was also given regulation-making powers, and in the following year by Act No. 7443 of 1966, the Government of Victoria guarateed repayment of the debenture funds borrowed.

As early as 1964, the original company had intiated conversations with Maunsell and Partners of Melbourne, consulting and civil engineers, on an informal basis. They asked this firm to conduct a preliminary investigation. The preliminary report was dated 23rd June, 1964.

In 1966, borings were carried out by George Wilmot & Company Limited, and further information was obtained about the sub-soil and foundations for the proposed crossing. In February, 1966, Maunsell suggested that because of their own limited experience with major bridges of structural steel there should be called in as consultants, an English firm of consulting and civil engineers of world reputation—Messrs. Freeman, Fox and Partners.

The Authority accepted the suggestion, and requested a further report from both those firms. This somewhat more detailed report which was made in November, 1966 was in evidence.

In that report, the general concept of a scheme was broadly outlined. On 7th July of 1967, the matter had progressed to the stage where the Authority entered into an agreement with Maunsell and FF & P as joint consulting engineers. The agreement required the joint consultants to submit to the Authority, a report on the works together with preliminary plans, designs, and an estimate of cost, and upon receipt of a direction to do so, to prepare all such detailed designs, drawings, working drawings, dimensions, sections, plans, bills of quantities and the like, as might reasonably be required for the purpose of enabling tenders to be called.

The agreement of the 7th July, 1967, was amended by an agreement dated 10th October, 1967. The purpose of the amendment was simply to extend part of the approaches to the bridge from the original plans.

In February, 1967, " Preliminary Information to Tenderers " had been published by the Authority, inviting prospective tenderers to submit applications for qualification as registered tenderers, supported by information and particulars establishing their capacity and experience. The terms of the relevant qualification requirements were settled by the joint consultants.

Pursuant to publication of this notice, applications were received from a number of prospective tenderers, those accepted as registered tenderers being selected upon the recommendation of the joint consultants.

When the tender documents were issued, only the registered tenderers in respect of each contract were invited to tender.

The tender documents for—

Contract F—Bridge Foundations ;

Contract C—Concrete Bridge Works ;

Contract S—Steel Bridge Works ;

were ultimately issued to registered tenderers in October, 1967.

In January, 1968, Dr. W. C. Brown of FF & P came to Australia for further discussions with the Authority, and representatives of CRB, in relation to the design and specifications for the steel bridge and he stayed on until after tenders closed on 14th February, 1968, to assist in evaluating the tenders.

On the advice of the joint consulting engineers, it was decided that Contract S should be awarded to World Services and Construction Pty Ltd., a subsidiary in Australia of Werkspoor Utrecht N.V., a company of international reputation having its base in the Netherlands. Contracts C and F were awarded to John Holland (Constructions) Pty. Ltd., a Melbourne based company much experienced in concrete work.

A letter notifying the Authority's intention to award Contract S was sent to WSC on 27th February, 1968. The necessary approval of the Governor-in-Council under the *Lower Yarra Crossing Authority Act* 1965, was formally given on 2nd April, 1968, and notices to proceed were issued by the joint consultants to the contractors on 8th April, 1968, in relation to Contracts F and C, and on 9th April, 1968, in relation to Contract S.

All of the contracts were later formally signed in July, 1968.

From April, 1968, work under Contract F, proceeded satisfactorily and practical completion was reached on 25th September, 1969.

Contract C also proceeded satisfactorily, despite some early loss of time and prior to the occurrence on 15th October, 1970, was expected to reach practical completion in late March, 1971.

In relation to Contract S, the specifications for the quality of the steel to be used and the tests to be carried out, thereon, were worked out by WSC in conjunction with the Broken Hill Proprietary Company Limited, and approved by the joint consultants.

WSC's first steel purchase order was finally submitted to BHP on 16th August, 1968.

The steel specifications and testing requirements continued to be reviewed and revised until January, 1969, when they were finally settled.

When the work of construction commenced in April, 1968, it was hoped that the bridge would be finished by the end of December, 1970. However, by the end of 1969, it was perfectly clear that WSC was behind in its programme. This company had a great deal of trouble with labour, and there were many times when all work was stopped owing to strikes. Some of the strikes were caused by disputes between individual unions and the company, but it appears from the evidence, that at least as many were caused by what are known as " Demarcation Disputes " between various unions.

In February, 1970, the Authority gave notice to WSC, under the terms of Contract S, which required the company to show cause why certain clauses of the contract should not be enforced against it. In reply, WSC gave a counter notice calling upon the Authority to allow it to advance good and sufficient reasons why these penalty clauses or the other powers that the contract conferred, should not be enforced, and sought an opportunity to debate the allegations which the Authority was making against it.

A settlement was reached, in which it was agreed, that WSC should continue to fabricate the boxes and carry out the work of sub-assembly of them, but that completed boxes would be handed over to JHC, who would be responsible for all further operations involved in erecting the boxes and completing the construction of the steel portion of the bridge, including all concrete work and black top for the roadway.

The agreement between the Authority and JHC, whereby JHC undertook to carry out that portion of the work, which had now been taken from WSC, is known as Contract E, and is dated 10th July, 1970, although the date upon which JHC, in fact took over the work was the 17th-18th March, 1970, under a letter of authority dated 16th March, 1970, so that before the contract was actually signed, its general terms must have been agreed to and the parties were proceeding on that basis. This contract and the contract made between WSC and the Authority are discussed in Sections 8.1.2. to 8.1.5.

On 2nd June, 1970, one span of the Milford Haven Bridge in Wales collapsed during construction [1, 2]. This bridge had many features in common with the West Gate Bridge including the use of a trapezoidal box girder. The collapse, however, was dissimilar to that which occured in the collapse of span 10–11 in that it originated at a point of negative moment over one of the columns when a span was being cantilevered out, whereas in the West Gate Bridge, the failure was at a region of positive moment near to the centre of what was, at the time, a simply supported span. The failure at Milford Haven was subsequently found to have been due to a compression failure of the vertical diaphragm over the column, thereby inducing a compression failure of the lower part of the inclined side webs and the bottom flange of the box section. The weakened section was unable to sustain the moments imposed and the cantilever rotated about the virtual hinge so formed until its further end hit the ground.

In view, however, of the fact that the Milford Haven Bridge was a bridge of a somewhat similar design, and in view, no doubt, of the fact that FF & P were the designers of both bridges, the Authority arranged for G. Maunsell and company in London, to make an immediate check of the design of the steel spans in the West Gate Bridge. That report was completed subject to certain addenda by January, 1971, and was received into evidence in this Commission.

Following upon the Milford Haven collapse, certain steps were taken to strengthen the steel spans of the West Gate Bridge.

At the time that JHC took over Contract E, WSC had in fact, assembled the two half spans on the east side of the river between peirs 14 and 15, and had successfully lifted them into position on top of the pier, but the joining of them, and the bolting together had not been completed, so that JHC had to complete the joining of those two half spans. On the west side there had been a certain amount of work done on the northern half span between piers 10 and 11, and at the time of the change-over, this partly assembled span was still on the ground.

Jacking-up of this north half span between piers 10 and 11 commenced on the 15th May, 1970. After delays caused by strikes and bad weather, the rolling beam level was reached by 9th June, 1970 and on the 19th June, the rolling of the north half span across the rolling beam commenced. This operation was completed on the 22nd June.

The jacking of the southern half span was commenced on the 17th August. By the 28th August, it had reached the rolling beam level. On the 29th, it was rolled across, and that process was completed on the 1st September.

The situation immediately prior to the 15th October, 1970, was that on the east side, two half spans 14–15 had been joined, and cantilevering for the next span reached box 12, where it rested temporarily on a trestle specially erected for that purpose, about mid-way between piers 11 and 12. On the west side, the two half spans 10–11 had both been elevated and were resting on the rolling beams. They had been brought to their positions, but the longitudinal jointing between the two halves was less than a third completed.

1.2.2. Description of the Bridge Structure.

Originally, the whole Lower Yarra Crossing project was conceived as part of an over-all plan for development of the Melbourne-road system. It consists of a highway running from Port Melbourne in the east, to the Geelong-road in the west.

The Authority's franchise covered that portion of the project which ran from Graham-street, Port Melbourne in the east, to Williamstown-road in the west, a distance of $2\frac{3}{4}$ miles. The remaining portions of the project were the responsibility of the CRB.

Our inquiry, and hence this Report, is concerned only with the main steel bridge across the River Yarra, referred to hereafter simply as "the bridge".

The bridge and the approach viaducts which are shown in Fig. 1 are together approximately 8,500 feet long with the roadway rising to a height of 192 feet over the river and carrying two 55 ft. 2 in. wide carriageways.

Pre-stressed concrete approach viaducts on either bank supported on slender concrete columns and comprised of 220-ft. spans, lead up to the bridge itself which is 2,782 feet long. The five spans of the steel bridge between piers 10 and 15 form a cable stayed continuous box girder with spans of $367\frac{1}{2}$–$472\frac{1}{2}$–1,$102\frac{1}{2}$–$472\frac{1}{2}$–$367\frac{1}{2}$ feet.

These three central spans are partly supported by steel cable stays which are in turn held by two steel towers rising on top of piers 12 and 13 to 150 feet above the roadway. The towers and cable stays are all in one vertical plane on the longitudinal centre line of the bridge.

The steel spans are supported on rocker bearings at piers 11, 12, 13 and 14, and on expansion roller bearings at piers 10 and 15. Piers 10, 12, 13 and 15 are vertical cantilevers, being fixed to their pile caps at their lower ends. Piers 11 and 14 in the final state are pinned both top and bottom.

The cross-section of the trapezoidal box girder is shown in Fig. 5. The section is 13 ft. 2 in. deep at the centre line, 83 ft. 6 in. wide at the upper flange and 62 ft. 6 in. wide at the bottom flange. Cantilever brackets at 10 ft. 6 in. intervals bolt on to the top of the sloping outer webs to extend the total deck width to 122 ft. 6 in.

The girder is sub-assembled in units called boxes. Each box is 52 ft. 6 in. long by about 42 feet wide. A typical box is shown on Pl. 4.

Two such boxes make up a full cross-section. All boxes on the upstream side of the bridge are referred to as north boxes as opposed to the south boxes on the downstream side. Boxes are identified as to their location in the span by a system of numbering starting with box 1 west over pier 10 and box 1 east over pier 15. Boxes are numbered from 1 to 27 sequentially starting from both ends to meet at the centre of the main span where box 27 east is finally joined to box 27 west to complete the girder.

The geographical reference N. S. E. and W. used in this Report was that used by the consultants and contractors. The directions are nominal only, the nominal north being, in fact, 37° east of true north.

The concrete approach viaducts are both curved in plan and the associated transition curves extend on to the steel bridge at both ends. The longitudinal axis of the steel box girder is, however, maintained straight. The curvature in plan of the deck is achieved by adjusting the lengths of the cantilevers towards the ends of the bridge.

1.2.3. THE PARTIES.

Apart from Maunsell & Partners, of Melbourne, Freeman, Fox & Partners, World Services and Construction Pty. Ltd., and John Holland (Constructions) Pty. Ltd., all of whom were concerned with the design, construction and erection of the bridge, the following were suppliers of materials used in the structure : The steel originally ordered was supplied by The Broken Hill Proprietary Company Limited. The company which supplied the materials for the concrete work was Pioneer Concrete (Vic.) Pty. Ltd. The great quantity of special type bolts required were supplied by McPherson's Limited of Melbourne. All these organizations were represented at the inquiry. While Maunsell, FF & P, WSC, and JHC remained before the Commission throughout the hearing, the other three parties were granted leave to withdraw at various stages—because in each case there was no evidence criticizing the materials, and indeed positive evidence that the materials were satisfactory.

In this Report we of course have occasion to mention the names of a great many individuals, some of whom appear in the list of witnesses, Appendix A, and others do not.

The more important of these individuals, with some note of their positions in their respective organizations and other relevant detail, will be found at the front of this Report.

1.2.4. THE CONTRACTS ENTERED INTO BY THE AUTHORITY FOR THE STEEL BRIDGE.

This Report is not concerned with the contracts for any part of the structure other than the five main steel spans, except in one aspect, namely that Contract F (Foundation) and Contract C (Concrete Brigework) were both awarded to JHC. It should also be noted that one of the original tenderers for Contract S, (Steel Bridgework) was a consortium consisting of JHC and two well-known companies of high reputation, Redpath Dorman Long & Co. Ltd. and Johns & Waygood Limited. The relevance is that when it became necessary to select a contractor to take over that part of Contract S, which had been excised from the work to be performed by WSC, one possible candidate for substitute contractor was naturally JHC.

The contracts entered into by the Authority which related to the steel spans were—

1. The basic contract with Maunsell and Freeman Fox dated 7th July, 1967, as amended by agreement dated 10th October, 1967.
2. Contract S with WSC, formally signed on 17th July, 1968.
3. The contract entered into with WSC on 13th March, 1970, whereby the work that that company was required to do under Contract S was limited and the erection proceedings excised.
4. Contract E with JHC formally signed on 10th July, 1970, to take over that portion of the WSC contract excised by the agreement with WSC.

Of these four contracts the first requires no comment. However, the second contract mentioned does warrant some brief consideration. Basically, the draftsman has taken and considerably amended and adapted the " Australian Standard General Conditions of Contract for Civil Engineering Works ". He has also incorporated a number of other documents, some of which are appropriate, and some not. The vice of such a wholesale incorporation is that some of the documents included purport to impose liability upon persons not parties to the contract. Our attention was drawn to an example of this, namely the inclusion of a document marked " K ".

Briefly, the history of this document is, that at a meeting between Roberts, Brown, Hardenberg and Van Veldhuizen, in London on 12th March, 1968, it had been agreed as follows :—

> " FF & P will supply to Werkspoor in the course of this or next week, three sets of full scale contract drawings and the necessary data for establishing the camber in the end spans. To ensure that erection calculations will be in accordance with the engineers' design criteria—notably the end condition under full dead load—FF & P will supply to Werkspoor a set of the bridge design calculations ".

The minutes of this meeting including the above minute were sent to FF & P shortly after 12th March, and were received without comment. When it came to drafting Contract S, a copy of these minutes was among the documents included, and was marked " K ".

Before inclusion in the contract these documents were sent to FF & P, and Roberts in fact suggested that one of them, document " E ", should be omitted, but no objection was taken to the inclusion of document " K ".

In these circumstances, we find it very difficult to accept Roberts' evidence that this document was included without his knowledge. Be that as it may, WSC sought this material without success, and it never was supplied.

As this matter was of major importance to WSC, its inclusion, merely by way of an incorporated document, can only be regarded as slipshod drafting.

We make no comment on other similar instances, save to observe that contracts for works of the magnitude of this project, merit rather more careful consideration.

One term of Contract S requires special mention, that is Clause 47, headed " Proceedings on Default or Bankruptcy of Contractor ". Sub-clause 47.1 gave the Authority power to—

(*a*) suspend payment under the contract or ;

(*b*) take the work remaining to be completed wholly or partly out of the hands of the contractor or of any person in whose hands or possession the work or any part of it may be or ;

(*c*) determine the contract.

These sanctions came into force on the occurrence of various defaults by the contractor including failure to proceed with the work " with due diligence and expedition ".

As to the third and fourth of these contracts, except as to one or two clauses, the precise terms need not be closely examined. However, the circumstances surrounding and leading to their execution must be considered, and are dealt with in Part 8.

The only clause of Contract E which requires specific reference is Clause 19 which is set out in Appendix C and is considered in some detail in Part 8.1.5.

PART 2. THE COLLAPSE.

2.1.1. EVENTS LEADING UP TO THE COLLAPSE.

It is possible to take any of a number of points as that from which stemmed the later events leading ultimately to the failure. One such starting point is certainly the manner in which spans 10–11 and 14–15 were erected.

There are a number of ways in which these spans might have been erected. The simplest is probably to erect all the boxes or half boxes and connect them together up in the air resting on temporary false work. This is a straightforward method, but a lot of temporary supports would be required, many of them founded in the water.

Another method is cantilever erection in which the half boxes would be lifted up into position by means of a travelling crane running out on the spans, starting at piers 10 and 15 respectively, and have their outer ends supported on a succession of temporary trestles. The last temporary trestle would be at mid-span, and beyond this the remaining four boxes would be cantilevered until box 8 landed on pier 11 (or pier 14).

In this scheme it would be necessary to support the ends of the boxes on temporary packings on top of the trestles, so as to tilt them slightly upwards. Thus, after allowing for the downward deflection of the cantilever under dead load, its outer end would still be at a sufficiently high level to land on its bearings on pier 11 (or pier 14).

The advantages of both these schemes are that the lifts are relatively light and although the connections between the boxes have to be made up in the air, they present no special difficulties because each box is small and there will be little, if any, distortion along the edges of the projecting flange plates at the longitudinal splices.

Both these schemes have the further advantage that they permit the early placement of concrete in 10–11 (14–15) since the temporary props or false work can be left in position while concrete is being placed. This not only gives a solid working platform from which future stages can be cantilevered out, it also provides a more effective composite action between steel and concrete for the bridge in its service condition.

Another method of erection would be to assemble the whole span 14–15 or 10–11 on the ground and jack it up into place on the piers. This scheme has the advantage that all the boxes can be connected together at ground level instead of up in the air. But the total weight to be lifted would be some 1,200 tons to a height of 170 feet, and the jacking operation would be slow and costly. Nevertheless, many bigger spans have been erected in this way and the operation should be reasonably straightforward with no unforeseen difficulties.

The method proposed by WSC for erection of spans 10–11 and 14–15 was to assemble each span in two halves on the ground—full length and half width. Each half span was then lifted by jacking straps to the top of the pier, landed on a rolling beam and moved across into position. The cross-diaphragms in the centre panels of each box were assembled on the ground in the south half spans.

In joining the two half spans together, it was therefore necessary not only to correct any difference in their vertical camber but also to pull them together horizontally. It should then be possible to connect all the cross-diaphragms at the centre panels and also the full lengths of the longitudinal splices in the upper and lower flanges.

Compared with cantilever erection, the adopted scheme had the advantage that much more of the assembly of the boxes could be done on the ground and no temporary props were required.

Compared with lifting the whole of each span in one, the adopted scheme had the advantage that each half span naturally had only half the weight, so that the lifting operation would be halved in magnitude, although it had to be done twice.

There were a number of serious disadvantages however, in lifting each span in two longitudinal halves. Each half span was asymmetric so that there was a horizontal bowing outwards, amounting to about $1\frac{1}{4}$ inches from the centre line at mid-span. Moreover, the upper and lower flanges each projected some 10 feet towards each other at the longitudinal centre line, and substantial temporary stiffening and bracing were needed, particularly on the upper flange which was stressed in compression, to prevent these projecting flanges from buckling. It was also essential to ensure that the two half spans were accurately assembled on the ground before lifting and that their vertical cambers were the same within at least 1 inch or preferably $\frac{1}{2}$ inch in order to avoid having to carry out a slow, difficult job of correcting the cambers up in the air.

This is not to say that the method could not have been successfully adopted, provided that very careful forethought had been given to dealing adequately with all the potential difficulties. But these difficulties are substantial and that may be the reason why no evidence could be found that this method of erection had ever been attempted before, anywhere in the world, under conditions similar to those prevailing at the West Gate Bridge.

2.1.2. Erection of Span 14–15, East Side.

The first girder of eight half boxes to be lifted was that for the northern side of span 14–15. The boxes which had been pre-assembled elsewhere were brought together on temporary staging at ground level and there bolted together, at what was thought to be the correct camber.

Once the bolting together was completed the span was lifted off the temporary staging so that it spanned freely between the jacking beams at either end. At this stage significant compression instability was observed in the projecting flange plates of the inner upper panels.

This instability resulted in severe buckles to the plate edge and also to the outermost bulb flat stiffener. The buckles in the plate edge were stated to be as much as 15 inches deep ; they can be seen on Plate 6. The amplitude of the plate buckle reduced away from the free edge until it disappeared at the connection with the inner longitudinal web plate. The extent of buckling of the outermost bulb flat longitudinal stiffener is shown in Fig. 10, from a survey made at the time. The sharp kink shown over the transverse splice was associated with a crippling failure of the fish plate which splices the bulb flat stiffeners (referred to in this Report as a K-plate). It is evident that both the flange plate itself and the bulb flats were bent beyond the elastic limit although this fact does not appear to have been appreciated at the time. Surveys made at the request of the Commission have confirmed that post elastic distortion is still present in the affected panels, some longitudinal stiffeners being as much as $\frac{5}{8}$ inch out of straight in a length of 10 ft. 6 in. between transverse ribs.

To understand the causes of this instability, it is necessary to refer to the details of the structural arrangement in Section 3.2.1., wherein it is pointed out that the transverse beams under the inner upper panel do not fall opposite the transverse beams in the outer upper panels so that there is no possibility of continuity from a beam of the outer panel to one of the inner panel. The transverse beams of the inner panel are bolted to the $\frac{3}{8}$-in. inner web plate at one end but during erection are entirely free at the other end. They are, in effect, cantilevers with a very limited moment capacity at the root and can give little resistance to their outer ends being distorted up or down when the main flange plate becomes unstable.

It is true that in the permanent condition, when the two half boxes have been joined, the opposing pairs of transverse beams in the centre cell are spliced and can then span as simple beams between the two inner webs. Precautions to ensure stability are, however, essential during the temporary stage, when the half girder is self supporting.

WSC had indeed provided some sort of support to the upper transverse beams, but this, we believe, was intended only for the purpose of stiffening a box during handling and transporting. Whatever its purpose, the bracing provided proved quite inadequate to prevent the instability which did in fact take place. Details of the temporary braces provided by WSC for the 14–15 north span can be seen on Plates 2, 4 and 6. In general, an angle iron tie was bolted between the tips of the upper and lower transverse beams and a wire rope brace, with turnbuckle, was fixed diagonally from the tip of the bottom transverse beam to the root of the upper beam, i.e., to the point where it joined on to the inner web. This system gave some partial fixity to the upper beam and the presence of the wire rope brace would prevent gross downward movement of the tips of the transverse beams. There was little to prevent upward movement and in any case this bracing was provided only at every other transverse beam.

It is easy to see how, in these circumstances, instability could occur. The remedy by a more effective bracing system is equally apparent. What is so hard to understand is why this obvious difficulty was not anticipated by WSC in drawing up their erection proposals and secondly why the defect was not detected by FF & P when it was their duty to check the safety of the proposed erection method.

On the remaining three lifts of half spans 14–15 south and 10–11 both north and south, the outstanding tips of all upper transverse beams were held by substantial angle iron braces intended to hold the beam from both upward and downward movement. In addition, the exposed edge of the flange plate, which extended 21 inches beyond the last bulb flat, was stiffened with a temporary channel, 6 in. x 3 in. bolted close to the plate edge. WSC had not apparently fully appreciated the importance of all these stiffening measures because on 14–15 south this edge channel was bolted on in convenient lengths, leaving gaps of several feet unspliced between the end of one channel and the start of the next. It is not surprising that instability of the plate edge occurred also in this 14–15 south span, but owing to the existence of the diagonal braces, the buckling was much less extensive and the consequences much less serious than in the corresponding north span.

Once this major buckling of the inner upper flange plate on span 14–15 north had been observed, it was necessary to decide what should be done about it. In our opinion there was only one proper course of action and that was to lower the girder back on to its supporting trestles so that the buckles could be removed and an adequate system of bracing and stiffening fitted. WSC were, however, anxious to go on lifting the span in its damaged condition They hoped that by doing so they would not hold up the commencement of assembly of the next half girder and at the same time hoped that repairs could be effected without too much difficulty up in the air.

Roberts, who happened to be in Melbourne at the time, saw this span after it was lifted off the trestles and took part in the discussions with WSC. He was no doubt influenced in his attitude by certain calculations made by Crossley, the deputy resident engineer, as to the safety of the damaged half span. Unfortunately these calculations by Crossley were based on the false premise that the half spans would bend about the horizontal axis instead of about the inclined axes as is appropriate to the asymmetric cross-section. Crossley's calculation made certain conservative estimates of effective plate widths, but it also included at least one significant error of arithmetic. Roberts attempted to justify Crossley's calculation on the grounds that he came up with an answer which was not too far wrong. That this may have been so was an accidental and fortuitous combination of errors and in our opinion is no way to undertake an engineering calculation. The safety of going on with the lifting was in fact judged on a totally inadequate analysis of the situation, neither FF & P nor WSC having any real idea of the stresses imposed.

In the outcome WSC requested permission to go ahead lifting the span and this was not opposed by FF & P either by their resident engineer or by Roberts, either of whom could have vetoed the operation if they had considered it to be unsafe.

The north half span, 14–15, was raised to the top of the jacking towers and rolled across to take up its proper position over the columns. The operation was repeated some weeks later with the south half span.

As described above, the northern half girder had a badly buckled plate edge while the southern half girder also had minor buckling on the plate edge. In addition, it was found that the northern half span had a mid-span camber about $3\frac{1}{2}$ inches greater than that of its southern counterpart. It was not possible to measure the camber difference at the projecting plate edge owing to the large buckles ; the $3\frac{1}{2}$-in. difference refers to the cambers at the two inner webs.

Before the internal diaphragms, which join the north and south halves, could be bolted up, it was necessary to get rid of the difference in vertical camber and also to pull the two halves together horizontally. The method adopted for overcoming the difference in camber for the 14–15 span was to jack up one end of the south half girder by an amount equal to about twice the camber difference, i.e., by about 7 inches. This was done by means of four 200-ton hydraulic jacks on pier 14 beneath box 8 south. This brought the two halves into vertical alignment at around mid-span so that after pulling the two halves together so as to close any horizontal gap it was possible to line up one or two diaphragms sufficiently close for the connections to be made at those places, after which box 8 south was lowered again. Unfortunately this relative movement between half spans whilst they were pressed into tight contact resulted in some structural damage to the outstanding edges. The two half girders now secured together at mid-span had a greatly reduced camber gap at other places. This residual gap was handled locally by hydraulic jacks enabling the connections to be made to the remaining diaphragms.

The buckled plate edges were pulled into line by fitting a 6-in. universal beam on the underside of the longitudinal splice between north and south boxes. Normally only an upper cover plate was specified. By means of specially long bolts the buckles were pressed out between the upper cover flat and the stiff beam section underneath. Once the plates had been brought into line the temporary long black bolts were removed, one at a time, and replaced with the final friction grip bolts. The distorted transverse beams on the north half girder were easily forced back into line and bolted to the ends of their opposite numbers on the south side.

This procedure for taking out the buckles on the plate edge of the north boxes worked fairly well for most of the upper longitudinal beam, but there was one section around box 5 which could not be straightened out in this way. The longitudinal splice of the top plate had been worked on from both ends almost to the middle, but for this short section remaining unbolted there was no flexibility left. The only way that could be seen whereby these bolt holes would be brought fair and the last bolts entered was for the transverse seam of the north inner panel to be undone, so that small movements might take place between plates. It was hoped that this movement would be enough also to take out the buckle.

At this time the upper plating under self weight bending was nominally stressed to about 7·8 tons per sq. in. It was decided that it was unwise to carry out the operation of undoing transverse seams until the high positive bending at box 5 could be reduced by adding on some of the cantilever boxes 9–12. In fact, the operation was performed when only boxes 9, both north and south, had been added. This reduced the theoretical compression stress at box 5 by only about 10 per cent. from about 7·8 tons per sq. in. down to 7·0 tons per sq. in.

It is curious to note that the actual approval for the workmen to commence undoing the bolts was given to Atkinson, a JHC foreman, by Simpson, the FF & P engineer at a time when Burbury, the contractor's engineer, was not even on the job. It is true that the general procedure for undertaking the operation had been agreed earlier between Burbury and Simpson, but when Burbury arrived on site at about 9 a.m. he was considerably annoyed to find the work had been started in his absence some two hours earlier.

Bolts were removed from one side of the transverse joint between boxes 4 and 5 and also at the same time from the joint between boxes 5 and 6 ; they were not jammed in the holes and came out easily. Unbolting started at the longitudinal centre line and worked across the seam to within about 2 feet of the line of the inner web, i.e., about 30 bolts on each line. This virtual isolation of the inner upper panel of box 5 had the desired effect of flattening out the remaining bulge. The movement of the plates at the splice was only slight and it was found possible to re-enter new bolts with only minor reaming of holes. The K-plates splicing the longitudinal bulb flat stiffeners were not unbolted during the whole of this operation and were stated to have shown no sign of damage, despite the fact that they must have been called upon to carry some of the load shed by the plate when the transverse seam was opened up.

Although it apparently achieved its object, this operation was a regrettable necessity. The act of releasing plates while under stress means inevitably that some other parts of the section must carry the load. The relaxed plates can never return to the same strain condition that they would have had if they had not been disturbed. The consequential redistribution of internal stress which was not envisaged when making the design calculations, would almost certainly cause some reduction in the real safety margin on the span.

The apparently successful elimination of the buckles which occurred on the east side was to have another unfortunate consequence in that it engendered a certain degree of unconcern when a somewhat similar buckle occurred on the west side. Having successfully dealt with the buckle on the east side, the engineers were reasonably confident of their ability to handle the buckle on the west side ; a confidence that was fatally misplaced.

2.1.3. Assembly of Half Boxes on the Ground.

Much of the difficulty experienced in erecting span 14–15 arose directly from the errors in camber of the two half spans. As will be seen later, the experience was to be repeated, with even greater errors of camber, in the west span 10–11.

It can be argued, with some justice, that if proper procedures had been adopted when bolting together the boxes on the ground, then there need never have arisen the large camber difference experienced between north and south girders. Some camber difference is unavoidable, but in our opinion the difference of approximately 4 inches which occurred at the West Gate Bridge is inexcusable. With proper care a difference of $\frac{1}{2}$ inch, or certainly less than 1 inch, should have been achieved, which would have enabled the holes to be brought fair by means of successive careful drifting throughout the length, thus obviating the use of jacks at all.

It is of interest therefore, to examine the methods actually used in order to see how the errors in camber came to exist.

A sub-assembly area was located on the east bank where the individual panels were bolted together in special jigs in order to form the half boxes. These half boxes were then transported to where they were needed close to their final location in the span.

So far as spans 10–11 and 14–15 were concerned the half boxes, numbers 1 to 8, were lifted on to a series of carefully aligned trestles so that the attitude of each box relative to its neighbours could be determined prior to bolting together, and in this way the proper camber for the span could in theory be achieved.

While with this system the weight of each half box is supported by its own particular trestle, WSC, in planning this operation were concerned that this might not always be so. They anticipated that sunshine on the top plating could cause partly completed spans to hog upwards at the centre and thereby press down more heavily at the first and last supports. WSC's concern was that the light transverse diaphragms in boxes 2 to 7 might thereby be overstressed, or even that the temporary footings for intermediate trestles might be overloaded.

Rather than providing temporary strengthening to guard against these possibilities, WSC chose to employ a system which would ensure that the total load of the partly constructed span would be more or less shared equally between all trestles, no matter whether there was thermal hogging or not.

This equal sharing of load was achieved by floating the half boxes on a series of hydraulic jacks inserted between the trestle head and the underside of the box. The jacks were interconnected on a common pressure line so that the total load was shared equally between all jacks.

As Hardenberg described the system, the pressure in the jacks was regulated to be somewhat less than that required to sustain the full weight of the boxes. In this way all jacks were fully retracted and " grounded " when there was no temperature difference to induce curvature. Again according to Hardenberg, all important survey work to establish the camber profile was carried out only in this " jacks grounded " condition, generally at day-break or under other near uniform temperature conditions in the steelwork.

The system adopted by WSC to adjust the camber profile was to jack the base of the trestles so as to alter the level of the head of the box-supporting jack, when in its fully retracted position.

The floating support system may have been good in theory but there is no doubt it caused difficulty in practice. Only a few degrees temperature difference between top and bottom flanges were enough to set the floating action going, and it was probably operating for most of the time when actual connection work between boxes was being made.

Once the partly-erected girder was afloat on the jacks, any fixed reference plane was lost. It was then difficult to ensure the correct angular relationship between a new box being added and the girder as so far assembled.

When JHC took over the assembly of the half spans on the west side, they do not appear to have understood the logic behind the floating system. Rugless stated in evidence that it was a daily duty to make sure all jacks were floating, apparently under the impression that they were to be kept in this condition at all times, irrespective of the temperature differences.

The surveys for camber on the west side were not made at sunrise and were made presumably when the boxes were floating on the jacks. Under this condition some longitudinal bending is induced in the girder, due to the fact that the boxes are not all of uniform weight while the jack loads upwards are all nominally equal, although even there friction effects might account for up to 5 per cent. variations. Longitudinal bending induced in this way might account for a camber error up to 1 inch, depending somewhat on the jack friction. Such an effect was ignored by JHC engineers. It is noted elsewhere (Section 3.2.1.) that the tapering out of the super-elevation necessitates a twist in the assembly of the boxes on the convex side of the curve. The floating system of supports made control of this twist very difficult, as some of the surveys make abundantly clear. One such survey adds a comment blaming the floating jacks for the errors, which it was noted had caused departures from the desired deck levels of as much as 3 inches due to errors in twist alone.

Under the conditions imposed by the floating support system a greater reliance than usual had to be placed on the perfection of the fabrication. If every hole was drilled in the right place, and all relevant holes were properly lined up in erection, then the required camber would be automatically established.

The difficulty was, of course, that there were inevitable small errors of fabrication and the bolt clearance allowance, especially with the waisted shank bolts, meant that bolts were not necessarily concentric in their holes. If all these tolerances worked in the worst possible way for both the upper and lower transverse splices, it would be theoretically possible to have an unwanted rotation of one box relative to the next in line of about $\frac{1}{4}°$. Again, if all these errors of rotation were cumulative in their worst sense which is a statistical improbability, then an error in camber for the span 10–11 of about $\pm$ 16 inches could occur.

While the statistical chances of an error of 16 inches is extremely small, it is not at all improbable that errors of $\pm$ 3 inches could occur, as indeed they did.

Once all the half boxes 1 to 8 had been bolted together, the girder so formed was lifted off the trestles to be supported at boxes 1 to 8 by the jacking straps, prior to being lifted to the tops of the piers. At this stage all uncertainties about trestle reaction disappeared and meaningful surveys cound be undertaken, although even then so far as the west span 10–11 is concerned, the necessity to undertake the surveys at a time when the temperature difference was minimal does not seem to have been appreciated.

On the east side the surveys of the freely supported half spans relate to levels at the line of the inner web, since the buckled plate edges on the facing inner upper panels made any measurement at the plate edge of little meaning (see Plate 6). The survey showed that at the inner web the north half span was some $3\frac{1}{2}$ inches higher at mid-span that its southern counterpart.

On the west side, where the plate edges were adequately restrained against buckling, the north half span was again the higher. At mid-span the difference in level between the two facing plate edges was about $4\frac{1}{2}$ inches.

The assessment of the significance of these differences in camber was not simple because there were valid reasons why some differences should be present because of differences between the loads on north and south half spans at the time the surveys were made.

The principal differences were that the inner transverse diaphragms were all assembled with the south half spans (see Plate 2), and secondly that the stiff-legged derrick to be used for subsequent stages of erection was assembled on box 7 of the south span and lifted up with that span.

The deflection at mid-span caused by the weight of the inner diaphragms is about 0·3 inch, while that caused at mid-span by the derrick is about 1·2 inches.

If, then, we accept the valid camber differences as totalling about $1\frac{1}{2}$ inches, it emerges that the real errors in camber amounted to about 2 inches on the east side and 3 inches on the west side.

In our opinion the importance of getting the two cambers the same to within close limits was so vital that the assembly procedure should have been designed so that the north and south half spans were as near identical as could be arranged when the surveys were made. This could have been achieved by attaching alternate inner diaphragms to the north and then to the south girder, and secondly by positioning the crane over the end support at box 8 or leaving it off altogether until after the surveys had been completed. If any gross error of camber, say in excess of 1 inch, had been detected, the right thing to do would have been to lower the south span back on to the trestles and make such adjustment to one or more of the bolted transverse joints as was necessary to correct the camber difference.

In the event it was not until both spans had been lifted on to the top of the piers and brought into close proximity that any precise determination of the camber difference was made, and that by direct measurement.

As pointed out above, the presence of the crane caused about 1·2 inches of the camber difference. The logical step would therefore have been to move the derrick from box 7 to box 8. This was not done.

The reason for not raising the derrick on box 8 in the first place was stated to be the difficulty of assembling it there while the south half girder was still on the ground. This arose because the mobile crane employed was said not to be able to lift the derrick directly onto box 8 due to intereference with the temporary lifting tower.

The difficulties created on the east side (span 14–15) by errors in camber of the two half girders should have been sufficient warning to all concerned that the very greatest care should be taken to see that nothing like that happened on the west side ; indeed, McIntosh wrote to JHC in April, 1970 pointing this out, but was not nearly precise enough in specifying the accuracy that must be achieved. His warning therefore was ineffective and the errors on span 10–11, as stated above, were even greater than those which had occurred on the east side.

We think that Hindshaw made an error of judgment in not insisting on a more effective camber control and in approving the raising of the south half span 10–11 when there were already clear indications that its camber would not match up with that of the north half span.

2.1.4. Provision for Vertical and Horizontal Adjustment of Half Girders on Span 10–11.

The method of erection whereby spans 10–11 and 14–15 were erected in two halves inevitably resulted in a horizontal bowing apart of the two halves and, as we have seen, an accidental vertical camber difference was introduced as well. The method adopted on the east side for the elimination of the horizontal and vertical gaps between the half spans had proved troublesome and resulted in some structural damage, so that when drawing up procedures for the west side, an attempt was made to evolve a better method.

The method adopted for dealing with the vertical gap was to use a 10-ton jack at each of the intermediate diaphragms (boxes 2 to 7), fitted so as to bear down on the north half span and push up to the south half span. The procedure stipulated that the jacks should be fitted with gauges so that the load could be measured, but no gauges were ever fitted.

So far as the horizontal correction was concerned, the procedure laid down that the longitudinal splice joining north and south halves must first be bolted for a short distance over piers 10 and 11. This was to prevent the uncontrolled movement together of the half spans when horizontal pulls were subsequently applied.

For pulling of the two halves together, horizontally, short lengths of Macalloy bar were used. The nuts on the bars reacted against special brackets bolted on to the north and south halves. It was unfortunate that such short bars were used, because the tilt on the bar, caused by any vertical misalignment, made it impossible to apply horizontal correcting forces until after the vertical correction was nearly complete. The force on each Macalloy bar was supposed to be limited to 15 tons. There were more than enough bars provided to close the gap even if all were working to only half that limit. No means were provided, however, for ascertaining the load in each bar and as a result some bars were overloaded and failed.

The Procedure Manual stated that these pulling devices were appropriate only when the vertical gap to be closed was less than $2\frac{3}{4}$ inches but we consider this $2\frac{3}{4}$-in. limitation entirely artificial. It had to do with interference of the pulling brackets themselves and could have been avoided by a re-design of the bracket. The jacking system itself could have dealt with a vertical gap of about 4 inches.

The procedure set out for operating the system was the same, in principle, for both the horizontal and vertical correction. The steps, as specified, for the vertical adjustment are set out as follows :—

(*a*) Jack at diaphragms 4 and 5 until the gap is closed or the jack loads each reach 10 tons.

(*b*) If (*a*) is not enough to close the gap perform the same operation with jacks at diaphragms 3 and 6, leaving jacks at 4 and 5 untouched for the time.

(*c*) If (*a*) and (*b*) are not enough repeat the operation with jacks at diaphragms 2 and 7, leaving jacks at 3, 4, 5 and 6 untouched.

(*d*) If still more movement is needed go back to jacks at 4 and 5, which will by now have on them less than 10 tons, and bring them back to 10 tons each.

(*e*) Continue the cycle, until either the gap is closed or all six jacks have the full 10 tons on each of them simultaneously.

In theory the operation of the two adjustment systems should have presented no great difficulty. In practice it was found very difficult to get the two half girders to the correct relative positions throughout their lengths. The difficulty without doubt was due to the fact that the camber curves on the two halves were different not only in amplitude but in the shape of the curve, so that even when connections had been made at a number of diaphragms it was still necessary to use large forces to make the remaining parts fit. This all stems from the failure to equalize the cambers when assembling the half spans on the ground.

2.1.5. The Kentledge.

When the two half girders on the west side, span 10–11, were brought into close proximity up in the air it was established beyond all doubt that a camber difference of about 4½ inches existed.

It was proposed by JHC that time might be saved if the vertical difference of level could be taken out by using kentledge to push down the north half span relative to its south counterpart. It so happened that ten cube-shaped concrete blocks, each weighing about 8 tons, were on site from a previous operation and it was thought that these would give about the right order of load to remove the camber difference, if positioned as a more of less concentrated load near mid-span.

Hindshaw's reaction to this proposal, according to some witnesses was unenthusiastic but, it was said, because he could not raise any rational objections he gave his "reluctant" approval for kentledge to be used.

There is not enough evidence to determine with any certainty whether Hindshaw gave a blanket approval to the use of kentledge as such, or whether he agreed to adopt a detailed scheme involving the use of a limited amount of kentledge.

On balance we incline to the former view, first because the subsequent action of JHC in lifting up ten blocks makes it appear unlikely that they ever worked out a rational scheme supported by calculations which they could have put to Hindshaw. Secondly, by the reaction of Hindshaw himself when he discovered that the use of kentledge had caused a buckle to develop. Hindshaw records the buckle in his diary for Wednesday, 9th September, 1970, and adds—

"Obvious overstress due to concrete kentledge."

Neither in his diary nor elsewhere does Hindshaw comment that this unfortunate consequence was the result of JHC using more kentledge than the amount he had approved, although it may be considered that if Hindshaw had had a genuine complaint against JHC for using too much kentledge he would have made it, as at that time he was seeking for any cause of complaint against them.

Moving the stiff-legged derrick from box 7 to box 8 would have eliminated about 1·2 inches of the camber difference and there is overwhelming evidence that most, if not all, of the parties were aware of the advantages to be gained by doing this. WSC's notes sent to FF & P in London and to both the resident engineer and JHC on site called for moving the derrick over the pier ; Ward too realized the desirability of doing it ; but it was not done.

Instead the crane was used, still on box 7, to hoist ten blocks of kentledge on to the span where they were temporarily parked around box 8 before being moved out on a pair of universal beams and placed over the longitudinal inner web of the whole length of box 4 and the west half of box 5.

The loading of kentledge was commenced in the first week of September, 1970, and by the end of the morning shift on Saturday, 5th September, seven blocks had been finally positioned and three more remained parked around box 8. No shift was worked on Saturday afternoon or during the day on Sunday. The night shift on Sunday, 6th September, noticed that a major buckle had developed in the inner upper panel at the 4–5 splice of the north span. The nature of this buckle can be seen in Pl. 7, wherein are also to be seen some of the kentledge blocks.

After the appearance of the buckle any attempts to reduce the remaining camber difference by more kentledge were abandoned and the three unwanted blocks at box 8 were lowered to the ground.

After the buckle had taken place there was still about 1 inch of camber difference to eliminate, before the inner diaphragms projecting from the south half could be bolted into place on the north half span. It appears that even at that stage the obvious step of moving the crane was not immediately followed and this remaining camber gap was taken out by jacks pushing down on the north half and reacting upwards on the south half. Later the crane was moved.

After the camber difference had been eliminated, the connections of the inner diaphragms were made to the north half span. At box 4, however, the transverse beam, to which the diaphragm should have been bolted, was so much displaced by the buckle that it was not found possible to bolt up at that place. All other diaphragms were, however, fully bolted and even at box 4 the diaphragm was bolted along its lower and vertical edges. With the two girders effectively interconnected, the kentledge was deemed no longer necessary and it was lowered to the ground.

When JHC put forward the proposal to use kentledge they did so without making any supporting calculation. WSC were still on site to give JHC technical advice, but as Hardenberg was away on leave, they discussed the matter with Van Veldhuizen who was less familiar with the stress situation than Hardenberg.

Hindshaw, when making the decision to permit the use of kentledge, was in no position to know whether there would be overstressing or not, because he did not have available to him the necessary calculation on the bending of the half boxes about their asymmetric axes. The only relevant calculation in Melbourne was that done by WSC. There is no evidence that FF & P either in London or in Melbourne, ever made such a calculation until after the collapse, although Brown claims to have done so at the time of opening tenders. If he did, his calculation was not preserved.

The symmetrical bending properties of the box were available to Hindshaw, but the use of these would have resulted in a serious under-estimation of the stresses in the inner upper panel.

Hindshaw did not think it of sufficient importance to advise London that the question of kentledge had been raised, neither did he later tell them about the buckle, despite the fact he was at the time making frequent telephone calls to Kerensky on other matters. Nor did Kerensky in his subsequent evidence regard the use of kentledge as a matter that should have been raised with him.

It was a fairly simple matter for Hindshaw to have seen that, for the elimination of the same amount of camber error, the use of kentledge, a more or less concentrated load at mid-span on one girder only, would cause about 2·5 times the bending stress in the half girder as that caused by the jacking system, which was a more or less uniformly distributed load acting up on one half girder and down on the other.

Admittedly this was only a temporary condition, and after the kentledge was removed the stresses caused by the differences in camber would have been shared by the two half spans.

In spite of the urgency of the matter, we consider that Hindshaw made a serious error of judgment in not referring the question of the use of kentledge to London for advice on its effect on the stresses. In our opinion the matter of kentledge from start to finish was badly handled and reflects little credit on either the resident engineer and his staff or on the contractor's site staff on the west side.

2.1.6. The Buckle at Joint 4–5 Span 10–11.

There can be no doubt that the act of adding the kentledge was the precipitating cause of the buckle which, as described in the preceding section, appeared on the inner upper panel around joint 4–5 north. The buckle was a clear indication that partial failure of the structure had occurred. The margin of safety against complete collapse must then have been small.

The factors on the design side which contributed to the partial failure were the generally low margins of safety used and the poor detail of the splice plates joining the bulb flat longitudinal stiffeners, (the K-plates). The behaviour of the temporary diagonal brace which was fitted to restrain the inner transverse beams may also have been a factor.

The buckle which did in fact develop was greatest at the inner edge of the inner upper plate in way of the transverse splice, where the departure from the true plane was about $3\frac{1}{2}$ inches. The distortion at the same plate edge was down to $2\frac{1}{2}$ inches at the nearby transverse diaphragm, which was not at that time bolted to the upper flange plate. In the longitudinal direction, the buckle faded away to zero at the transverse beams, 10 ft. 6 in. away on either side of the diaphragm, and in the transverse direction it reduced to zero at the inner web. At that time, there was no sign of the buckle propagating into the outer upper panels.

The cover plates at the transverse splice showed considerable tilt and there can be no doubt that plastic yielding of the $\frac{3}{8}$-in. thich upper flange plate occurred immediately to both sides of the cover plates. Witnesses speak of rust flaking off in the vicinity and one witness even speaks of zig-zag markings which would be evidence of the Lueder lines.

When buckling took place, the K-plates joining the bulb flat stiffeners were also crippled, although it appears that the K-plate nearest to the inner web had not in fact ever been fitted. JHC removed the four crippled plates without the prior authority of the resident engineer, though by that time they were probably of little use. JHC's idea was that the absence of the splice plates would make it easier to straighten the flange plates. In fact, the efforts to flatten the buckled plates at that stage proved to be unsuccessful. Following the unfastening of the crippled K-plates, the buckle appears to have become marginally worse.

Hindshaw, when he saw the buckle, did not anticipate that there would be too much difficulty in eliminating it. His diary entry for the 9th September, 1970, records—

> " Examined plate edge buckling on west side, i.e., unrestrained edge of boxes 1–8 north. Obvious overstress due to concrete kentledge. Rectification problem will be less than on east side. Discussed procedure carefully with David ".

(David refers to D. Ward).

Hindshaw was clearly drawing on his experience with the successful removal of the buckle on the 14–15 span on the east side, and was confident that this new buckle would be relatively easy to eliminate.

The logical steps which should have been taken were to have completed as much of the longitudinal splice at both upper and lower flanges as was necessary to permit the lowering of span 10–11 down from the rolling beams on to the final bearings ; next to have cantilevered out boxes 9 and possibly 10, and finally to have completed as much of the longitudinal splice as was possible, particularly on the upper flange. At that stage, the compression stress at the buckled 4–5 joint would have been so reduced that the proposed undoing of the bolts in the transverse splice might have been safely undertaken.

Although Hindshaw had just received from London a comprehensive summary of the theoretical stresses throughout the structure at all stages of erection, this did not consider the possibility of undoing a transverse splice ; so that neither he nor Crossley had in their possession any calculations which would have given them a true insight into the danger of unbolting the splice at that time. A most rudimentary assessment by anyone familiar with stress analysis would, however, have recognised that undoing the bolts before the longitudinal splices between the half spans were complete would create a dangerous " notch effect " with its associated stress concentration at the head of the notch. As a first approximation, it is likely that the concentration factor would be of the order 2, and this, working on the existing stresses, would theoretically cause yield to be exceeded. The consequences of local yielding on over-all stability would have been harder to assess, but the possibility of its producing calamitous failure was not to be dismissed without serious consideration.

If Hindshaw was really aware of the true situation, and if, as we believe, he did discuss the problem with Ward, than he should have instantly vetoed any suggestion of taking out the bolts.

Hindshaw was, however, faced with a difficult problem. At this time, he was in the midst of the conflict with JHC on the issue of strict adherence to the Procedure Manual.

The relevant procedure required that the span could not be lowered on to the permanent bearings until all the intermediate diaphragm connections had been made and cantilevering could not be commenced until the span was on its permanent bearings. Thus so long as the centre diaphragm remained unbolted along its northern top edge, no further step could be taken towards cantilevering, yet until the buckle was removed, the necessary bolting of the whole upper flange could not be achieved. Hindshaw's problem was to break this circle without contravening the Procedure Manual.

We quite agree with Ward's view that the unbolted upper edge of box 4 diaphragm was relatively unimportant in relation to lowering the span to its permanent bearings. This view might have appealed to Hindshaw had he not felt embarrassment at suggesting any departure from the prescribed procedures while insisting that JHC must strictly adhere to them.

It follows that the first step must have been to bolt up the diaphragm in box 4, which in turn required that the buckle be first straightened out.

At about this time, the relations between FF & P and the Authority had become strained over the matter of extra stiffening which had been found necessary following the failure of the Milford Haven Bridge and Hindshaw was particularly anxious that Wilson should not see the buckle and thus have further cause for complaint.

It is well established that for some weeks prior to the 15th October, Hindshaw, Ward, Crossley and Tracy were in agreement that no attempt should be made to remove the buckle until at least box 9 had been cantilevered out.

In evidence, Ward maintained that at a meeting taking place within two days of the 15th October, between Hindshaw, Crossley and Ward, Hindshaw changed his attitude and after discussion, Crossley allowed himself to be persuaded by Hindshaw that an attempt should be made to remove the buckle as soon as possible.

It was decided that Ward should implement the operation. Ward's evidence was that Hindshaw's instructions were given to him in some detail ; that he was told to undo six or eight bolts at a time and to make a thorough examination both inside and outside boxes 4 and 5 to see that there had been no undesirable reaction before proceeding to remove the next group of six or eight bolts.

Ward is the sole survivor of the three persons involved in this meeting and everything depends on whether we accept his evidence because, if it is accepted, it is conclusive on the issue of whether Hindshaw knew and approved of the action he took on the 15th October. It was urged upon us very strongly that Ward's evidence should be rejected. Firstly because he had incurred severe head injuries in the disaster resulting in some degree of retrograde amnesia, and it was obvious that his memory was deficient as to many details. Secondly Mr. Beach presented a careful analysis of the evidence of other witnesses which, if accepted as accurate in regard to the time to which they had deposed, would make it well nigh impossible for the meeting described by Ward to have taken place when he said it did.

A number of other factors were also pressed upon us as raising an inference against Ward's credibility.

However, upon careful consideration of these matters, we felt that they were at least outweighed by other factors from which the contrary inference could be drawn.

One factor which impressed us was the change in Ward's attitude within a day or two of the 15th October. There can be no doubt that until the 13th October, Ward and Tracy were both resigned to leaving the buckle in place until a more advanced stage of erection had been reached. Something happened about the 13th October which changed Ward's attitude and caused him to direct Tracy to co-operate in removing the bolts.

On Wednesday, 14th October, Ward told Tracy that the decision had been made to straighten the buckle on span 10–11 without further delay. Tracy was apparently surprised and concerned about the advisability of doing it at that time ; he asked Ward to give his formal written instructions for the work to be done. This was unusual as it does not appear to have been customary for Tracy to ask for formal written confirmation of an instruction from Ward. Ward gave to Tracy the written authority he was seeking ; the instruction he issued is reproduced in Appendix D. It will be noted that Ward refers first to the necessity to complete the bolting of the No. 4 diaphragm ; unbolting the 4–5 splice is to be done with the object of making possible the completion of the diaphragm connection. Ward goes on to make it clear that he would personally supervise the operation.

After careful consideration of all the evidence, we are satisfied—

1. that Hindshaw had every intention of eliminating the buckle by removing the bolts at some stage ;
2. that he was concerned that Wilson should not see the buckle ;
3. that at some time prior to the 15th October, he had given Ward detailed instructions as to the method of removing the bolts ;
4. that when Ward removed the bolts, he honestly believed that he was acting under Hindshaw's instructions.

The real question at issue is what were Hindshaw's precise instructions to Ward. Did Hindshaw instruct Ward to remove the bolts at some time in the future, meaning thereby after erection of the span had proceeded to a safe stage, or did he, as Ward says, specifically ask him to remove the bolts as soon as possible ?

The possibility that Ward honestly misunderstood Hindshaw's instructions cannot be eliminated.

We reject the theory that Ward and Tracy ventured on a frolic of their own when deciding to remove the bolts, but on the evidence we are unable to make any positive finding that Hindshaw either authorized or knew of the action taken by Ward and Tracy on the 15th October.

2.1.7. Unbolting of 4–5 Splice on 10–11 North.

When writing the authorization to Tracy (see Appendix D), Ward had made it clear that he would be personally responsible for supervising the work of unbolting the splice. In fact, Ward, in the outcome, appears to have taken direct command of the operation and was instructing the JHC foremen in what they had to do.

Tracy was present most of the time and appears to have co-operated well with Ward. Enness, a FF & P inspector, whose responsibilities had only just before been extended to cover the west side as well as the east side, was also present for much of the time and made suggestions on how the work should be done.

Ward stated that it had been agreed with Hindshaw that when the work was done, he was to loosen the bolts in groups of six or eight. After each such group had been loosened, he was to make a thorough inspection both inside and outside the boxes to ensure that nothing undesirable was happening.

Work started at about 8.30 a.m. on 15th October. After about sixteen bolts had been loosened, there was significant slipping of the two plates relative to one another such that the loosened bolts were jammed tightly in their holes and could not be removed. At this stage, Enness suggested the bolts be tightened with the air gun until they broke. The shock reaction of the bolts failing in tension dislodged the broken pieces and thus cleared the holes.

Eventually about 30 bolts were removed from the box 5 side of the splice, extending from the longitudinal centre line to within about 2 feet of the inner web. Also about seven bolts had been removed from box 4 side of the splice, all close to the longitudinal centre line. The bulge had flattened from about $3\frac{1}{2}$ inches initially to about $1\frac{1}{8}$ inches, but adjacent to the longitudinal centre line the sliding movement was said to have been so great that some holes were completely blind. Examination of the relevant overdrilled holes on the wreck shows that to be an exaggeration ; the sliding movement appears to have been limited to $\frac{1}{2}$ inch. (See plate 12).

At this stage, a dramatic change took place and the signs of distress for which Ward had been on guard suddenly appeared. First the vicious buckle which up to that stage had been limited to the inner upper panel spread into the adjacent two outer upper panels. This was accompanied by the buckling failure of the upper part of the inner web plate. These new buckles were known to Ward and Tracy as soon as they happened. Enness saw them shortly after, but when he tried to tell Ward, he found he already knew.

About this time, Ward and other witnesses say that they felt a gentle settlement of the north half span of the bridge. Prof. N. W. Murray has expressed the opinion, with which we agree, that this settlement coincided with the spreading of the buckle to the outer upper panels. From that time onwards, the north half span had inadequate strength to sustain its own weight and only survived because it was able to bear down on to the south half through the interconnected transverse diaphragms.

The margin of safety in the south half span was not such that the entire dead load of the north half span could be borne in addition to its own self weight. There must therefore have been some residual strength still remaining in the north half, but in view of the extent of the buckles in the top flange and inner web, this resistence must have been only a small part of its value in the undamaged state.

It is indeed surprising that total collapse did not follow until some 50 minutes after the propogation of the buckle into the outer flange plates. Plastic creep in the steel and possibly the effect of thermal changes all contributed to the slow diffusion of yielding, first in the north half span and subsequently across the full width of the south half span, to bring about total collapse.

Although the full implications of what had happened on the bridge were not realized, nevertheless a sense of urgency developed. More men were fetched from elsewhere to speed up the work ; the niceties of procedure were dropped in the urgency of getting the bolts back into place.

Ward, at about 11.00 a.m. tried to contact Hindshaw, telling him that things were not going according to plan and that he (Hindshaw) should come over to the west side as soon as possible. In fact, Crossley took Ward's telephone call and went out to find Hindshaw. The rebolting was going well and the buckle had come out sufficiently to render it possible to make the bolted connection between the transverse diaphragm in box 4 and the inner upper flange plate. To do this, however, the diagonal brace had to be removed, because its plastically yielded end palm plates would have otherwise prevented the pulling down of the top plating.

At about 11.30 a.m. Hindshaw arrived on the West span accompanied by Crossley. Hindshaw rapidly assessed the situation which superficially did not appear to be deteriorating. He was nevertheless gravely concerned with what was clearly a potentially dangerous situation and decided to ask Hardenberg's advice. Hindshaw telephoned Hardenberg and gave him a brief sketch of the situation asking him to come over. The last thing Hardenberg heard on the 'phone was as if Hindshaw was thinking aloud, "Shall I get the bods off ?"

Hindshaw at least had become aware of the frightening possibilities but, according to Hardenberg, there was no sense of panic or even of urgency in his voice.

Almost immediately after that telephone conversation at 11.50 a.m. span 10–11 collapsed. Among those who died were Hindshaw, Crossley and Tracy.

2.2.1. Consequences of the Collapse of Milford Haven Bridge.

In Section 1.2.1. we have described the collapse during construction of one span of the Milford Haven Bridge, on the 2nd June, 1970.

In the Milford Haven Bridge, the box girder was much less in width but much greater in depth than at West Gate. It was also only a single cell section having no internal webs. The collapse happened when the box girder was being extended by cantilevering out. It had reached almost 200 feet and another box was being run out when a buckling failure took place over the last pier. The moment of resistance at the root of the cantilever was so reduced that the outstanding 200 feet rotated about the crippled section until the far end hit the ground.

Sir Hubert Shirley-Smith was asked to investigate the cause of collapse and report to the Coroner [1, 2]. This Report attributes the primary cause of the failure to the collapse of the stiffened steel diaphragm over the pier.

The buckling of this vertical diaphragm together with the failure of the lower part of the inclined side webs and also of the bottom flange, brought about the total collapse of the span. There were a number of other associated factors which to a greater or lesser extent contributed to the failure of the Milford Haven Bridge. These matters are all set out in the report to the Coroner, which was in evidence before us.

The parallels between the Milford Haven Bridge and the West Gate Bridge, and the fact that FF & P had been the designers in both cases increased the concern of the Authority as to the safety aspects of the steel spans. Hardenberg had earlier expressed concern over the structural sufficiency of the design, and had provided for some strengthening claimed to be necessary not only for the erection conditions, but also for the bridge in its permanent state. Wilson was so disturbed that in March, 1970, he requested FF & P to carry out a check on their design calculations. Kerensky agreed that FF & P should do this, but by June when the Milford Haven collapse took place, very little, if any, progress had been made on this design check.

After Milford Haven, Wilson demanded that the check on the stresses throughout the whole steel bridge should be given high priority and called for a check by an independent authority. It was finally agreed that an " independent " check should be made by G. Maunsell and Partners, the London firm of consulting engineers. Because of the close links between Maunsell in London and Maunsell in Melbourne, we could not regard the former as independent in any real sense. Nevertheless, we are satisfied that the report finally submitted by Maunsell, London, does give an objective statement of their thorough analysis of the stresses in the bridge.

Unfortunately, the final report from Maunsell, London was not received until many months after the West Gate Bridge had fallen. However, an interim report in September, 1970, drew attention to the over-stressing which was likely to occur in some of the K-plates and warned about other regions where some over-stressing was probable.

In the meantime, as a result of the Milford Haven collapse, FF & P, as a matter of some urgency, conducted their own design check on West Gate and found that high stresses were likely at a number of places. This led to their recommendation for a massive programme of strengthening, requiring an additional 160 tons of steel, at a cost of $120,000.

At the same time as this stress check was going on, FF & P were also trying to evolve a method by which the concrete deck could be applied to the steel bridge at an earlier time than proposed by WSC.

In the original bar chart for the time schedule of the construction work, which was part of the tender documents, it had always been envisaged that the placing of deck concrete would take place progressively while further steel boxes were still being erected. WSC, in trying to work out an erection scheme, had been unable to devise any scheme that did not delay the start of concrete work until the steelwork was at a very advanced stage of erection. All other schemes investigated by them involved over-stressing of the structure, which, as we now know, was almost inevitable, since the structure was over-stressed even for its service condition.

In the course of their re-appraisal of the design, FF & P put through a series of computer runs in July, August and September, 1970. These same runs were also used in investigating the proposals for accelerating the concreting work.

Because the two investigations were conducted with a common computer analysis, FF & P were able to put forward the view that the need for the extra stiffening was shared between the two causes, that is between the stiffening needed to give an overall extra safety, referred to by FF & P as a " belt and braces " approach and the stiffening needed for the accelerated concreting of the deck. In early evidence to this Commission, FF & P represented that the two causes were inseparable, but that if anything the speedier concrete sequence was the main reason for adding the extra steel.

Freeman, the first witness to appear before us, made it clear that he was unfamiliar with the details of the West Gate Bridge, because he had played no direct part in the project. Nevertheless, he was prepared to state as his belief that " more than 50 per cent. and probably more like 80 per cent." of the added stiffening was due to the concreting sequence and that only the smaller part was for the " belt and braces " approach.

Roberts went so far as to claim a separation of the two analyses in point of time and stated that the analysis, made for the speeded up concrete, was done first and showed that it was necessary to strengthen the bottom flange in boxes 8 and 9.

Brown at first followed the same line, but under pressure of questioning admitted that the strengthening of the flanges and webs of the boxes had nothing to do with speeded up concrete proposals. He assented to an estimate of only 15 per cent. of the extra stiffening being due to concrete schedules.

Kerensky, the last FF & P partner called, would have none of this confusion. He stated flatly and unequivocally that " only a small part of the stiffening was for concreting ; the major part was for belt and braces for cantilevering ". He added that there had been no attempt to mislead anyone.

We cannot escape the conclusion that FF & P, whether intentionally or not, were misleading. The importance of the design deficiency was minimized and the influence which any change in concreting schedules had on the stiffening programme was overstated.

FF & P have advanced the argument that the " belt and braces " was among other things done for psychological reasons. They argued that adding the extra stiffening would give the men working on the bridge greater confidence in the safety of the structure. That it had the opposite effect is hardly surprising.

Kerensky has recorded notes of a telephone conversation with Hindshaw on 19th September, 1970, in which he records Hindshaw as saying—

> " . . . labour is out of control and takes every opportunity to strike. Any suggestions of lack of safety of the structure and the continuous revisions to permanent sections give additional grounds for the men's behaviour ".

The use of the expression " belt and braces " by FF & P is intended to convey that the extra stiffening was probably not necessary, but put on just to make doubly sure. Both Brown and Roberts state that this was indeed their understanding of the situation. After a careful examination of all the evidence on this point, we find that we cannot agree with this view. We find that most of the stiffening added was essential if safety factors were to be maintained. Even now, we are not satisfied beyond reasonable doubt that the stiffening added was sufficient, or that other parts of the structure may not also need to be stiffened if safety margins are to be preserved. This conclusion applies to both the bridge in its erection stages and in its service condition.

FF & P, themselves, were sufficiently disturbed over the possibility of a structural failure that a stop order was issued to prevent the erection of any more boxes until the strengthening measures on the boxes already erected had been completed. This cessation of progress on the erection was made despite the very great emphasis then being placed on speeding up the job. The ban lasted from 22nd July to 24th August, 1970.

In carrying out the stiffening work it was decided not to wait until steel of the previously approved quality should be available, but that the strengthening should be done with steel obtained locally from stockists in Melbourne.

Only mild steel was available from the stockists in the sizes and quantities required, and it was not always possible to get satisfactory manufacturers' certificates. Hence mild steel angles of a heavier section were used in some places, to provide additional stiffeners to high yield steel plates. Where additional longitudinal angle stiffeners were fitted they were made continuous through the transverse beams by burning out appropriate slots. This burning was done on site and involved cutting the web of the transverse beam right down to where it met the flange plate. A risk of burning damage on the flange itself was thereby introduced, unless the operation was very carefully controlled.

Wilson, who was very sensitive on the question of steel quality and welding, was not unnaturally angered by what was going on.

On 3rd September, 1970, a stormy meeting was held between Hindshaw and Wilson, Simpson also being present. Wilson made a savage attack on the efficiency of FF & P as he saw it, and expressed doubts about the safety of the bridge. At the end of this meeting, Hindshaw showed Wilson over the work on the east side, so that he (Wilson) could see for himself what had been done to stiffen the structure.

There can be little doubt that Wilson had for some time been increasingly unhappy about the service the Authority was getting from the FF & P end of the joint consultants. The whole affair of the post-Milford Haven strengthening brought his dissatisfaction into the open. While Wilson's quarrel was basically with the policy decisions of FF & P, made in London, the only man in Melbourne to whom he could voice his anger was Hindshaw. Wilson does not appear to have borne any personal animosity towards Hindshaw.

At about the same time Hindshaw was further embarrassed when it was found that an item had been overlooked in the strengthening programme and that more delays would follow while this was fixed up. The offending detail this time was in the bottom flange at the 8–9 joint where it was found the extra long K-plates would have been stressed so highly that they could have buckled when box 12 was cantilevered out. Stabilization of these K-plates was achieved by placing concrete in a 3-ft. wide strip right across the entire bottom flange so as to bury the K-plates in a concrete matrix. The opportunity was taken to give the same treatment to the K-plates at the 7–8 and 9–10 splices, although in those cases only normal length K-plates were in use.

2.2.2. BULGE ON BOX 9 SOUTH, EAST SIDE.

At the same time as difficulties were being experienced over the problems of adding extra strengthening, another incident occurred which was to have consequences out of proportion to its real importance.

Boxes 9 north and south had been cantilevered out early in July, and by September the cantilever had been extended further. On, or about, 8th September, a bulge was found in the sloping outer web around the splice 8–9 south. The bulge took the form of a dishing inwards of the web to a maximum of about $1\frac{1}{4}$ inches. It covered a roughly circular area with a diameter of about 10 feet, but faded away almost imperceptibly at the edges of this area. The bulge was very difficult to detect from inside, partly because the intensity of lighting inside the box was not high and also because it would be difficult for an observer to put himself in a position where he could look along the distorted plate. From outside, by leaning over the edge of the deck, the bulge was very obvious. It was not so obvious from the ground.

It was subsequently established that the bulge was the result of a fabrication distortion or an accident in erection which had gone unnoticed for two months. At the time, however, it was by no means certain that the outer web plating had not become unstable due to the stresses in it, and that the situation would not get worse as further boxes were cantilevered out.

This possibility alarmed JHC and led to a confrontation between them and FF & P.

C. V. Holland personally entered into the discussions on this issue. The reports which he had received from his own internal engineering service had only tended to increase the general lack of confidence felt by JHC in the safety of the structure.

A top-level meeting was held on 16th September, 1970. Those present included C. V. Holland, Birkett and Hindshaw. Birkett took the chair and asked that no minutes be taken as the meeting was informal only.

C. V. Holland expressed his concern about the safety of the structure, particularly in view of the need for the post-Milford Haven stiffening and the bulge on box 9 south. He demanded written assurances from the joint consulting engineers that the erection stresses were within safe limits, and announced that his company was not prepared to proceed with the erection of any more boxes in the air until he had received such assurances.

Later in the same meeting the discussion turned to the actual values of safety factors. JHC's engineer, Barmby, pressed for quantitative values to be stated, but Hindshaw could only reply that they were " adequate ". Repeatedly pressed on this point, Hindshaw gave non-committal answers and finally lapsed into silence.

Despite his reluctance to give quantitative answers, Hindshaw, nevertheless did ultimately give Holland the written assurance he was demanding. The actual letter was dated the day of the meeting (16.9.70), and Birkett says that Hindshaw had the letter already written and in front of him at the meeting.

In fact, Hindshaw was not in a position to hand over the letter at that stage because it had been drafted before he had received information from London that concreting was necessary to stabilize the extra long K-plates at the 8–9 joint on the bottom flange. After making sure that the weak K-plates at the 8–9 joint and at two other joints had been rendered safe, Hindshaw released the letter. It was received by Hollands on 29th September, 1970.

In the letter Hindshaw, in addition to giving a general statement on the safety of the erection procedures, goes on to deal specifically with the bulge on box 9 south. He mistakenly refers to it as being at the 9–10 joint and not the 8–9 joint. He gives an assurance that the bulge will not create difficulties so far as stress is concerned and that it can be safely removed at a later stage of erection when the shear stresses at the relevant section are low.

Meanwhile the CRB were asking for quantitative information on safety factors in the newly stiffened structure. The telex from Maunsell to London seeking the values talks of " approximate " values without " need to be explicit on whether load factors are against temporary or permanent condition ". It is not surprising that the table, when it arrived, was misleading. Hindshaw sent it to Wilson on 6th October, 1970, for transmission to CRB. It quoted safety factors as high as 5.90 which is manifestly absurd in the light of all that had gone on and if it were true could only indicate uneconomical design, and certainly unnecessarily heavy extra stiffening added by FF & P.

The true reason why such erroneously high values of safety were quoted is because the figures were based on critical elastic buckling stresses as calculated for mathematically flat panels without any regard for the inevitable imperfections which occur in actual structures ; also ignored was the limitation imposed by the yielding of the whole section. In consequence the values quoted are dangerously misleading.

This table of " so called " load factors is yet another example of the cavalier manner with which FF & P treated the Authority whenever it sought to be informed on the technical matters concerning the design.

2.2.3. The Diagonal Brace*.

The buckling of the plate edge which occurred when half span 14–15 north was lifted led to the adoption of a system of temporary bracing designed to prevent a recurrence of the same effect when half spans 14–15 south and 10–11, both north and south, were lifted. As part of this system of stiffening, diagonal braces were fitted to restrain vertical movement of the tips of the outstanding ends of transverse ribs (see Fig. 12).

These braces were made from 5 in. x 5 in. x $\frac{3}{8}$ in. mild steel angle about 13 feet long running diagonally from the bottom transverse beam at a point near to the inner diaphragm up to the upper transverse beam at its outermost tip. Five such braces were fitted to each half box.

For the normal situation the brace was bolted directly through one leg of the 5 in. angle on to the vertical face of the bent plate which formed the transverse rib. A single friction grip bolt was used at each end. With this arrangement both the brace and its end connections should have been able to resist about 3 per cent. of the axial load in the adjacent parts of the inner upper flange panel. Experience has shown that this should have been sufficient to ensure stability.

A special situation arose for the transverse ribs which were in the planes of the diaphragms. For this case the normal connection detail was unacceptable because the diaphragm itself had to be bolted to the vertical face of the bent plate rib and the presence of a diagonal bolted to that face would have made it impossible to bolt up the diaphragm.

For this special case therefore, palm plates were welded to both ends of the angle brace and the end connections were made by single friction grip bolts holding the palm plates to the horizontal flanges of the bent plate ribs (see Fig. 12). The palm plate itself was $\frac{3}{8}$ in. thick but the bent plate rib to which it was attached was only $\frac{5}{16}$ in. thick, both of mild steel.

Two independent errors were made when fabricating the special braces for span 10–11, the details of which are given in Appendix E. The issues raised by these errors were, first, did they cause the end palm plates on the brace to project out beyond the vertical face of the transverse rib and so make it impossible for the diaphragm to be fitted without fouling, and secondly was the structural strength and rigidity of the brace reduced by the errors.

After careful examination of the brace recovered from the wreck and of the transverse rib to which it was attached we are confident that it was impossible to fit the brace in such a way that the end plate could project beyond the face of the rib. Ironically it would appear that had the brace been made correctly there would have been a significant projection. We are equally satisfied that in its incorrectly fabricated form the diagonal brace would fail at a lower tension load and would have a smaller axial rigidity than if it had been made correctly.

The buckle which occurred when kentledge was added caused the inner end of the transverse rib at box 4 to move vertically by about $2\frac{1}{2}$ inches. This could have happened either because the brace was not in place at the time or because sufficient tension force was induced in the brace, and its end connections, to cause gross distortion.

For reasons which are set out in Appendix E, we have concluded that the brace was in fact in place at all relevant times.

So far as the second alternative is concerned there appear to us to be three ways in which tension force might have been induced in the brace—

(i) That the buckling of the stiffened inner upper panel took place over a double spacing of transverse beams, i.e. over a 21-ft. length, with the brace acting as a non-linear " spring " causing partial restraint against buckling at mid-span. In this case the " spring stiffness " of the brace, reduced as it was by errors in fabrication, would be important.

(ii) That the initiating action was the failure of the 4–5 splice, weakened by the K-plate detail. With the moment capacity at the crippled splice reduced to a very low value, instability of the two half span " cantilever " portions of the 10 ft. 6 in. long stiffened panel would follow, both parts bending upwards and thereby inducing a tension force into the diagonal brace. For this form of failure, the characteristics of the diagonal brace would have little influence on the failure load.

(iii) That interference took place between the top of the diaphragm and the underside of the transverse beam. It is postulated that if the diaphragm was under the transverse beam, instead of being just to one side of it, then when kentledge pushed the rest of the north half span down, the end of the transverse rib would be held back, thereby creating the buckle, not so much by compression instability as by mechanical brute force. The bending moments induced by the mechanical action would then be a contributory factor in the subsequent collapse of the

* Many of the problems associated with the diagonal brace are of detailed technical nature and it has not been thought proper to include them in the body of the Report. They are given in detail in Appendix E.

weak K-plates at the nearby splice. For this type of failure, errors in fabricating the end plates would be of little consequence because no matter which way the brace was made it would have been quite incapable of stopping the kentledge from pushing down the north half span and thus creating the bulge.

After considering all the evidence on this matter we have concluded that in all probability fouling of the diaphragm on the transverse beam did occur and that the bulge was, to that extent, mechanically induced. At the same time we believe that the stress in the 10 ft. 6 in. panel containing the splice was in any case so close to critical that little if any margin remained against crippling of the K-plates and the consequent buckling of the panel.

We consider as most improbable the suggestion, under (i) above, that failure could have taken place over a 21-ft. panel partially restrained at the centre, by the brace.

FF & P's advocacy has forced us to give a great deal of attention to the part, if any, played by the diagonal brace in bringing about the buckle at the 4–5 splice and hence the collapse of span 10–11. It was implied, for example, that the drilling of the holes in the end plates of the brace just 2 inches out of place could have been the factor which determined the fate of span 10–11. We think this is placing an altogether too high importance on this temporary member and cannot accept that, had the proper safety margins been preserved throughout the structure, these errors would have had significance.

On the basis of our conclusion that mechanical interference of the diaphragm did take place it follows that the bulge must have developed step by step as each block of kentledge was added. The buckling of the K-plates and the consequent upward kink of the upper flange at the 4–5 splice must have happened quite suddenly but we have no evidence as to the stage at which it took place.

After the buckle had formed attempts were made to force the flange plate back into place. This was done by attaching two pulling devices of 3-ton capacity to the tip of the upper transverse rib and pulling against the corresponding bottom rib. The damaged diagonal brace was not removed during this operation and not surprisingly the attempts met with little success.

On the morning of 15th October, 1970, the attempt to pull down the flange plate was repeated after the transverse splice in the upper flange panel been unbolted. On this occasion the diagonal brace was removed before the pulling forces were applied and the flange plate was easily pulled into place. The connection of the transverse rib to the top of the inner diaphragm was then made without difficulty, the work being completed within an hour of the final collapse.

The diagonal brace for the diaphragm position in box 4 north was recovered loose from the wreck, Plate 13 shows the distortions which had taken place in one of the end plates.

2.3.1 Other Failure Modes Examined.

We are convinced, beyond any doubt, that the failure of span 10–11 on 15th October, 1970, initiated with a compression buckling instability of the upper flange at mid span. When we began our inquiry there was, even then, a strong indication that the failure would be traced to the buckle on the upper plating ; there were, however, a number of other possibilities which had to be considered either as prime causes or as contributing factors to the accident. The other possibilities examined include—

(*a*) An explosion in, or adjacent to, boxes 4 and 5 on north side.

(*b*) A tension failure of the bottom flange or one of its splices near to the 4–5 joint.

(*c*) A structural failure of pier 11.

(*d*) Movement of the steel span on its temporary bearings on top of pier 11.

Evidence of witnesses and a careful examination of the wrecked structure make it most unlikely that any explosion occurred prior to the collapse. Various loud noises were heard as the bridge collapsed, but this would be normal for any structural failure in which large amounts of energy are suddenly released. During one stage of the collapse the boxes 5 to 8 took on a very steep angle and a great deal of equipment was tumbled down the deck to finish up in the area of the severe buckle at the 4–5 splice. Among the items tumbled into this chaos were compressors and welding machines, powered by internal combustion engines, a drum of white spirit used for clearing the faying surfaces of plates to be bolted together, numerous oxygen and acetylene cylinders and finally a 500-gallon tank of diesel oil. Not surprisingly a conflagration broke out, during which there may have been a number of consequent explosions. This fire hampered rescue work but since it broke out after the bridge had already fallen, it cannot possible be a factor in the occurrence of failure.

Several witnesses gave evidence that they had seen the bottom flange of the box girder tear across transversely and that the span had jack-knifed as a consequence of this failure in tension of the bottom flange. Rather more witnesses described the compression buckling of the top flange and it appeared to us an unlikely coincidence that both failures should have taken place together.

In the early days of our inquiry the bottom flange at the 4–5 splice was buried under the Yarra mud and it was not until March, 1971, that the relevant parts were recovered. Examination of these recovered parts completely disproved the thesis that the bottom flange had failed in tension. The only fracture at all in the bottom plate, or its splice at the 4–5 joint, was over a short length of two to three feet. This was in the transverse splice, close to the inner web, where the cover plates themselves had failed in tension. This damage almost certainly resulted as a secondary consequence of the collapse, probably when the falling span hit the ground. Quite clearly there had been no " opening up " of the bottom flange.

The witness Simmons was certain that he had seen the bottom plate tear across in the north-south direction, and we accept that he was quite sincere in relating what he believed to be true. Simmons, at the time, was suddenly involved in an unexpected and unbelievable calamity, added to which he was, at the time, in considerable personal danger. The incident demonstrates the fallibility of the human brain under such conditions.

One witness gave a detailed account of how he had seen pier 11 disintergrate somewhere about half way up and that the collapse of the pier had been followed by the fall of the steel span. An examination of the wrecked pier showed that it was still virtually intact at, and around, the section which it was claimed had failed. The mechanics of failure described by this witness appear highly improbable and are entirely at variance with the elements of the wreck as they finally came to rest. We have investigated the design aspects of the concrete pier and seen test reports on concrete cores cut from the fallen pier and we see no possibility of a structural failure in the column as being the initiating cause of the collapse.

The suggestion that movement might have taken place of the steel span relative to the top of pier 11 was also examined. There are three possible causes which might account for such a movement. Firstly, a movement in the system of temporary supports by which the span was propped off the pier head, secondly, the movement of the pier head itself due to activities at the base of the pier ; and, finally, the possibility of pier head movement due to interference with the system of guy wires supporting the top of pier 11 against longitudinal movement.

So far as the first of these three is concerned, it must be recalled that when the half spans were lifted up to the tops of the piers they had to be rolled sideways to their correct positions over the piers. To do this, the girders were mounted on temporary skates which rolled on horizontal rolling beams (see Pl. 2). When, therefore, the girder arrived at its correct location in plan it was still several feet too high due to the fact that it was still on the rolling beam. To get the girder down to its correct level involved a complicated sequence of operations including the cutting up of the rolling beam, piece by piece, in order to remove it. All these procedures were fully set out in the Procedure Manual. The lowering operation should not have been started until the splicing of the longitudinal joint in boxes 1 and 8 had been virtually completed.

In fact, in the days preceding the collapse, work had begun at pier 11 on lowering operations, although the necessary bolting up of the longitudinal seam at that end was by no means complete. The bridge had been packed up on cribs of Wandoo bearers and a start made on oxy-cutting the rolling beam itself. Work was stopped when it was noticed that movement was taking place ; this movement was detected when the flame-cut gap was seen to be closing up.

Although we consider that the act of cutting up the rolling beam before the completion of bolting up was most unwise, and certainly contrary to the Procedure Manual, we cannot believe that this premature action had anything to do with initiating the collapse on 15th October.

The evidence shows that on that date no work of cutting or lifting was going on at the top of pier 11 ; the main lifting jacks were not at that time connected to a hydraulic pressure line and so could not have been operated. Such work as was going on at the top of pier 11 was preparatory work for the subsequent jacking and lowering operations.

On the second point relating to work at the base of pier 11, it must be recalled that pier 11 in the finished bridge is pinned both at top and bottom. During the construction of the pier, however, the bottom was temporarily made rigid with the pile cap by constructing a short skirt wall between the pile cap and the underside of the column. This skirt wall ran right round the periphery of the column. Once the pier had reached full height another system of longitudinal stabilization was provided by fitting guy wires. These guys ran from the top of pier 11 to anchorage points near the bases of piers 10 and 12. Once the guy wires were in position, it was decided to cut out the skirt wall. The decision to do so was sound, but the way in which the operation was conducted was not. The east side of the wall was demolished first, leaving the west side still under some load, thus tending to exert a bending moment about the otherwise pinned base of the column. This moment would tend to make the head of pier 11 move towards pier 12. Ward pointed out to Hindshaw the wall ought to be demolished symmetrically on its east and west sides, but Hindshaw indicated that the piers were the responsibility of Maunsell and told Ward to contact James. Shortly after this work on cutting out the skirt wall stopped, and remained in the half completed stage until the bridge collapsed. We feel that while it was unwise to chop out one side of the skirt wall while leaving the other intact, it was even more unwise to leave the situation in the half-finished state and thereby set up a tendency for the column head to move. Despite this, we have no evidence which suggests the column did in fact move and we do not think that the method used in demolishing the skirt wall to pier 11 had anything to do with initiating the collapse of span 10–11.

Finally, it has been suggested that movement of the column head was caused by interference with the guy wire system. This interference is presumed to have come from some load being lifted at the time. We have carefully examined the evidence on operations going on immediately prior to the failure and it does not appear that any lifting at all was being undertaken either at the time of the failure or for some minutes before. Neither is there any suggestion that any substantial weight fell from the bridge in a manner likely to foul the guy wires.

We conclude, therefore, that no movements of the bridge girders, relative to column 11, took place immediately prior to the failure, which could have had any influence on the collapse.

2.3.2. THE DYNAMICS OF THE FALLING SPAN.

The relative positions in which some parts of the wrecked span were found did, at first sight, present something of a puzzle as to how the span had fallen, once the buckle at the 4–5 joint had taken place. For example, massive components from the temporary bearings on which box 8 had been seated before collapse were found in the wreck on top of the upper flange. One such part came to rest at the extreme north end of the 5–6 transverse splice. This was particularly unexpected as span 10–11 had, before collapse, cantilevered about 26 feet beyond pier 11, on which the bearings had been seated.

In order to throw some light on what had taken place a theoretical analysis was made of the falling dynamics and this was backed up by a high speed cine film taken on a model designed to collapse in a manner similar to that hypothesised for the bridge.

Both these analyses were based on the assumption that in the early stages of collapse the moment of resistance at the 4–5 joint was only marginally lower than that needed for stability. As the collapse proceeded, the moment of resistance was assumed to drop to a low value.

On the evidence of observations made at the wreck and the results of these analysis, it appears probable that the collapse developed through the following stages :—

(*a*) As span 10–11 began to form into a shallow V, the over-all shortening between the ends caused the west end at pier 10 to be pulled off its roller bearing. Once off its bearing, the free end appears to have fallen at an angle about 25° from the vertical, as evidenced by the fact that the falling span just failed to clear the handrail to some scaffolding located near to the top, and on the eastern side of pier 10. It is possible that the top of pier 11 also moved east at this time, perhaps as a consequence of the partially demolished skirt wall. Any such movement would eventually be limited by the pull on the guys, which, at that stage, were still intact. The guys provided a " soft " restraint so that a movement of the column head of perhaps 10 inches might have taken place.

(*b*) Boxes 1 to 4 were then falling with only small inclination from the horizontal, certainly not enough to cause loose equipment to slide. At this stage the accelerations at about boxes 4 and 5 exceeded the acceleration due to gravity and loose objects would have left the deck. During this phase the boxes 5 to 8 rotated about the top of pier 11 and so built up a large rotational momentum.

(*c*) The falling span next fell on to the guy wires which ran from the bottom of pier 10 to the top of pier 11. The smart jerk on these wires most probably pulled the pier head 11 to the west sufficiently to break the guy wires which ran between the top of pier 11 and low down on pier 12. As the span continued to fall the tension on the 10–11 guys increased, but the pier head could not move further west because the steeply tilted half span comprising boxes 5 to 8 was reacting against it, tending to push the column the other way. Eventually the 10–11 guys also failed. One of the saddles which carried the guys over the top of pier 11 was recovered with the wire ropes broken at both the east and west ends of the saddle.

(*d*) Boxes 1 to 4 next hit the ground still more or less in a horizontal position. The instantaneous centre for the boxes 5 to 8 unit was suddenly transferred from the top of pier 11 to the 4–5 joint, now at rest on the mud. In view of the angular momentum stored in the 5 to 8 unit this sudden change in the centre of rotation caused a very violent acceleration around box 8. This violent shock was apparently enough to fracture the attachments of the stiff-legged derrick to the span. The entire derrick appears to have catapulted off the span, subsequently crashing back again on box 7 where the deck plating was punctured. The wrecked derrick finally slid down the steep slope to finish in the general chaos at the 4–5 joint.

(*e*) Conservation of angular momentum caused the 5 to 8 unit to continue rotating about the 4–5 hinge, so that box 8 came completely clear of pier 11 and the unit reached a maximum inclination of perhaps 60° to the horizontal.

In this condition there was a considerable axial compressive force acting on box 5 resulting in substantial secondary folding of the already buckled panels, thus reducing the over-all length of the 5 to 8 unit by about 10 feet.

(f) Gravity acted to reverse the rotation of the 5 to 8 unit and in its slightly shortened form this unit crashed back on to pier 11, but owing to its reduced length the end of box 8 hit the side, not the top, of the pier.

It is possible that the pier itself which, for a short time, had been free-standing, was oscillating with a considerable amplitude, and that the pier on one of its westward swings hit the end of box 8 as the latter rotated to the east.

In any event the shock of the collision was enough to jerk one of the temporary bearings from the top of the column on to the upper deck plate, where it was subsequently found. A number of wandoo packers used as cribs between the underside of the girder and the column head were also jerked on to the deck where they slid various distances towards the 4–5 chaos (see Plate 1).

The rolling beam used to roll the two half girders sideways from the jacking towers, was also separated from the column head by the shock of the impact. This very heavy beam, for some short interval of time hung on the end of box 8, where it left a clear impression.

(g) The pier 11 was then entirley unstable since its base was pin-supported and the guys had been severed ; furthermore, the still rotating 5 to 8 unit was forcing the pier to fall to the east.

As the pier fell, rotating about its base, the heavily stiffened centre portion of box 8 scarified the face of the pier showing that throughout this stage the two were in close contact.

(h) The pier, on hitting the water and mud, was for an instant supported throughout its 160-ft. length on a virtually frictionless medium and shot forward about 40 feet to the east, coming completely clear of the pile cap. Shortly afterwards box 8 hit the remains of the skirt wall on the west end of the pile cap and this arrested further movement of the box girder. The shock of this last contact shook free the rolling beam which had been precariously lodged on the end of box 8. The beam appears to have turned into a nearly vertical position and then speared down to impale itself into the shattered pier 11.

This reconstruction of the dynamics of failure which is set out in diagramatic form in Fig. 13 agrees well with the evidence of the witness Smith, who watched the collapse from the nearby public viewing area.

PART 3. THE DESIGN.

3.1.1. DESIGN CRITERIA.

The designers of the bridge approached their task having been apprised of the following basic criteria :—

1. The Melbourne Harbor Trust required a clear width of 1,000 feet between piers across the river Yarra and a minimum vertical clearance of 170 feet over a 600-ft. width.

2. The design traffic speed was to be 70 m.p.h., the minumim radius, in plan, 2,900 feet and the maximum super-elevation 1 in 25.

3. Two 55 ft. 2 in. wide carriageways were to be provided, each for an initial four lanes of traffic with a break-down lane allowing for a possible extension to five lanes operation without break-down lanes.

4. No footways were to be provided.

5. Provision was to be made only for those services directly concerned with bridge operation.

6. The loading was to be H20–S16 in accordance with the N.A.A.S.R.A. Highway Bridge Design Specification 1963 with special lane load reduction applicable to loaded lengths in excess of 400 feet, the design to be checked for a 240-ton special load, running on the two lanes nearest the median.

7. The maximum design wind speed was to be 100 m.p.h.

3.1.2. THE CABLE STAYED BOX GIRDER BRIDGE.

The cable stayed girder bridge was developed in Western Europe in the period of reconstruction which followed the ending of World War II. It has been found that bridges of this type are well suited for spans in the 500–1,200-ft. range.

At the time of commencing construction on the West Gate Bridge there was no cable stayed girder bridge in existence with a span as large as the proposed 1,102-ft. central span. Subsequently three larger bridges of the same type have been started (3) : The Duisberg-Neuenkamf Bridge in West Germany, with a central span of 1,148 feet, and two bridges over the Paraná River in Argentina, each with a central span of 1,116 feet, due for completion in 1972.

By any standards the West Gate Bridge will be one of the world's major bridges.

The choice of a cellular box section is almost universal for cable stayed girder bridges. In comparison with plated I section girders, the box girder has the advantage of concealing all the web and flange stiffeners, thereby improving appearance and reducing maintenance costs. Another important characteristic of the box girder, when used on these long, slender bridges, is the high torsional rigidity, a most desirable characteristic if the effects of aerodynamic flutter are to be kept to an acceptable minimum ; this feature is particularly important when, as in West Gate, the cable stays are kept in a single vertical plane on the longitudinal centreline of the bridge.

The use of a single plane cable system in conjunction with a torsionally rigid box girder reduces the weight of the cable stays to about 70 per cent. of that needed for a dual cable system. This is because the mono-cable is much less affected by asymmetrical live load conditions.

A trapezoidal section for the box girder, in which the outer webs are inclined, is a natural development which provides for a wide top flange, where it is needed as a deck for traffic, and at the same time a narrower bottom flange which is structurally more efficient. The trapezoidal section also has the superior characteristics of less wind drag and greater aerodynamic stability.

A box girder section composed of stiffened plating was used as early as 1850 by Robert Stephenson in the Britannia Bridge across the Menai Straits, but there has been no continuity of experience since that time and it is only in recent years that this form of girder has achieved popularity among bridge designers.

The proper structural analysis of box girders is found to be a complex matter and this is particularly so when, as in West Gate, the girder is continuous over many supports, has a section divided by internal webs into multiple cells, and finally, includes sloping outer webs to give a trapezoidal section.

A review of the research literature on the subject of box girders (4, 5, 6, 7) shows that this is still an area where much research work is currently being undertaken ; and that some aspects of the stress analysis of these structures remain so far unresolved.

It is true that the use of simple theory will give answers which in many cases are sufficiently accurate for normal design requirements. In view of all the imponderables to be encountered in such sophisticated structures, and with theory pushed to the limits of engineers' knowledge, it is essential to include safety factors high enough to provide sufficient allowance to cover the inevitable crudities of the analysis and other unavoidable factors and imponderables which may arise.

3.1.3. General Features of the Design.

In one sense, every major bridge is unique in so far as it brings together in a unique combination the several component features of the design. These individual components may have been used many times before on other bridges.

The particular elements which characterize the West Gate Bridge are—

(1) Cable stays and supporting towers in a single vertical plane on the longitudinal centre line of the bridge.
(2) Trapezoidal box section for the girder with two internal vertical webs.
(3) The use in appropriate places of low alloy high yield steel for much of the box girder.
(4) The careful control of steel quality and construction techniques to aviod brittle failures in the steel. These measures include limiting the thickness of the steel plates and the avoidance when possible of transverse welds across plating subject to tensile stresses.
(5) The fabrication of panels of stiffened plating in the fabrication shop ; the panels being standardized so far as that is possible. The size of prefabricated panels was the largest possible consistent with the availability of plate and the handling facilities.
(6) The sub-assembly of panels into units, called boxes. In the West Gate Bridge each box was 52 ft. 6 in. long by 41 ft. 9 in. wide. Two such boxes made to opposite hands comprised the full cross-section of the box girder, 83 ft. 6 in. wide at the upper flange.
(7) The use of high strength friction grip bolts for all connections other than those made in the fabrication shop. This included the connections in assembling each box as well as all site connections between boxes.
(8) The use of a reinforced concrete road deck made to work in a composite manner with the upper flange plate by the use of stud type shear connectors welded to the upper flange plate.
(9) The use of cantilever brackets from the main box girder to extend the overall width of the road, in this case to 122 feet overall.

All the above features have been used successfully on other bridges but not necessarily in this same combination.

3.2.1. Structural Arrangement.

The general description of the structure has been given in section 1.2.2., and further details can be seen in Figs. 1 to 9 inclusive.

The cable stay system operates only between columns 11 and 14. Spans 10–11 and 14–15 are therefore unstayed box girders, roller supported at the outermost ends but pin supported and continuous at the inner ends. The investigation into the causes of failure of span 10–11 was this limited to a comparatively simple structural system.

As mentioned at the end of section 1.2.2. the transition curves of the approach viaducts are carried over on to the steel bridge. The actual transition length is from the end up to the joint between boxes 15 and 16, the super-elevation of the deck being reduced to zero over this length. The way in which this has been achieved is shown in Fig. 2. It will be seen that the boxes on the concave side of the curve have a constant deck slope of 1 in 40, for drainage purposes. The boxes on the convex side have cross slopes of the deck which change from 1 in 40 upwards at box 1, to 1 in 40 downwards at the 15–16 joint. This twist on the boxes on the convex side of the curve, i.e. the north boxes of span 10–11 and the south boxes of span 14–15, amounts to about 23 minutes of arc per 100-ft. length. To achieve this twist when fabricating the boxes, it was necessary for all the panels so affected to be made as rhomboids, slightly off the perfect rectangle. In practice the panels themselves were kept rectangular but the holes round the edge were set out on lines which formed a rhomboid. The use of transition curves on the steel bridge is seen, therefore, to have introduced a considerable complication in detailing and to have resulted in many non-standard panels.

It will be seen from Fig. 2 that the two half boxes are rotated relative to one another about the upper longitudinal splice. This means that the gap at the lower splice was changing over the length of the transition curve. In fact the gap started at $7\frac{5}{8}$ inches at box 1 and tapered out uniformly to $\frac{1}{2}$ inch at the 15–16 joint. As can be seen, this non parallel gap required non standard cover plates. In the erection stages until the cover plates were fitted, this open gap created a saftey hazard, since items of equipment and loose fittings could and did drop through.

The change of super-elevation meant also that every inner diaphragm between boxes 1 and 15 was a different shape and had to be tailor made to suit the particular deck slopes needed. The diaphragms for similarly numbered boxes on the East and West sides were, however, identical.

All this special setting out needed for the " twisted " half-boxes did without doubt increase the risk of fabrication errors. Evidence of such errors is clear from Ward's note book in which he records errors of $\frac{1}{8}$ inch in the location of holes in some of the inner diaphragms.

Errors in the setting out of holes would increase the difficulty of erecting the boxes and could have been a contributory factor in the failure to maintain the proper longitudinal camber in the half spans 10–11 and 14–15.

3.2.2. The Stiffened Panels.

One of the features of box girders is the relatively thin plates which can be used for the flanges. This arises because the flanges are so wide that only thin plates are needed to provide the required cross-sectional areas. Such thin plating must be stabilized against buckling in compression by the addition of suitable systems of stiffeners. Another factor which arises with these very wide flanges is that the use of the simple theory of bending may lead in some cases to significant errors due to the fact that plane sections, on bending, do not necessarily remain plane.

In the West Gate Bridge it was intended that for the service condition the concrete deck slab, made to act compositely with the upper flange plating, would contribute significantly to the stabilising of that plating. For the pre-concreted stages of erection it was, however, necessary to provide steel stiffening ribs sufficient to stabilise those parts of the upper flange which were in compression. It was the failure of these panels of stiffened plating in the upper flange which led to the collapse.

The thinnest plating used on the bridge was 5/16 inch, which was used on intermediate transverse diaphragms, that is to say those not over a permanent pier or a temporary prop. One such diaphragm was fitted to each box and located 2 ft. 9 in. away from the " river end " of the box.

All the flanges and webs, both external and internal, were a minimum of $\frac{3}{8}$ inch thick. This thickness was used almost throughout on the upper flange and outer web, although in the former there were short stretches of $\frac{1}{2}$ inch used around support points, either piers or cable anchorages.

The inner web varied between $\frac{3}{8}$ inch and $\frac{1}{2}$ inch while the bottom flange ranged in thickness from $\frac{3}{8}$ inch to $\frac{7}{8}$ inch, the latter around the points of high negative moment at piers 11 and 14.

All the material was high yield steel except for those areas where the stress in $\frac{3}{8}$-in. plate fell so low that mild steel could safely be used. The mild steel was specified to ND11 quality to avoid any chance of brittle failure.

Fig. 7 shows the thickness of plates and size of stiffeners, as shown on the tender drawings and also as finally fabricated by WSC.

At the failed section, around boxes 4 and 5, all material was high yield steel, except for the inner web plate and its connecting brackets and attached stiffeners.

The longitudinal stiffeners to the panels were in general bulb flat sections welded to the plate with the bulb outermost. The bulb flats were imported from the United Kingdom as the section is not rolled in Australia.

Longitudinal stiffeners under the top flange were shown on the tender drawings at 42-in. centres, but this was modified by WSC to 21-in. centres in some boxes. This modification was made to cater for some of the erection conditions. The spacing of longitudinal stiffeners was 18 inches for the bottom flange, 24 inches for the inner web and $30\frac{1}{2}$ inches for the outer web.

The transverse stiffeners to both upper and lower flanges were in general at 10 ft. 6 in. centres. The section of the transverse stiffeners was in two parts, as can be seen in Fig. 8, first a flat plate through which the longitudinal bulb flats were slotted and which was attached by welding to the individual panels, and in consequence, discontinuous at each panel edge, and secondly a continuous outer bent section bolted to the first plate and spanning in one length between adjacent main webs.

It was intended that the discontinuous plates should be fillet welded to the flange plates only in areas where the flange plate was in compression. For all tension areas the fillet weld was to be omitted, and in these cases the only connection between the discontinuous plates and the flange plate was through the short runs of fillet weld which attached the longitudinal stiffeners to the flat plate where they were slotted through the latter. The longitudinal stiffeners were of course themselves welded to the flange plate, but this indirect connection of the transverse stiffener to the flange is of doubtful efficiency in so far as making the transverse stiffeners and part of the flange act as a combined section to provide transverse flexural rigidity. The way in which these indirect connections failed during the collapse can be seen in plate 5. The fillet welds to the longitudinal bulb flat have sheared, leading to the complete isolation of the transverse stiffener from the plating. The members shown in the photograph had of course been subject to quite exceptionally high forces during the failure.

The omission of the transverse weld on tension plating no doubt originated from Wilson's very great concern that there should be no brittle fractures. With the unusually severe control of steel quality for the plates and the relatively thin sizes used, it is unlikely that brittle fracture could have developed. It appears to have been overlooked that the bulb flats themselves, equally subject to tension stress, do have transverse welds. The quality checks on the imported steel for these bulb flats were somewhat less stringent and indeed testing after the collapse has shown some with stresses considerably higher than the specified yield.

The logic behind these indirectly connected transverse stiffeners is questionable, particularly when it is noted that in the very highly stressed tension area of the top flange at box 8, the principle was abandoned and the transverse stiffeners were fully welded to the plate.

An unusual feature in the arrangement of transverse stiffeners is that stiffeners under the outer upper flange plate, which are themselves continuous with the external cantilever arms, do not fall opposite the stiffeners under the inner upper flange plate, but are displaced 5 ft. 3 in. longitudinally from them.

It appears that a certain amount of confusion existed even in FF & P's own office over this odd arrangement. Both tender drawings T4 and T21 (Figs. 3 and 4 of this Report) show the inner beams incorrectly in line with the outer beams. Hardenberg stated that he had been misled by the errors on these drawings and this was in part the reason why he had failed to recognise the potential instability of the inner upper flange panel, and hence the indirect cause of the buckle which happened on the half span 14–15 north.

The transverse stiffeners without continuity were virtually pin ended at their connection to the inner web so that, in the half box as sub-assembled, the half length upper inner transverse stiffeners would be practically useless until bolted to their opposite numbers on the other half span. They would, in effect, be cantilevers with almost no moment capacity at the built-in end. It was this lack of rigidity which made necessary the various systems of temporary bracing, described in Section 2.1.2.

3.2.3. Transverse Splices.

The transverse splices between boxes were all made with friction grip bolts. Splices to all the plates consisted of double cover plates, but the fish plates used to cover the longitudinal bulb flat stiffeners were single only.

In general, the longitudinal bulb flat stiffeners were stopped 6 inches from the panel ends so that either a single or a double row of bolts could be used for the plate splice without changing the standard panel detail. This meant that the fish plates splicing the bulb flats, referred to as the " K-plates ", were a nearly standard length although this length was longer than the minimum which could have been used where the plates were connected by only a single row of bolts in the transverse seam.

The standard gap between the facing ends of bulb flat stiffeners was thus about $12\frac{1}{2}$ inches, allowing for a $\frac{1}{2}$-in. gap between boxes. On the flanges at joint 8–9, however, a triple bolted joint was used and in this case special K-plates had to be used covering gaps of up to $22\frac{3}{4}$ inches between bulb flats.

The K-plates for all bulb flats in both upper and lower flanges were of 4 in. x $\frac{1}{2}$ in. section, high yield steel. It was perhaps unfortunate that the sectional area of the K-plate was some 10 per cent. less than that of the bulb flat which it was splicing. This is particularly so when it is clear that considerable bending effects must also be induced into the K-plates by their eccentricity to the centroid line of the bulb flats. The high slenderness ratio of the normal K-plate and the even higher ratio for the long K-plates at the 8–9 joint further reduced the load carrying capacity of the K-plates.

We are in sympathy with the views of Grassl when he stated that in his capacity of " Pruefingenieur " in Germany he would have rejected the K-plate detail out of hand without wasting time on calculations.

Any meaningful stress analysis of the K-plate, acting as a component in the particular structural system would in any case, be quite impractically complex. It is obvious, without having to do any elaborate calculations, that the K-plate is less than 100 per cent. effective as a splice for the bulb flats. Engineering judgement should have warned both WSC and FF & P that this reduction in efficiency was likely to have serious consequences on the overall structural behaviour.

That the K-plate detail was unsatisfactory is clear beyond any shadow of doubt. It is no good claiming, as FF & P did, that if the stresses in the bulb flats were low enough the strength of the splice might still be " adequate ". The fact is the stresses in the bulb flats were not low, and on at least two occasions prior to October 15, 1970, K-plates, either singly or in groups, had buckled. In any case, it is bad engineering practice for an important structural element in a design such as this to be connected by a splice that is less strong than the element itself.

Late in 1970 FF & P appear to have realized that some of their design procedures for box girders required examination and an internal design memorandum was drawn up for use inside the FF & P organization. We have been shown a copy of the fourth draft of this document, dated 30th November, 1970. It has this to say on splices in stiffeners —

> " Splices in stiffeners should develop the full relevant strength of the stiffener . . . . and should be of at least the same area as the stiffener and in compression zones they should in addition be of at least the same second moment of area as the stiffener about each major axis. Furthermore, splicing plates should be disposed so that their neutral axes coincide as closely as possible to the neutral axes of the parent stiffeners, and due allowance should be made for any eccentricity ".

We fully agree with the views expressed in this recommendation. The splice plates as fitted to the West Gate Bridge, however, failed to comply with every single one of the several conditions set out above. Had the plates been designed to comply with the 1970 FF & P design memorandum, one of the major contributing causes of the disaster would have been removed.

The use of a single cover plate on one side only of the bulb flat stemmed from the detail given by FF & P on the tender drawings. On the panel drawings they fully dimensioned the location of the holes in the ends of the bulb flats in such a way that the only rational interpretation was that a single cover was intended. FF & P did not, however, give a section for the splice plate and WSC must carry the resonsibility of having selected a flat section and of choosing the size, 4 in. x $\frac{1}{2}$ in., although FF & P, in checking WSC's drawing, approved of this without raising any objection.

Brown, in a written statement, claimed " the tender drawings called for double covers at the splices, but Werkspoor sought permission to change these for single covers on bulb flats ". We have not been able to find any note on any of the tender drawings that states, either directly or indirectly, that the cover plates for the bulb flats splices shall be either single or in pairs. No evidence has emerged to show that WSC did in fact make any special request to FF & P for permission to use single covers. We conclude Brown's statement on these points is untrue.

As early as 29th October, 1969, McIntosh wrote to FF & P in London, pointing out the weakness of the K-plates, saying that in his view their deficiency was one of the reasons for the buckling which took place on the inner upper panel of half span 14–15 north. He did not get a reply to his letter and a month later, on 24th November, 1969, McIntosh wrote again, specifically advising that a check should be made on the design of the K-plates. A hand-written note on the original of this second letter, added presumably in London, reads thus —

> " We understand this was all covered by SGR during his visit. We are not too worried about this at all since it is all elastic ".

" SGR " refers to Sir Gilbert Roberts, and the comment on it being " all elastic " presumably refers to the buckling which took place on 14–15 north. There is no evidence at all that Roberts did deal with the matter when he was in Melbourne late in October ; indeed, if he had done so, there would have been no need for McIntosh's second letter. Whoever made the comment on it being " all elastic " was very badly informed. The surveys by Hardenberg and photographs taken at the time prove that the flange plate itself, the bulb flats and the bulb flat K-plates were all stressed into the post elastic range. Indeed the bulb flat stiffeners at box 4 on the still standing span 14–15 show very high out of straightness which is a permanent result of their yielding when the panel buckled.

Despite these warnings, FF & P took no action at all to re-examine the K-plate detail until after the collapse of the Milford Haven Bridge on 2nd June, 1970. At that stage, Maunsell, London, were asked to make an independent examination of the structure and in their first interim report, dated 25th September, 1970, they state that " we have noted that the fish plate detail adopted for the joint in the bulb flat stiffeners substantially reduces the buckling capacity of the compression flange panels ".

At this same time FF & P conducted their own stress checks (see 4.2.4) and as a consequence stiffening measures were put in hand. As a matter of urgency some highly stressed K-plates already erected were buried in a matrix of concrete to prevent instability of the plate. In some other splices not yet erected it was decided to substitute a 6 in. x $3\frac{1}{2}$ in. x $\frac{3}{8}$ in. angle for the 4 in. x $\frac{1}{2}$ in. K-plate. Such a section would meet almost all the requirements of the draft code of FF & P referred to above.

Although a number of these angle section splices had been fabricated, none had, in fact, been fitted to the structure when span 10–11 collapsed. The extra cost of angle sections would have been trivial compared with the overall cost of the steelwork. The fact that they, or something like them, were not proposed in the first instance reflects adversely on WSC for proposing the flat K-plate and on FF & P for approving it.

Whatever criticisms may be offered about the K-plates, it is obvious that the situation is very much worse if they are left off altogether. It is surprising, therefore, to learn from surveys of the wreckage that several K-plates were not in place at the time of the collapse. All five K-plates on the inner upper panel at box 4 north were missing, but we know that four of these had been taken off after they buckled early in September, 1970, and the remaining one had never been fitted. At several other places, K-plates were missing and had clearly been missing prior to the collapse. In one case the plates had apparently been removed because they fouled pulling brackets fitted to close the horizontal gap between the two half girders. At other places there was no clear reason why the K-plates were missing. An example can be seen on Plate 11, where the plate is missing on the third stiffener from the bottom, on the inner web.

The casual way in which these K-plates were left off, or removed from the box girder while it was under stress reflects, adversely on JHC, but also on FF & P for allowing such a state of affairs to exist.

The stress analysis for the K-plates is dealt with in Section 4.2.4.

3.3.1. Design Procedure.

The practice among British consulting engineers, when designing a major bridge, is to design for the final service condition of the finished bridge, leaving the analysis of the stresses during the erection stages to the successful tenderer. It is claimed that this system allows contractors greater freedom in putting forward proposals for novel methods of erection which may achieve economies. The system also allows each tenderer to plan an erection method in keeping with his own particular plant and previous experience.

Some consultants refuse, as a matter of principle, to give a detailed check to the erection proposals of the tender on the grounds that to do so might, in some way, reduce the liability of the contractor to be completely responsible for the works.

Other consultants, in the interests of the client, do accept the responsibility of checking the erection calculations made by the contractor. FF & P, at least in principle, were in this latter category ; they demanded sets of calculations from WSC and several of the FF & P witnesses agreed that it was their duty and intention to have the erection calculations of WSC checked. After the change-over, when JHC replaced WSC as contractors, the situation was somewhat changed. JHC were not in a position to undertake stress calculations, and not having been provided with the basic data, they were not in fact required or encouraged to do calculations. As a consequence, whenever there were departures from the methods established by WSC, it devolved on FF & P to do the stress analysis for the new erection conditions. This was a distinct departure from normal British practice. The main area in which FF & P found themselves involved in analysing the erection stresses was in the development of a quick and safe programme of placing the concrete deck. As has already been stated, FF & P claimed that this analysis was inseparable from the over-all stress check they were carrying out to satisfy Wilson that all was well.

Australian consulting engineers do not always follow the British practice and they may, and frequently do, work out erection procedures in some detail for inclusion in the pre-tender design. Any contractor wishing then to tender on a variation of the procedures laid down may do so, provided he can demonstrate to the consulting engineer that the proposed variations are safe.

The West Gate Bridge exemplifies this difference very well. For the pre-stressed concrete approach spans designed by Maunsell the erection procedures were fully specified and detailed. The successful tenderer JHC put forward a few modifications and these were subsequently approved by Maunsell. On the other hand, FF & P left the tenderers a free hand so far as erection procedure was concerned.

While it is true that the British practice means that the consulting engineer is primarily concerned with the safety and efficiency of the bridge in service, he must have some regard to the limits imposed by erection problems. There must be at least one practical method by which the bridge can be erected.

In the early days of design, FF & P did give some thought to possible erection methods and a note on how the bridge might be erected was included in one of the preliminary reports to the Authority. Some calculations made by FF & P early in 1967 on possible erection methods have also survived.

When the principal dimensions of the bridge had been established and the tender design carried out, in mid 1967 it appears that no serious regard was given to the problem of erection, even though one of the computer runs was claimed to be an " erection analysis ". The analysis made at that time was limited to an examination of the stresses in the steel bridge complete except for deck concrete and another similar analysis for the stresses in the bridge with all the concrete in place and already hardened so as to act in a composite manner with the steel. The instantaneous placing and hardening of the whole concrete deck is scarcely a practical condition and a little thought would have shown that any practical scheme for concreting was likely to impose higher stresses at some places during erection.

By adopting this over simplified approach, FF & P achieved a design which they believed was satisfactory for the service condition, but required considerable extra strengthening at many places to meet the stresses imposed by any practical erection scheme.

At this stage the situation was still under control, since in theory WSC would be able to ascertain what extra stiffening was required, but to do so they would have to understand just how the structure was designed. Contract S included a provision for an exchange of design information as follows :—

> " To ensure that erection calculations will be in accordance with the engineers' design criteria—notably the end conditions under full dead load FF & P will supply to Werkspoor a set of bridge design calculations ".

Despite repeated request from WSC, FF & P resolutely refused to hand over any calculations. Roberts, who claimed he was unaware at the time that this clause had got into the contract, stated that the request could not have been complied with as no suitable calculations existed at that time which could be handed over. If he is correct then we can only deplore this reprehensible state of affairs.

Beneficial co-operation between WSC and FF & P thus broke down at an early stage so far as calculations were concerned. The evidence shows that FF & P were principally to blame for this unhappy state ; in addition to refusing to hand over a copy of their calculations they rarely replied to correspondence asking for technical information.

In the outcome WSC found certain areas of the structure where additional stiffening was needed. According to their calculations some of this extra strengthening was required not only for the erection conditions but for the service condition as well.

WSC submitted their calculations for checking by FF & P and in the absence of any critical comment assumed they were satisfactory.

From then on future trouble was almost inevitable. WSC had imperfectly understood the over-all structural behaviour and FF & P, if they checked at all, had failed to detect the flaws in the WSC analysis. The result was that certain elements which would have become over-stressed in erection, and some even in service were not strengthened. From this regrettable lack of co-operation sprang the panel buckling on span 14–15 north. Most of the stiffening added to West Gate after the Milford Haven disaster was to strengthen details overlooked earlier and even the fatal buckle on span 10–11 can be traced back, at least in part, to the inadequate analysis by WSC and inadequate checking by FF & P.

3.3.2. The Procedure Manual.

When JHC took over Contract E from WSC they were virtually committed to carrying on the erection procedure already started by WSC. This was so because of the existing heavy commitment in plant and the long delays which would result from any changes at that stage.

JHC argued that as they were not a party to the evolution of the erection method, and as they were not conversant with either the original design calculations of FF & P or the erection calculations of WSC, they could hardly be expected to direct the erection sequence without help from FF & P or WSC., or both. JHC took the view that they would do precisely what others, presumably FF & P or WSC, told them, but would not be responsible for anything untoward which might occur unless it was due to gross negligence on their part.

In order to make this sort of arrangement work, JHC were required by Contract E to draw up a Procedure Manual which set down, often in the greatest detail, the various operations which had to be carried out. The manual had little to say about the bolting together of boxes 1 to 8 and raising of these up the jacking towers since that operation was essentially completed before the manual could be drawn up.

It is a pity that no procedures were set out for getting the required camber of boxes 1 to 8 on the " floating " trestles. WSC could not help much in telling how to do this for they had been unable to work out a satisfactory method when doing this operation themselves on span 14–15 east.

The manual was not a single document, once and for all, but was continually evolving, new sections being added and others being amended. At the time of the failure of span 10–11 almost all procedures had been written up but the operation of making the joint at mid-span 12–13 had yet to be set down.

Although JHC had the responsibility of producing the manual, by the very nature of their lack of this particular experience they could not do so without help from WSC and FF & P. The whole document was the result of a co-operative effort by the engineers from WSC, JHC and FF & P.

The whole point of the manual was that all procedures should be set down and that JHC would then be made to abide by these written procedures.

In fact, FF & P on occasion authorized procedures which were different from those set out in the manual. The use of kentledge is an example of this. On other occasions FF & P authorized procedures which were not set down at all ; an example of this is the unbolting of transverse seams in order to remove buckles, an operation which was safely conducted on the east side, but brought disaster when attempted on the west.

JHC did in fact from time to time proceed not in accordance with " the book ". Whether these deviations by JHC were really serious or whether they were " blown up " by Hindshaw to be made to appear serious, for political ends, is not easy to determine. We are inclined to take the latter view.

3.3.3. SAFETY FACTORS.

In statements to the Authority FF & P made it clear that, where appropriate, the design stresses would be in accordance with BS153, including the increase of 30 per cent. above working stresses for the erection conditions, as set out in that British Standard.

In the minutes of the earliest technical meeting held between FF & P and WSC on 12th March, 1968, which was even before the contract with WSC had been signed, this statement was repeated, but with the significant additional comment—

> " Due attention will be given however, to stability problems, plate buckling in particular ".

Briefly the safety factor for the service condition in BS153 is 1·70, although it would be more correct to describe this as a load factor, which may be defined as that factor by which the actual loads must be multiplied such that an elastic analysis using the factored loads will result in yield stress just being reached at some part of the member.

The elastic analysis involved must of course take into account all the factors operating, particularly when dealing with panels of plating under compression or shear.

For the erection condition the 30 per cent. increase in working stresses has the effect of reducing the safety (or load) factor for erection conditions to 1·31, i.e., 1·70 ÷ 1·30.

It is important to recall that when BS153, or for that matter any other similar code, quotes allowable stresses in compression members, it means that if the axial load on the member divided by the cross-sectional area is equal to the allowable stress, then yield would be reached at some point in the member if that load was increased by the load factor.

For example an allowable stress for high yield steel might be, say, 8·0 tons per sq. in. and for this an axial load of 100 tons would need a cross-section of 12·5 sq. in. Increasing the load to 170 tons, by bringing in the load factor, would cause the stress at some part to reach the yield, 23·0 tons per sq. in. This non-linear relationship between load and stress arises because of the interaction of the axial load on the imperfections in the strut, such as any departures from straightness of the axis of the member. Also included will be the influence of residual stresses in the unloaded member caused by the action of rolling or welding.

It is clear that the allowable stresses quoted in the codes remain valid only so long as the imperfections in the actual members do not exceed those incorporated into the code formulae. Control of such imperfections, particularly initial straightness, is, therefore, most important. The way in which FF & P exercised, or failed to exercise, control over initial straightness is discussed in the next section.

BS153 also sets out the various combinations of forces and actions which must be taken into consideration when ascertaining whether the actual total stresses are within the limits prescribed by the code. FF & P in presenting the stress analysis for the service condition included only the dead and live load and ignored other effects such as—

Wind pressure effect,

Temperature effect,

Resistance of expansion bearing to movement,

all of which, according to BS153, should have been included. These minor effects, ignored in the primary calculation, thus became eroding actions on the safety factor, more dangerous because their significance, either singly or collectively, was not assessed.

The preamble to BS153 states that it applies to the design of girder bridges, of spans up to 300 feet. Neither box girders nor continuous span are specifically mentioned, but where appropriate the specification can be used, at the engineer's discretion, for other cases.

It was clearly the intention of FF & P to use this latter provision when they repeatedly stated that they intended to use BS153 as a basis for determination of allowable stresses.

It has been suggested to the Commission that because West Gate Bridge was a cable stayed box girder bridge with spans exceeding 300 feet., BS153 should not have been used. We disagree with this argument since many of the design elements in West Gate might equally well have been found in a bridge which fully complied with the BS153 limitations.

Applicable or not, it remains a fact that FF & P said they intended to use BS153 and by implication the safety factors therein contained. Indeed so far as we can determine, they did so up until July, 1970, when the urgent necessity to justify their design, following a re-analysis made after the Milford Haven disaster made them seek for more liberal formulae.

One of the astonishing features of the FF & P design calculations is that up to July, 1970, there is an almost complete absence of any statement on, or values of, allowable stress. Nor are there any comparisons of allowable and actual stresses. In some of the rare cases where values are given, BS153 has been used.

WSC, in making their own analysis of the erection conditions, did not stick to the BS153 stresses, particularly for the panels of compression plating. For the buckling of the plate itself, without involving buckling of stiffeners, WSC used a safety factor of 1·25. For the over-all buckling, involving bending of both longitudinal and transverse stiffeners, they used a factor of 2·0 for erection conditions and 2·6 for the service condition. For buckling of longitudinally stiffened plating between transverse beams Hardenberg stated that WSC checked by use of the appropriate Dutch code.

This regrettable lack of co-ordination between WSC and FF & P arose as a consequence of Brown's unwillingness to let Hardenberg see FF & P's calculations or to know the basis on which they were made—see 4.1.1.

In thinking about safety factor, it is desirable to know what items are supposed to be covered. These items, including the " factor of ignorance ", may differ in importance from element to element.

For example, simple beams and simple tie bars are elements for which the stress analysis has been tested by experiment and experience over a great number of years and over a wide range of conditions ; furthermore, such members have a known, and benign, post-yield behaviour. It follows that simple beams and ties can be designed with complete confidence so far as stress analysis is concerned. The " factor of ignorance " for such elements is, therefore, very low.

Panels of stiffened plating under compression fall into a different category. Not only are there the intangibles of initial distortions in straightness and locked up stress, but there is also considerable difference of opinion as to the precise manner in which such panels behave when loaded to failure. Furthermore, when failure occurs, it will usually be catastrophic.

Prof. G. Winter[8] states —

> " It so happens that the rigorous treatment of buckling of thin plates along classical lines of elastic stability proved to be of little practical value ".

He goes on to describe how experimental values for collapse loads in such plates differed by as much as $\pm$ 30 per cent. from theoretical analyses based on elastic stability.

Bresler, Lin and Scalzi[9] state in relation to compression plating —

> " Exact theoretical determination of the compressive strength would have to account for large deflections, inelastic behaviour of the material, variations introduced by eccentricities of loading and slight irregularities in flatness of the plate. The complexities of these variables make a purely mathematical analysis impractical ".

Other authorities could be quoted on the same lines. The conflicting evidence on these matters, placed before this Commission, itself makes the point clear. Experts could not agree on the determination of the theoretical failure loads for the panels and hence the appropriate allowable working stresses. The " factor of ignorance " for these elements is therefore large. It would appear inappropriate to use the same safety margins for such elements as those used for elements where no such uncertainty exists.

The same comment on uncertainty can be made on the stress analysis of box girders. Any survey of recent literature on these structures will reveal that many problems, including some that affected the design of West Gate Bridge, have not yet been solved.[4, 5, 6, 7] It would have been prudent to have recognized these areas of uncertainty and to have adjusted the safety margin accordingly.

In our opinion the safety factors adopted, particularly the factor of 1·31 for erection conditions, were too low for the design of compression panels in a box girder.

Because of the neglect of secondary actions such as wind, temperature, &c., the actual safety factors for the combinations of load prescribed in BS153 would have been even lower than the stated 1·31 for the erection condition and 1·70 for the service condition.

The safety factors mentioned above are what should have been achieved. Because of design deficiencies however, the actual factor under service conditions would have been considerably less than 1·70 had not extra stiffening been added following the Milford Haven disaster. We are, even now, not satisfied that the safety margin is adequate unless further stiffening is added at certain sections.

3.3.4. Straightness of Stiffened Panels.

The importance of initial straightness in determining the allowable stresses for panels of plating in compression has been pointed out in the previous section.

One would have expected that a section of the specification would have given the permitted variations from straightness for the various conditions. No such clause exists, and apart from a general statement that the plating shall be straight, no comment is made on this point.

Roberts stated that it was a deliberate policy not to quote a tolerance, because if one was set down, the contractor would aim just to hit the tolerance instead of aiming at getting the best result he could. We think this is an unfairly jaundiced reflection on the attitude of a responsible fabricator, and in any case leaves the consultants' own agents up in the air when it comes to approving panels. Perfect straightness is a mathematical concept that has little to do with practical bridge building, and someone on the site staff must know what tolerances of straightness were in the mind of the designer so that he can ensure that on site these tolerances are not exceeded.

Precisely the situation outlined above arose in 1969, when panel production commenced. Preliminary trial panels showed considerable departures from flatness and McIntosh, the resident engineer at that time, wrote to London asking FF & P what tolerances he could accept. Roberts' reply ignored the request for tolerances, describing instead the method that ought to be used to try to make panels that were flat.

McIntosh, who remained resident engineer for the production of panels throughout, was left to his own devices. It does not appear, in these circumstances, that any specific procedures were used by McIntosh or the FF & P inspectors to check out-of-flatness. They kept documented records of the panels so far as dimensions and location of holes were concerned, but their standard inspection form had no provision for entries relating to out-of-straightness.

Roberts, when questioned on this, stated that when he was in Melbourne, presumably in October, 1969, he inspected panels with McIntosh and was able to show him by example what were acceptable limits of flatness. He claims that this was sufficient and that thereafter McIntosh could be guided by the examples he had shown. Unfortunately there is no record to show that Roberts found any panels at that time which he caused to be rejected, so that McIntosh was still without criteria. Roberts admitted his examination was by eye only and not with instruments, not even with a string line, and since the out-of-flatness under discussion was of the order 1/1,000 part of the length it is hard to see that he could have established any reliable standards for McIntosh to follow.

The principal matter for concern, so far as lack of straightness is concerned, is that buckling mode which involves the crippling of the longitudinally stiffened panels over the 10 ft. 6 in. length between transverse beams. The out-of-straightness which matters for this case is that of the bulb flat stiffeners, in a direction perpendicular to the plane of the plate.

The formula given in BS153, which was used by FF & P, is basically the Perry Robertson formula with an initial unfairness allowance $\eta = \cdot 003\, l/r$. Translated in terms of the dimensions of the panels concerned, this gives an out-of-straightness of about $\frac{1}{8}$ inch. in the 10 ft. 6 in. panel. This lack of straightness should be an upper limit for any panel and not just an average value.

It should be pointed out in fairness that the Perry Robertson formula was not developed for struts having the section of a plate with ribs sticking out and in consequence these values of " allowable " unfairness must be regarded with some caution. In any case the unfairness allowance does not relate entirely to geometric imperfections and is merely a convenient way of introducing all the imperfections of the strut, including items such as residual stresses and non-homogeneities in the steel.

Despite the note of caution above, the value of $\frac{1}{8}$ inch in 10 ft. 6 in. will be of the right order and this was generally agreed as the appropriate upper limit by FF & P witnesses. Brown, on the other hand, stated that he had his own private allowable stress formula which he used when checking designs. This, he stated, was based on an initial eccentricity factor of $\eta = \cdot 002\, l/r$, i.e., only two-thirds of that adopted in BS153. Under examination on this point, Brown stated that in practice this meant that he simply increased the BS153 values (Table 4 of that specification) by a straight 10 per cent.

Brown had no reason to suppose that initial unflatness was being held to two-thirds of the BS153 values, because there was absolutely no instruction or checking to see that this, or anything else, was done. In default of any justification for the use of the lower eccentricity term, it must be assumed that whenever Brown used his private allowable stress formula he was in fact cutting into the already slender safety margin by 10 per cent., i.e., cutting the 1·31 for the erection state down to a safety factor of only 1·19.

Despite the fact that Brown stated to the Commission that he used this 10 per cent. excess when checking calculations, there is no evidence from the calculations themselves that he did, in fact, ever do so.

Brown further claimed that when designing the longitudinally stiffened panels against buckling between the transverse beams, 10 ft. 6 in. apart, he worked on the confident certainty that all top panels would be initially bent slightly downwards. He argued that, in consequence, the series of such panels behaved as if they were fixed, at least partially, at the transverse ribs, and that this justified him in taking $0 \cdot 7l$ as the effective length of the panel. The logic of this later assumption is, to say the least, doubtful and the matter is discussed again in section 4.1.4. At this stage all that is necessary is to examine the facts as to what the initial deflections were and not what Brown assumed them to be.

The first, and, so far as we can determine, only attempt of any engineer to relate design assumptions to actual conditions on the bridge was that made by Wallace. In August, 1970, Wallace was back in London working on the Maunsell, London, check, during the course of which he became aware of the importance of initial unfairness and also of temperature effects. On 19th August, 1970, Wallace wrote to Fernie asking for site data on measured unfairness of panels and also on measured temperature differentials. Oddly, Wallace restricted the request for out of straightness of longitudinal stiffeners to the bottom flange, but we can reasonably assume that much the same values would occur on the panels in the top flange. Wallace received a reply to his request on 2nd October, 1970, only two weeks before span 10–11 collapsed. The maximum out-of-flatness of any bulb flat in the bottom panels was stated to be $\frac{5}{8}$ inch over the 10 ft. 6 in. length. This is five times the Perry Robertson allowance assumed in BS153, and would reduce the appropriate allowable compression stresses of any such panel to considerably less than the Perry Robertson values. Surprisingly, this one case appeared to be isolated and the average initial distortion, in the report to Wallace, was said to be " nil ".

During the hearings before the Commission the matter of initial shape became of such importance that we caused surveys to be made of some of the panels in the upper flange of the spans still standing on the East side. These showed that in two bays of the inner upper panel in box 4 north there were out of plane distortions of the bulb flats of around $\frac{5}{8}$ inch in the 10 ft. 6 in. between transverse beams. There is no doubt that these particular very large deflections are the result of permanent set caused when post yield curvature was induced in these bays at the time the half span 14–15 north was lifted. These surveys were extended to cover the much smaller normal deflections around boxes 4 north and 5 north, but the survey techniques then used were unreliable for the smaller measurements, and no positive conclusions could be drawn.

At that stage of the hearings of the Commission, April, 1971, Kerensky was still confidently claiming that Brown was right and the deflection of all upper panels was slightly downward, thereby justifying Brown's use of a reduced effective length.

The matter was finally settled by a further inspection of span 14–15 in the presence of two of the commissioners and senior counsel. By use of a straight edge it was amply demonstrated that while the majority of the panels were dished slightly down, as Brown claimed, every now and then the pattern was upset by a panel being dished upwards. Most significantly it was found that around box 4 and 5 in the middle of span 14–15 there were several longitudinal runs where panels were dished up and dished down alternately, which is, of course, the condition for which the effective length for elastic buckling is $1 \cdot 0l$. In general, the magnitude of the out of plane distortion was less than $\frac{1}{8}$ inch in the 10 ft. 6 in. panel.

In summary we say that even if the design assumptions had been normal there was an obligation on the part of FF & P to see that the resident engineer was informed on flatness tolerances. The very special assumption which Brown claims he made, namely the assumption of unfairness being only two-thirds of that allowed for in BS153, and the assumption that all panels were dished only one way imposed a much greater obligation on FF & P to see that the design assumptions were, in fact, met in actual fabrication and erection.

No evidence has been offered to show that FF & P made any attempt to meet these obligations.

Brown's assumption of $0 \cdot 7l$ as the effective length, we regard as one which would seriously imperil the safety of the structure. We consider it a matter of dispute as to whether the reduced effective length is justifiable even if the condition on initial distortion of panels were to be met. When, as we know, this condition was not met the assumption is clearly unacceptable.

Brown's attitude to this we find incomprehensible. He must have known at the design stage the considerable difference in allowable stress between a panel with an effective length of $0 \cdot 7l$ and one with an effective length of $1 \cdot 0l$. If he had based his design on the $0 \cdot 7l$ it appears to us to have been essential that he made absolutely sure that the conditions he had assumed were met on site. Instead of communicating any of this to the site staff Brown says he knew it was all right because he had worked out theoretically by consideration of welding distortions that all panels were dished downwards. He was wrong.

3.3.5. Loading.

In a structure such as the West Gate Bridge the self weight of the structure constitutes a major part of the load to be carried. As the design is evolved, re-assessment must be made from time to time of the self weights in order that the stress analysis relates to the loads as they actually are.

We were concerned to check that the estimates of self weight used in the design were of sufficient accuracy, particularly in so far as they affected the stresses in span 10–11 at the stage of progress reached in that span in September and October, 1970.

Detailed weight computations were made by FF & P and WSC for design purposes and we also had available the very detailed assessments made independently by Sewell and Grassl. In these latter two assessments made after the collapse it was possible to include weights of the temporary erection material, huts, scaffolding, plant, &c. as they actually were, and not as an overall blanket estimate as had to be done by the designers.

Although there are considerable local variations in the weight estimates, the overall assessments show remarkable consistency. A convenient index for the purpose of comparison is the bending moment caused in span 10–11, as it was at the time of the collapse. In the table which follows, the bending moment at the 4–5 splice is given for the north half span only. Conditions on the south half span are made somewhat complex by the presence of the derrick.

So far as the values by FF & P and Sewell are concerned, there has been no modification to the weight esitmates given. The Grassl estimate has, however, been modified to include a rolling margin on the structural plates and sections since this was known to exist and was included in the other estimates. The WSC value is based on the load estimates used in their design, but the vertical component of wind, included by WSC, has been eliminated. WSC also had a final reassessment of self weight some 3 per cent. higher than that used throughout in their calculations but this also has been ignored.

It will be seen that all values are in close accord and we are satisfied that the values used by both FF & P and WSC were realistic.

Self Weight Estimate :	BM at 4–5 splice on 10–11 north half girder.
FF & P design estimate in October, 1970 :	284,000 tons in.
WSC design estimate in June, 1970 :	294,000 tons in.
Sewell's values made after collapse :	281,000 tons in.
Grassl's values made after collapse :	286,000 tons in.

It should be pointed out that weight estimates were also made by investigators Stevens, Murray and Roderick. Because of inadequate data available to the engineers concerned, some of these estimates of self weight are too low and the conclusions drawn may, in consequence, not be correct.

So far as live load is concerned, FF & P stated to the Authority that the bridge was designed generally in accordance with N.A.A.S.R.A. 1965 specification for H20–S16–44 loading, but that " to make allowance for the probable reduction in load intensity and where applicable ", a modified loading curve as shown in Fig. 11 was used.

Fig. 11 has been copied directly from the relevant calculation sheets in the FF & P files. The design curve used follows the A.A.S.H.O. and hence the N.A.A.S.R.A. rules for load lengths up to 400 feet although the clause in those specifications which gives a minimum of 10 per cent. for impact has been ignored. For loaded lengths between 400 feet and 1,200 feet a straight line has been drawn joining the value of 4,206 lb. per foot at 400 feet to the value 2,989 lb. per foot at 1,200 feet. This latter value of 2,989 lb. per foot is arbitrarily selected as 75 per cent. of the full A.A.S.H.O. value for 1,200 feet. The values on the curves already include a separate allowance, made on statistical grounds, that only 75 per cent. of lanes will be fully loaded, they also include impact factors, although as mentioned above, these have been allowed to fall well below the stipulated minimum of 10 per cent.

The load design curve reproduced in Fig. 11 is only one part of the A.A.S.H.O. (N.A.A.S.R.A.) load requirements. The other part is a knife edge load, or, in some cases for continuous girders, two such loads, which, allowing for the fact that on statistical grounds only 75 per cent. of the eight lanes will be occupied, comes out to a value of 48·2 tons for the purpose of calculating bending moment and 69·6 tons for the purpose of calculating shear. No mention of these other loads is made in the design memorandum submitted by FF & P to the Authority and it appears that this second part of the load requirement, at least for some of the calculations, has been ignored.

It is also interesting to note at this stage, that the use of eight lanes as the basic function of the bridge ignores the Authority's design criterion that provision should be made for " four lanes of traffic with a breakdown lane allowing for a possible extension to five lane operation without breakdown lane ". In this connection it is pointed out that the two 55 ft. 2 in. carriage-ways would be required by the N.A.A.S.R.A. code to carry ten lanes of traffic and not eight as designed.

For large values of loaded length the loading curve shown in Fig. 11 already represnts a substantial reduction in the " intensity of load ". For example, the 75 per cent. reduction which applies overall (because N is greater than 4— clause 2·9 N.A.A.S.R.A. code) and the further arbitrary reduction to 75 per cent. which applies for a loaded length of 1,200 feet, means, in effect, that only four and a half lanes of " full " traffic are included, or only four and a quarter lanes if the impact effect is as specified in N.A.A.S.R.A., a considerable reduction from the eight, or ten, full lanes.

When faced with inconsistencies in the FF & P design calculations, Roberts and Brown argued that they had used their judgment to allow further reduction of the live loading on the grounds that full live loading was statistically impossible. By implication they appear to have ignored the fact that the design curves have already taken such a reduction into account.

We agree with Sewell when he referred to the design clauses of N.A.A.S.R.A. as a " mathematical fiction ". The curves are useful for design but are not to be thought of in terms of particular patterns of traffic.

The calculations of FF & P for the live load stresses in the box girder were not presented in a form such that we have been able to determine what, if any, liberties have been used in assessing the intensity of traffic loads.

Calculations for the column reactions, however, show that in that case the live loads used bear little if any relation to the values which would have been obtained if FF & P had been using their own loading curve. For example the live load reaction on columns 11 and 14 is stated by FF & P to be 280 tons and this was also the value they gave to Maunsell for the design of the concrete columns, piers 11 and 14. It was, however, demonstrated to us by Sewell that the correct value of this reaction using the FF & P load curves and including the knife-edge load as required by N.A.A.S.R.A. was 845 tons. G. Maunsell in their report obtained an almost identical answer. We have conducted sufficient analysis of our own to satisfy ourselves that the value of 845 tons is about right ; indeed no serious challenge to its accuracy was made by FF & P during cross-examination.

The very large discrepancy between 280 tons and 845 tons requires a great deal of explanation. If the 280 tons were correct it would mean that the equivalent number of loaded lanes was down to under one and a half.

Roberts, when questioned on the live load reaction for pier 11, at first tried to argue that there was an error of transcription on to the relevant data sheet given to Maunsell. When this was shown to be incorrect, Roberts admitted that 280 tons was an error, and as a result of some calculations he did overnight, came up with the value 407 tons as the correct live load reaction. The Commission was told that these calculations of Roberts would be made available for their examination. Despite repeated requests and repeated promises, they were never produced. The extensive nature of the calculations by Sewell shows clearly that a thorough re-assessment of the live load reaction could not have been done " overnight " even if the relevant data was available to Roberts. The value of just over 400 tons would be consistent with neglecting altogether the knife-edge load and then arbitrarily dividing the rest by two, but we have no means of knowing what Roberts did. He claims that any result is a matter of engineering judgment, and on this he is probably right. We can only disagree with his judgment when he cuts the live load reaction to about one-half the value it should have if calculated according to the very rules which FF & P had told the Authority they intended to apply.

The origin of the erroneous reaction of 280 tons can in fact be traced in the FF & P calculations. Early in 1967, Maunsell were anxious to get reaction values so that they could get on with the design of the concrete piers. A calculation done by FF & P in April of that year, sets out quite clearly that the live load reactions were assessed by simple proportion from the dead load reactions. The ratio used, being the ratio of the loads per foot for live load and dead load. This extraordinarily crude device gave a value for pier 11 reaction of 240 tons, which, as Roberts said in evidence " was rounded up to 280 tons ". Roberts went on to say that he thought the assumption made " could not be justified ".

We fully agree that the assumption could not be justified ; as now demonstrated, it gave an error of 200 per cent. It should have been obvious to any responsible designer that the assumption could only lead to a gross under-estimation of the required reaction.

Roberts added that the error, which he claimed was only 127 tons (407 less 280), was not serious because an error of about the same amount, but in the opposite sense, had been found in the dead load reaction.

Roberts also stated in evidence that the live load reactions in question were preliminary only and that this was understood by all concerned. While this may be so, there is no evidence in the calculations produced by FF & P that the 1967 estimates of live load reactions were ever revised and the values obtained at that time appear to have been in current use by both FF & P and Maunsell right up to the time of the failure.

The comments above centre round the assessment of the live load reaction on pier 11. This case was chosen because it was dealt with in great detail in the evidence. There is no reason to suppose that the admitted errors and exercises of judgment noted in that case are an isolated phenomenon.

As far as the piers and their foundations are concerned the reactions would be subject to some small modification owing to the different impact factors which apply, but in any case it is almost certain that the errors could be absorbed without causing unacceptable over-stress.

So far as the steel diaphragms over the piers are concerned the matter is more serious. In saying this, we are conscious of the finding that the Milford Haven disaster was primarily the result of inadequacies in the diaphragm over a pier.

On the most up-to-date information available to us it appears that the correct values of the reaction at pier 11 are 1,200 tons dead load and 845 tons live load, a total of 2,045 tons. Even after the very substantial strengthening of this diaphragm in August, 1970, the design reaction for the service condition was stated to be only 1,700 tons.

In summary we are satisfied that the dead loads were assessed by the designers with sufficient care and accuracy. We cannot say the same about FF & P's treatment of live load.

PART 4 THE CALCULATIONS.

4.1.1. GENERAL.

In this section only the calculations made by the designers FF & P and the contractors WSC are considered, although there are references to other calculations. After the collapse a great body of calculation was undertaken by a number of investigators ; this work is reported separately in Part 5.

For simplicity the contractors' calculations are attributed to WSC whereas they were for the most part made by Werkspoor, WSC's parent body in Holland.

The chronological sequence of events in the designs for West Gate Bridge is set out in diagrammatic form in Fig. 14. The diagram is intended to convey a picture of the relative timing of the various stages of design and to give a rough idea of the intensity of design effort at the various stages. The ordinates on the diagram are not intended to represent any scaled quantities.

Reconstruction of the time sequence has been made difficult by the fact that neither FF & P nor WSC made a regular practice of either dating or signing calculations. There are however sufficient dates either quoted, or to be inferred from relevant correspondence, for the construction of Fig. 14 which we believe gives a correct over-all picture.

The earliest dated calculation sheet in the FF & P files made available to the Commission bears the date January, 1966, but very little work was done for the first six months. Brown, when discussing the design process stated—" this bridge was a new development. The design remained in a complete state of flux for some considerable time . . . . because this type of bridge involves new concepts and calls for much thought to arrive at acceptable practical solutions ". We find this attitude of Brown hard to accept because FF & P had a great deal of experience in box girder bridges, including those with trapezoidal sections and with cable stays. Freeman stated that the West Gate Bridge was very near to the same design as the Wye Bridge, on which Brown claims to have worked as principal bridge designer.

It may have been that FF & P were overcommitted at that time, in the design of many other large bridges. The fact remains that for twelve months very little real progress was made. As late as April, 1967, the shape of the cross-section appears still to have been undecided.

Roberts agreed that the actual tender design was completed in the three months May, June and July, 1967, and as he said, " it was very quick going ". We feel that it may have been too " quick going " and it is possible that a good deal of the subsequent troubles stem from inadequate consideration at the time of the tender design. Brown agreed that the presentation of the calculations was not up to the standard of other comparable FF & P jobs. He said that " the others would be in a little more detail and better collected together "—he explains this —

" Because it was developed fairly quickly towards the end . . . "

FF & P have stated that the tender design was preliminary only and that they would have expected the contractor to introduce changes when he was preparing working drawings. They have tried to convey that the design was only in rough outline. Examination of the tender drawings shows that this is not so. In Fig. 9 which is a reproduction from a typical tender drawing, it will be seen that design details are fully worked out, down to the exact location of every hole. The contractor could be excused if he came to the conclusion when tendering that proper care and attention had been given to the preparation of the design and detailing of the work.

The work went to tender in October, 1967, and six tenders were received. Brown was in Melbourne in February, 1968, to help in assessing the tenders, and spent eleven days on the work, most of the time being concentrated on the two most likely tenders. Brown claims that in this eleven days he personally conducted some detailed mathematical calculations to test the validity of the erection scheme proposed by WSC but his calculations were not preserved. We find it impossible to accept, that if Brown was giving proper attention to the complex problem of the relative merits of the various tenders, he could have engaged in the detail calculation work he says he did, and given adequate consideration to the erection schemes proposed.

Reverting to Fig. 14 it will be seen that WSC's first burst of activity was at the end of 1967, when preparing their tender. On 27th February, 1968, they received a " letter of intent " followed on 9th April by a " notice of acceptance ". The formal contract with WSC was signed on 17th July, 1968.

WSC recommenced work on receipt of the letter of intent and as early as 12th March, held a meeting with FF & P to discuss, among other matters, the problems of design.

At this meeting FF & P agreed to hand over to the contractors " a set of bridge design calculations " in order to " ensure that erection calculations will be in accordance with the engineer's design criteria ". Despite many requests by WSC these calculations were never handed over. Roberts in evidence, said that it was not possible to comply with such a request because no calculations existed at that time which were in a form that could be handed over.

WSC attempted to get ahead with their own stress calculations for checking the safety of the proposed method of erection. It was soon obvious to them that by the Dutch and German codes, with which they were familiar, the bridge would be overstressed not only in the erection conditions but also in the service condition. They naturally assumed that FF & P had some superior knowledge which enabled them to use significantly higher design stresses and indeed Brown hinted as much when he talked to WSC in vague terms about post buckling design.

WSC under these conditions pressed even harder for FF & P's calculations or at least some guide on allowable stresses for compression plates. Their repeated requests met with no success. Brown says he did not take the requests seriously because he thought Hardenberg wanted the information for his own private purposes. WSC persisted in seeking the information on stresses until as late as August, 1969.

The only sop which Brown threw to Hardenberg was a set of formulae, based on the work of Chapman and Falconer, ([12], [13]) which by implication, FF & P were using to get critical stresses. This information was unaccompanied by any instruction as to how it was to be used and in the event Hardenberg did not use it the way FF & P had intended. In any case these formulae related only to the critical buckling stress and Hardenberg was no nearer to finding out what FF & P were using for allowable stresses.

In consequence of FF & P's failure to co-operate in the matter of design stresses, WSC got off to a slow start. Eventually despairing of getting any more help from FF & P they went ahead and completed their calculations as best they could. During the period from mid–1968 until well into 1969 WSC were continuously seeking information from FF & P and asking for approval of work already done. The communication was very much one way ; letters and telex messages from WSC often remained unanswered, or when answered the information sought was often not given. During this period WSC discovered a number of deficiencies which they claimed were such that the bridge could not perform its service function ; a glaring example was the bottom flange in box 8. This was shown on the tender drawings as $\frac{1}{2}$–in. thick HYS. As early as May, 1968, WSC informed FF & P that they considered this thickness should be increased to $\frac{3}{4}$ inch and this was not challenged by FF & P. Indeed they claimed in evidence that they had made the same discovery at the same time that it was made by WSC, although there is no supporting evidence to show that they were doing any calculations at that time which would have enabled them to make such a discovery. Brown, in his evidence, went further and claimed that it was they, FF & P who had required the increase of the plate to $\frac{3}{4}$ inch. By August, 1968, WSC had found that even $\frac{3}{4}$ inch was not enough and suggested the increase to $\frac{7}{8}$ inch. Brown stated " I did not think this necessary. However, to avoid further delay we agreed with Mr. Hardenberg of an increase to $\frac{7}{8}$ inch ".

Almost a year later WSC were still very unhappy about the stresses in this particular panel. On 2nd July, 1969, Hardenberg telexed Brown pointing out that according to their latest calculations the actual stresses were considerably in excess of the allowable stresses and followed this by a letter of the same date pointing out that it was not the erection conditions which would cause this overstressing.

Brown on the next day, telexed a reply, which showed that he had given little if any thought to the matter. He said . . . " previous experience would indicate condition satisfactory but marginal. To eliminate any doubt simply increase height of transverse stiffener by 6 inches . . . "

Hardenberg on 4th July telexed back, not unnaturally, wondering if Brown had in fact received his telex of 2nd July. Hardenberg went on to point out to Brown that his recommended cure would in no way meet the deficiency and suggested tactfully to Brown that he had not perhaps grasped the significance of what WSC were saying.

This sequence exemplifies the difficulties under which WSC were labouring and the failure of FF & P to take Hardenberg's warnings seriously.

That Hardenberg was more than justified in his concern over the stresses in the bottom flange in box 8 is shown by the massive stiffening which was added in that area after the Milford Haven collapse and the conclusion by Maunsell, London, that even as then stiffened, the panel would still be overstressed for both the erection and the service condition.

While WSC were working at stress calculations during erection, FF & P had begun a stress check themselves. This was started as early as January, 1968, even before tenders had been received.

A proposed contents sheet for the check was drawn up which, in itself, was excellent. After setting out the section properties and the co-ordinates of the girder as preliminaries to a computer run the work appears to have been set on one side for six months.

It was picked up again in July, 1968, and some more hand calculations were made, using moment distribution. These appear to have been aimed at finding dead load reactions.

It should be emphasized that this FF & P check was only on the stresses in the finished bridge and not a check of the stresses at the various stages of erection. It was not therefore a check of WSC's erection calculations, which because of various delays including those referred to above, were not submitted to FF & P in any quantity until the second half of 1969, by which time fabrication of the boxes was under way and erection beginning.

It is hard to believe that FF & P made any serious check of WSC calculations. If they had done so they could not have failed to comment on Hardenberg's method of applying the critical stress formulae they had provided. Because Hardenberg's method of using these formulae was so very different from their own it would have been essential in any serious check to have set down many sheets of check calculations. No such sheets have been produced and we have been given no evidence that they ever existed. The only check calculations relating to stresses in the box girder, which have been produced are four sheets purporting to check the WSC calculations for the $\frac{7}{8}$-in. plate in the bottom flange of box 8. These four sheets contain arithmetical errors such that the conclusions FF & P may have drawn as to WSC's calculations are of doubtful validity.

As late as January, 1970, WSC were still submitting erection stress calculations to FF & P, though by this time span 14–15 was up in the air. The FF & P reply to this last set of calculations was to acknowledge receipt and go on to say " we are in reasonable agreement with your stress values ". How they could have come to this conclusion we fail to understand. Only two weeks had elapsed since receiving WSC's calculations and any proper check within that time would have necessitated a computer run, no evidence of which exists. It is small wonder that FF & P's reply is worded in such general terms. These latter calculations by WSC include the complete sequence of erection stages including the best they were able to put forward for placing deck concrete. Originally it had been hoped to place the concrete progressively as the steel erection proceeded but, according to WSC, this was not possible because of the overstressing which would result. Their final submission of January, 1970, envisaged delaying any concrete work until the steel boxes were almost all erected.

FF & P's design activity on the West Gate project was at a very low level from early 1969 until April, 1970, a vital period, when in our view, they should have been checking the final design of the structure. This was particularly so because their own checks at the end of 1968, when the second computer run was put through, showed stresses in some areas considerably higher than permitted by any published code. Brown under examination refused to acknowledge that there were regions of overstress and claimed that he was satisfied no overstressing would occur.

By early 1970, both FF & P and WSC appear to have ceased work on calculations for the stresses in the bridge, either in service or during erection. Construction, meanwhile, was going ahead in disregard of the unsatisfactory state of the calculations.

In March, 1970, Wilson, troubled by Hardenberg's repeated comments on overstressing, asked Kerensky to have another look at the stresses throughout the structure. Kerensky says that he had work started on this check in April, 1970, but there is no evidence that the work was given any priority and if check calculations were done between April and July, 1970, FF & P were not able to produce them to the Commission.

At the same time, March, 1970, the change-over of contractors had taken place. The Joint Consulting Engineers were anxious that every effort should be made to pick up time lost and one way that this might be done was to improve on the deck concreting programme proposed by WSC.

Normally it would have been the task of the new contractor, JHC, to have evolved a workable concreting sequence and to have submitted it for approval to FF & P as indeed had been done only a few months earlier by WSC.

It seems to have been generally accepted by all parties that JHC were not expected to make calculations on the stresses in the bridge itself. At all events they made no effort to work out a concreting sequence based on any analysis of the stresses likely to be set up. They did have Hardenberg on site as technical adviser and a logical step would have been to have sought his advice. Hardenberg had, of course, been unable to solve the problem of a more rapid concrete programme and indeed had come to the conclusion that no rapid solution was possible if safety margins were to be preserved. Had JHC sought his help they might not have come to a solution of the concreting problem but they would have gained a better understanding of the issues involved.

As it was, both JHC and Hindshaw put forward separate proposals for concreting, though neither appear to have been based on any study of the stresses set up. Surprisingly on 22nd June, 1970, FF & P gave tentative approval for Hindshaw to go ahead with his scheme although they had not checked it. The Milford Haven Bridge failure had occurred only three weeks earlier but FF & P were apparently not yet aware of the critical stress situation at West Gate.

When Hindshaw's scheme and that of JHC were examined later by FF & P in detail, it was found that neither was workable on account of over-stressing. From that time on, FF & P quite openly accepted the situation that if a workable scheme was to be found for an earlier placement of concrete, they themselves would have to find it and do the necessary analysis to demonstrate that the scheme was safe at all stages.

The failure at Milford Haven caused FF & P to take the overall stress check far more seriously, so that the double task of finding a better concrete sequence and checking the whole design caused an enormous resurgence of design effort on the part of FF & P. This new work built up in July, 1970, and was still going vigorously when the bridge collapsed in October. Three series of computer runs were put through in rapid succession. These used a more sophisticated programme and for the first time considered the erection realistically stage by stage.

A curious situation appears to have arisen from July, 1970, onwards as to who was in technical charge of the calculations. Brown states that he was no longer involved, the work being entrusted to other more conservative engineers. Kerensky, who by then had been nominated to the Authority as the Partner of FF & P in charge of the West Gate project, says that Brown was still in technical charge of calculations. Kerensky himself was not directing the work and Roberts had retired, so that if Brown says it was not under his control, we are puzzled to know who was directing the work at that vital time.

Also in mid 1970, some check calculations were done by Wallace, a Maunsell engineer, but at that time seconded to JHC. These calculations, simple as they were, drew attention to significant overstressing in the box girder when cantilevering out boxes 9 to 12. FF & P rejected Wallace's conclusions, as it turns out without justification, on the grounds that the problem was far more complex than he had assumed. It is true that Wallace had not considered all the possibilities, but the one he had considered was the one which mattered. Once again FF & P pushed on the site work in spite of the warning.

The final stages in calculations were those undertaken by FF & P after 15th October, 1970. These, as presented to the Commission, are intended to show that the stresses would not become critical except when bolts were removed on the 4–5 splice. These calculations make no claim that the stresses actually imposed were within the approved safety margins. It must have been obvious to FF & P when submitting these post-failure calculations that they would be subject to close scrutiny. It is astonishing therefore, to find in them so many errors both of arithmetic and of basic principal.

4.1.2. Calculations for the Tender Design.

Only those aspects of the tender design which relate to spans 10–11 and 14–15 are considered in this report and in general the discussion is limited to the design of the box girder to resist longitudinal bending.

Almost the only calculations which FF & P produced to the Commission which relate directly to the tender design were the computer outputs made at that time (early 1967). Neither Roberts nor Brown would admit to a knowledge of the details of the computer programme employed. It appears, however, that in drawing up the programme, sweeping simplifications were made when representing the structure. For example, the flexural rigidity of span 10–11 has been assumed constant throughout its length when, in fact, there exist large differences of rigidity from box to box. Other spans have been treated in the same way. Such an assumption would lead inevitably to an underestimate of the negative moment over pier 11 and this is no doubt the origin of the deficiency of strength in box 8.

Roberts in evidence stated that he knew that the computer programme used for the tender design was not satisfactory. We consider that this placed him under an obligation to warn the successful tenderer and also to proceed without undue delay in making a proper design check, but neither of these steps appears to have been followed.

The computer outputs, on which the tender design was based, stop at the stage of giving forces and bending moments throughout the structure. The calculations which relate these forces and moments to actual stresses, and the calculations of the appropriate values of allowable stresses are all missing. These missing calculations are the very ones which Hardenberg wanted so much to see, and which Roberts said did not exist in a form which could be handed over.

Brown was unable to explain the absence of calculation sheets relating to allowable stresses, but went on to add that they would know the right values from previous jobs. He said that the design stress was " in the order 12 tons per sq. in." without making any qualification as to where that might apply.

We are disturbed by these missing calculations, particularly when it is the same group of calculations relating actual to allowable stresses, which is again missing for the next analysis done at the end of 1968. FF & P have assured the Commission, on more than one occasion, that they have handed over all relevant information in their possession and they were specifically asked to search for these particular calculations.

We can think of only three explanations—

(i) that the calculations were never done,

(ii) that the calculations were done and have been lost,

(iii) that the calculations were done, but have been suppressed.

We cannot take seriously Kerensky's suggestion that the engineer who made them may have taken the calculations with him on leaving the service of FF & P. Consulting engineers' offices do not, or should not, work that way.

If the calculations were never done, which would be consistent with what Roberts said, then the tender design was guesswork and not the result of proper stress analysis. But if that were so, the very extensive computer calculations would appear largely pointless.

That the calculations were lost is a simple explanation, but it is hard to accept the coincidence that the same group of calculations was also lost 20 months later. If the calculations had been merely lost it is hard to account for FF & P's extreme reluctance to tell Hardenberg the basis on which they had been made, particularly the basis for the determination of allowable stresses.

Before one could accept the suggestion that the calculations may have been suppressed there would have to be some reason why FF & P would want to do so. This whole area of the relation between actual stresses and the allowable stresses used by FF & P is unsatisfactory. An elementary check using any orthodox values for allowable stresses shows that Brown must have been using significantly higher values. It may be that FF & P were unwilling to open their calculations to scrutiny and criticism, either by Hardenberg or by this Commission.

4.1.3. Post Tender Calculations by FF & P.

The computer runs for the post-tender check by FF & P were put through in the period November, 1968, to January, 1969. The programme used allowed for the variation in section properties from box to box in the several spans.

The dead load stress analysis was based on four steps—

(1) The dead load of the steel boxes themselves, carried by the steel sections only.

(2) Half the total load of the concrete deck considered as deposited instantaneously over the whole 2,782 ft. length of the bridge, the load being carried by the steel sections only.

(3) The remaining half of the concrete deck, again assumed instantaneously deposited and this time assumed to have attained full strength at the moment of deposition, the load being carried by the composite sections of reinforced concrete deck plus steel boxes.

(4) The finishing items, crash barrier, guard rails, blacktop, etc. carried on the fully composite sections.

The assumption of instantaneous placing of concrete over the full length in items (2) and (3) and the assumption of instantaneous hardening of concrete in item (3) are over simplifications which would tend to lead to an underestimate of the maximum stresses in the structure.

Towards the end of this computer series the programme has been extended to include the conversion of moments and forces into stresses at the upper and lower flanges. Elsewhere the collation of information on the print outs is well tabulated in the calculation sheets so that the analysis of stress at the various parts of the structure is shown clearly. Finally a large sheet summarising all the stress figures was drawn up (Ex. 193) and appears to have been the only information on stresses available to the resident engineer on site until more detailed values arrived in mid-September, 1970, only a month before the disaster.

This master sheet on stresses is dated January, 1969, and is, presumably, drawn up from the earlier dated tables set out in the calculations. There are some curious discrepancies between the figures on the master sheet and the corresponding figures taken from the calculations. Some of these discrepancies are obvious mistakes in arithmetic or in transcription, others appear to be the result of a deliberate modification to certain high values of stress when transferring them from the calculation sheets on to the master sheet.

Typical of the first type of error is the entry on the master sheet for the stress at the 9–10 splice. For the lower flange the compression stress due to bending is given as 7.46 tons per sq. in. and the axial compression stress is given as 3.14 tons per sq. in. The total is quoted as 5.70 tons per sq. in. instead of 10.60 tons per sq. in. There are a number of errors of this type.

An example of the second type of discrepancy is in the column in the master sheet dealing with the stress in box 8. The calculations sheets clearly set out the various items of live and dead load which make up a total of 13.69 tons per sq. in. for the highest stressed point in the lower flange. In transcribing the figures on to the master sheet the bending moments have been copied without amendment but all the bending stresses have been reduced, roughly by 10 per cent. In consequence the stress total quoted on the master sheet is only 12.35 tons per sq. in. No explanation is offered to account for this procedure and it is interesting to note that on an abbreviated summary sheet of stresses, sent to the Authority in November, 1968, the figure quoted for the same point was 13.2 tons per sq. in.

There were no significant alterations to the structural sections in the area, made during the period November, 1968, to January, 1969, so that the only reason why the stresses could have been reduced by about 10 per cent. is on account of a reappraisal of the section moduli. On the other hand the master sheet sets out values for the section moduli and these are the same as those used in the main calculations.

The stresses in the bottom flange of box 8 were the same ones which so much concerned Hardenberg and led him to increase the plate thickness. The FF & P calculations were based on Hardenberg's increased plate thickness so that the correct value for this very high stress was of great importance. When FF & P conducted a much more detailed analysis after the Milford Haven failure, the corresponding stress was calculated to be 13·46 tons per sq. in. so that it appears that the arbitrary reduction by 10 per cent. was unjustified.

We can only conclude that the values set down on the master sheet were not entirley satisfactory and, at least so far as box 8 was concerned, likely to mislead.

Calculations in which allowable stresses were worked out for the various situations have not been produced by FF & P so that it is difficult to see at a glance whether the theoretical stresses are within allowable limits or not.

If we examine the critical buckling stresses for the lower flange on box 8 it is evident that the lowest buckling mode is that for the buckling of the longitudinally stiffened panels between transverse beams. FF & P's analysis confirms that this is indeed the case.

The critical stress for that mode calculated according to the Chapman & Falconer formula, is 18·3 tons per sq. in., only slightly higher than the Euler value for one longitudinal stiffener and its attached piece of plate, which is 17·7 tons per sq. in.

It is clear that whether the stress in the panel is 12·35 tons per sq. in. as given in the master sheet, or 13·69 tons per sq. in. as in calculations, the safety factor, even against elastic buckling, was considerably less than the stated value of 1·7 for the service condition.

For the mode of buckling concerned, and for the relevant slenderness ratio, failure would occur at a stress well below the elastic crippling stress. This is so because of the imperfections of flatness inherent in any practical panel and the limits imposed by the onset of yield.

Using the compression stress formula in BS153, with the appropriate slenderness ratio, gives an allowable stress of 7·35 tons per sq. in. The slenderness ratio used in that determination is based on the half wave length of buckling being the distance between transverse ribs. This length is the same as that used by FF & P in applying the Chapman & Falconer formula and is the length which witnesses Roderick, Murray, Richmond and Grassl all considered as the right one to use. Brown on the other hand was prepared to use $0 \cdot 7l$ as the effective length. He based this reduction on his belief of how the panels of plating would behave if they were initially all dished slightly towards the stiffeners, i.e. slightly upwards on the bottom and downwards on the top panels. We think that Brown's logic is of doubtful validity even if the panels were dished as he assumed. When, as we know, there was no reliable pattern in the initial distortion of panels, Brown's assumption of $0 \cdot 7l$ is, in our view, unjustified and likely to lead to values of allowable stresses which are too high.

As discussed in Section 4.1.4. we do not entirely dismiss the possibility that there may be some degree of end fixity for the panels, but this would be for a totally different reason than the one suggested by Brown. In the present state of knowledge we think it unwise to use other than the full length when designing such panels.

It is interesting to note that even using the value of $0 \cdot 7l$ as the effective length the allowable stress calculated by BS153 comes out at only 10·1 tons per sq. in. Brown stated that he was in the habit of adding 10 per cent. to the BS153 values on the grounds that he assumed initial unfairness to be only 2/3 of that used in the B.S. formula. We do not think he was justified in doing this, particularly as he made no measurements to check the validity of his assumption. Even with the addition of the extra 10 per cent. the allowable stress, still using the $0 \cdot 7l$ basis, only comes up to 11.1 tons per sq. in., and is thus less than the 12·35 tons per sq. in. on the master sheet and considerably less than the 13·69 tons per sq. in. on the calculation sheets. So that even by Brown's own derivation of allowable values, the stress level in the lower flange of box 8 was still too high. Brown's final attempt at justification was to state that the stress in the panel was changing over the length between transverse ribs so that the peak stress should not be used when considering buckling. Diagrams of bending moment were, however, drawn on the master diagram and these show that the change in stress over the relevant 10 ft. 6 in. panel was not large and that even if the stress at mid-panel length were taken as the criterion, the value is still higher than the allowable value even with all Brown's modifications included.

It was several times claimed by FF & P witnesses that figures such as those presented above were invalid because they were based on BS153 and that specification does not apply to spans greater than 300 feet and does not specifically mention box girders, or continuous spans. This is all very well, but FF & P had announced to WSC and to the Authority that the stresses would, in general, be in accordance with BS153 and in the few design calculations on allowable stresses that have survived there is no doubt at all that they themselves were using that code for allowable stresses in the box girder.

When asked to tell the Commission what was the basis of their allowable stress formula, if it was not BS153, FF & P were unable or unwilling to be specific. A number of methods were mentioned but none of these would have justified stresses as high as those calculated for box 8.

Brown when questioned closely on the high stress figures appearing in the master sheet persistently maintained that there was no overstressing. We conclude that he had become somewhat detached from the realities of the situation and was designing by intuition, in disregard of the elaborate mathematical analyses available to him. Indeed he admitted in evidence that his decision about box 8 was " partly intuitive ". We deplore Brown's attitude to the design of this structure and consider that it generated a potentially dangerous situation. What makes it even harder to understand, or to excuse, is the fact that Brown virtually disregarded Hardenberg's repeated warnings that sections of the bridge were overstressed for the service condition. That Hardenberg was right was abundantly demonstrated both by Maunsell, London and by FF & P's own designers when the whole structure was rechecked after the Milford Haven collapse.

4.1.4. Buckling of Stiffened Panels in Compression.

Before going on to the next stage of design sequence, it is desirable to intrude some notes on the compression buckling of stiffened panels and the way in which this was dealt with by FF & P.

The largest panels on the West Gate Bridge which were considered, were the stiffened panels bounded longitudinally by transverse diaphragms and transversely by the longitudinal webs. These panels were 52 ft. 6 in. long and either about 20 feet, or in the case of the outer-upper panel, about 30 feet wide.

The classical theory of elasticity gives three distinct modes of instability for such panels when subjected to longitudinal compression—

(i) The smallest sub-divided panel of plate only may buckle, without buckling of either the transverse ribs or the longitudinal stiffeners which bound the edges of this sub-panel. The solution of this mode of buckling was first obtained by Bryan[10] and later developed by Timoshenko[11].

(ii) The longitudinally stiffened panels may buckle between transverse ribs, which themselves remain rigid. This form of buckling could develop across the whole width of the panel. For the West Gate Bridge, the influence of the longitudinal webs in restraining the panel at its sides would be small so that for this mode the whole panel would buckle in almost the same way as one constituent typical strip, composed of one longitudinal stiffener and its associated width of plate. The Euler formula applied to the typical strip will give a nearly correct value for the critical stress for the panel. The value obtained will be slightly on the conservative side, owing to neglect of edge restraints.

(iii) The whole panel may buckle, restrained only at its perimeter by the transverse diaphragms and longitudinal webs. This is a complex mode and the full solution is generally impractical for normal design purposes. FF & P used formulae developed by Chapman and Falconer[12] for the analysis of this type of buckling. In using the Chapman and Falconer equations, FF & P assumed that the initial unfairness terms were zero.

In general for the West Gate Bridge it was necessary to calculate the critical stress (i.e., that at which elastic instability would occur) for all three modes of buckling. Only the lowest critical is of importance since this will determine the limiting stress which can be applied, irrespective of how much higher the other two critical stresses may be.

In theory if Mode (i) is the lowest of the three critical stresses, it is possible to go slightly into the post buckling range. For the conditions applying at West Gate, however, in which the area of the section of the longitudinal stiffeners was small compared with the area of flange plate, the onset of Mode (ii) buckling follows so closely after the Mode (i) that no significant post-buckling reserve exists. While FF & P talk about post-buckling design they did not in fact use it for the flange panels of the box girder.

When FF & P gave a version of the Chapman formula to WSC the inference was that they themselves were using it. There is no evidence in the calculations that FF & P did in fact use this method for either the tender design or the post-tender check. The first time the formula appears in any FF & P calculations submitted to us is when they were checking the use of the formula by WSC at some undated time in the second half of 1969. On this occasion the formula was used only for Mode (iii) buckling.

Later in 1970, when the much more extensive checks were made FF & P used a variant of the Chapman formula which enabled them to find critical stresses for all three modes. The version of the formula they had given to WSC could not have been used in this way and would have given serious errors if used for Mode (ii). It could however, have been worked for Modes (i) and (iii).

It must be emphasized that all that these formulae give is the elastic critical stress. They do not give allowable working stresses.

The elastic critical stress may be of some help in assessing an appropriate allowable stress, but other factors such as the yield stress, the initial unfairness of the panels and the appropriate safety factor must all be taken into account.

It is pointed out that the above notes refer to the way in which panel buckling was handled by FF & P and WSC. We do not necessarily agree with what was done.

We note that no account was taken on the effect of locked in stresses in the plates due to rolling and welding. Recent research has tended to show that such locked in stresses can have a major influence on the crippling load for the Mode (i) buckling, to the extent that the classical Timoshenko type formulae can be misleading. The influence of locked in rolling and welding stresses on buckling of Modes (ii) and (iii) will be relatively unimportant.

For Mode (ii) buckling the formulae used by FF & P will give realistic values for the elastic buckling stress, irrespective of whether the panel buckles upwards or downwards. If however, the failure condition is defined as the onset of yield at some part of the section, then the direction of buckle will matter. For example an upper flange panel which buckles upwards will fail when the outstanding tip of the stiffener yields, but when the same panel buckles downwards the curvature due to bending tends to cause tension in the tip of the stiffener so that failure will not occur until later, when either the plate itself reaches yield in compression or the stiffener tip reaches yield in tension. In this way it is possible that, as failure approaches, the convex upwards panels of the upper flange might receive some degree of end fixity from the convex downwards panels. Experimental evidence on this point is lacking and it is probably safer in design to ignore it, particularly as there is no such effect on the elastic crippling stress. The same arguments might be applied when considering the buckling of a panel containing a splice. The obvious weakness in the splice detail will cause a premature failure of such a panel, at a load for which the adjacent panels are still stable and thereby able to give some degree of end fixity to the spliced panel. Any allowance for this effect must be somewhat conjectural owing to the uncertain behaviour of the splice itself. In the design, however, FF & P completely ignored the presence of the splice in so far as it might affect stability of the panels.

4.1.5. WSC Calculations for Erection Stresses.

In the early stages of calculation WSC, or more properly, Werkspoor, used the Dutch and German codes with which they were familiar, as well as BS153. They found at an early date that none of these codes could explain the high stress values used by FF & P.

Persistent requests for some guide on FF & P's basic stresses produced the version of the Chapman formulae referred to above. The formulae were rather roughly written out and included one error of sign which appears to have been corrected later. In addition to the basic formula a simplified form was given although the approximations made in deriving this simplified form appear unjustified and unnecessary. For example, a factor which should be 10·9 had been taken as 10·0 thereby introducing a quite unnecessary error of 9 per cent. Also given was a formula for calculating the desirable stiffness of the longitudinal stiffeners.

Hardenberg stated that no accompanying instructions on how to use the formulae were sent. On their face value they were only intended for Mode (iii) buckling and that is all Hardenberg used them for. This still left WSC with only the orthodox formulae for dealing with Modes (i) and (ii).

For Mode (i) Hardenberg used the Timoshenko values for the classical solution. He commented on the calculation sheets that for this type of buckling the factor of safety on critical stress ordinarily used was 1·35 for the service conditions. He proposed to lower this to 1·25 for erection conditions.

For Mode (iii) Hardenberg had the Chapman formulae but he was still uncertain what safety factor had been used by FF & P. He could get little co-operation from Brown and had to make the most of the information he had been given. He assumed that the formula he had been given for finding the size of longitudinal stiffeners was such that a panel with stiffeners so designed would have equal failure loads in buckling and overall yielding, using the appropriate safety factors in each case. This assumption was invalid, but he went ahead and by some ingenious manipulation of the formulae managed to convince himself that the formula he had been given for σcr implied a constant safety factor, for the service condition, of 2·6 against elastic buckling, and therefore a factor of 2·0 for the erection conditions.

In the calculations presented by WSC for checking by FF & P allowable stresses have been set out on the above basis. No specific mention is made of Mode (ii), although in the majority of cases that is the mode which controls. Hardenberg says that he kept a private check on Mode (ii) failure using the Dutch codes but he did not apparently submit these checks to FF & P for approval.

The values for allowable stress which WSC obtained were without doubt low and this was part of the reason for Hardenberg's anxiety about what he thought was overstressing in the FF & P design. Compared with BS153 values, however, his allowable stresses were not unreasonable. For example, in the bottom flange of box 8, dealt with in the previous section, Hardenberg got an allowable stress for the service condition of 6·9 tons per sq. in. compared with 7·35 tons per sq. in. given by BS153.

We deplore the actions on the part of FF & P which brought about this failure to co-operate on basic design stresses. Frank discussions at an early stage would have enabled Hardenberg to criticise the FF & P rules on a basis of equality. As it was he was kept in subservience by being kept in ignorance. It is just possible that Hardenberg, whom we regard as a highly qualified designer, might have been able to persuade FF & P to have another look at their own methods.

When WSC's calculations were submitted to FF & P it was obvious in the circumstances that a thorough and completely independent check was desirable. The only evidence of any check on the stresses in the box girder are four pages of calculations ; if other check calculations exist they have not been produced. The four sheets that do exist set out to check the critical buckling stress of only one panel, which happens to be the one discussed earlier, i.e. the bottom flange of box 8. First the full Chapman formula is used to get the critical buckling stress for Mode (iii), next the calculation is repeated using the simplified Chapman formula and finally the allowable stress is calculated by the BS153 formula, using $0 \cdot 7l$ as the effective length but not including the extra 10 per cent. which Brown said he was in the habit of adding. The calculations are marred by a number of arithmetical errors in calculating the moments of inertia for the longitudinal and transverse ribs. Curiously although the same values for these quantities are required in both the full and the simplified version of the Chapman formula, they have been recalculated for the second occasion. The first and second values for both quantities do not agree, in one case by over 50 per cent. We find it hard to understand why a designer getting such obvious differences did not at once check to find the source of his error. FF & P's values for σcr, both in error by about 10 per cent, straddle the WSC value so that it is hard to see how they could have drawn any conclusion on the validity of Hardenberg's calculations.

The last sheet in this group is interesting because it shows clearly that FF & P were using BS153 for getting allowable stresses. The value they obtained for the bottom flange of box 8 was 10·1 tons per sq. in. which should have alerted them to the undesirably high working stresses for that element under service conditions. As pointed out earlier, FF & P's own calculations showed a theoretical working stress of 13·69 tons per sq. in. at the same point.

4.1.6. Calculations Following The Milford Haven Collapse.

After the Milford Haven collapse FF & P conducted far more thorough analysis of the erection and final stresses throughout the whole bridge. As pointed out earlier, this analysis was combined with the evolution of a workable sequence for placing deck concrete.

A much more sophisticated computer programme was used on this occasion although it is surprising to find that with all the experience FF & P had gained on other box girder bridges, including those with cable stays, it was necessary to develop a computer programme for West Gate stage by stage as the design evolved.

The new programme gave proper allowance for the variations in stiffness from box to box, it also allowed for the change in stiffness of the boxes as concrete was added and composite action developed. The print-out was normally in the form of stresses at the upper and lower flanges.

Altogether three separate series of runs were put through starting in July, 1970. The third series was still in progress when the bridge failed in October.

In the first of these three series the erection sequence was originally that proposed by JHC, but after this was shown to cause overstressing, a trial sequence proposed by FF & P was used but also proved unworkable. At that stage it was realized that a very difficult problem existed in that overstress had been revealed at a number of places and a workable concrete programme had not been achieved.

For a while FF & P turned to the possibility of using light weight concrete for the deck, and some computer runs were done on this possibility. It is disturbing to think that such a major change was even contemplated when the bridge was partly built. The difficulties of using light weight concrete and the fact that it did not solve all the problems led to an early abandonment of that idea.

The second and third series of computer runs returned to the use of normal concrete. In the second series a complete re-analysis of the live load effects was undertaken.

The results of these various computer runs were summarized in a volume of analysis sheets first prepared in September, 1970, but later revised and corrected in January, 1971. In this volume were given the separate stresses caused by each of about 40 steps in the erection process, covering all stages from the lifting of the first spans until the final concreting has been completed. Adding up of all the relevant steps was also done so as to get the final dead load stresses. Also given were the stresses caused by live loads.

Unfortunately, the analysis, even in its revised and corrected form, was not free from errors, some of which were purely errors of arithmetic, others were errors of principle.

An example of the former type is the apparent double inclusion of the operation of placing concrete on the cantilevers of boxes 11 to 17.

Typical of the latter type of error is the manner of dealing with composite action. In some cases it appears that the assumption has been made that concrete possesses fully developed strength, and rigidity, as soon as its weight is applied to the structure. In others it has apparently been assumed that the concrete can partake in resisting loads which were applied to the structure even before that particular concrete was placed.

The earlier version of this analysis summary was sent to Hindshaw on 18th September, 1970, and for the first time he had available reasonably reliable figures on stresses throughout the structure, for all stages of erection. In the letter which accompanies this analysis summary, FF & P pointed out to Hindshaw that they had found it necessary to reinforce certain of the bottom panels. The letter goes on :—

> " Something may also have to be done to stiffen the top flanges of some of the boxes in the main span to take compression before they are concreted over, and we will send you details as soon as we have worked them out."

It is true that the reference is to boxes in the main span, but it appears unlikely that at the time anyone had given much thought to the top flange panels at all. Ironically the top flange at box 4–5 north in span 10–11 was already buckled when the letter was written, but London never knew of this until after the collapse, four weeks later.

After the Milford Haven collapse in early June, Crossley, the deputy RE, began a limited but thorough examination of his own. In the absence of any stress data for the overall structure, he concentrated on the erection stresses caused by cantilevering boxes 9 to 12. This was a statically determinate problem and one which he could easily solve locally without the computer. It was also an obvious priority in checking since this was the operation which was being carried out at Milford Haven when that bridge failed.

Crossley soon found out that the stresses which would occur in parts of the bottom flanges when box 12 was added as a cantilever were very considerably in excess of any allowable stress he could suggest. This overstressing according to Crossley's analysis would occur under the self weights of the steel boxes only, long before any concrete was added. Furthermore, the cantilevering operation which would bring about the overstressing was already started.

Crossley's report was received in London on 6th July, 1970. It is a well set out document demonstrating clearly and incontrovertibly the extensive overstressing which would occur if the then current programme of erection was maintained.

Crossley was called to London to assist on the overall checking and stayed there for most of July and the first week of August.

By mid-July, when cantilevering had already progressed to box 10, the significance of Crossley's evaluation of erection stresses appears to have been recognized and with it the acceptance of the fact that there was no simple way out of the dilemma. Accordingly, on 22nd July, a stop order was issued to arrest all future erection of cantilever boxes until after massive additional stiffening had been added to the boxes at the points where overstress would otherwise have occurred. The details of this extra stiffening are given later.

The calculations in the Crossley report are interesting because they contain the first proper analysis by FF & P of critical buckling stresses for the stiffened panels ; even then Crossley only considered the bottom panels. After getting the critical stresses for elastic buckling, Crossley found himself in the same difficulty that Hardenberg had experienced earlier, namely the conversion of critical buckling stresses into allowable stresses. If FF & P in London had themselves any method of doing it, they had apparently not let Crossley into the secret. Crossley's first attempt to relate the two quantities was to assume that the relation between allowable stress and crippling stress for the stiffened panels was the same as that inferred by the strut formula in Table 4 of BS153. What he did was to calculate the Euler crippling stress for various values of slenderness ratio, then by extracting from BS153 the allowable stress for those slenderness ratios, he was able to plot a curve of allowable stress against critical buckling stress. Crossley assumed that the curve would apply to all three modes of crippling.

On the basis of the curve he had produced, Crossley showed that over-stressing during cantilevering would be caused at many of the panels in the lower flange. Some of the panels would be overstressed more than 50 per cent. beyond the allowable, even after the 30 per cent. increase for erection conditions had been added, but some were only slightly over-stressed. Crossley then tried to justify an increase in the allowable stress which, if it could be achieved would have reduced the number of panels needing modification. His first attempt to do this was by invoking Clause 28 of BS153, which relates to allowable bending stresses. The section of the clause which he used is restricted to beams in which the moment of inertia about the vertical axis is less than that about the horizontal axis, which was certainly not the case on the box girders. Clause 28 is intended to guard against lateral instability of the compression flange of a beam and this was simply not an issue at West Gate. Although Brown admitted to having seen the Crossley report he did not apparently raise any objections at the time to the improper use of Clause 28. Under examination before the Commission, however, he agreed that the table of figures in Clause 28 had " no relevance at all ".

Crossley used Table 8 of Clause 28 in the same manner that he had earlier used Table 4 and got another curve relating allowable stress to crippling stress. For high values of allowable stress the new curve gave an increase of up to 10 per cent. on the old values but for allowable stresses less than 9·5 tons per sq. in. it actually produced smaller values.

Crossley tried to get further advantage by manipulating the BS153 formula for effective width of plates in compression. Again his method was completely invalid and again, to the Commission, Brown stated that what Crossley had done " has no relevance whatsoever ".

Crossley, however, applied his second modification not to the original Table 4 values but to the already boosted values he had got by using Table 8. The result was that for high values of allowable stress, the new values were up to 20 per cent. greater than the original values. Crossley found, however, that even using this doubly increased allowable stress, the actual stresses were in places more than 40 per cent. higher still.

In London, the attempt to justify even higher allowable stresses continued. Brown vigorously denied that there had been any such attempt, although he equally vigorously said that he had had no part in the work after July, 1970. The evidence is against him. Up until the post-Milford Haven check, the only formulae for allowable compression stress which appear in the FF & P calculations are those from BS153 or a more conservative formula due to Professor Merchant. After the check had revealed areas of over-stress several new formulae were introduced with allowable stresses higher, and in some cases considerably higher than the former values.

The critical buckling mode for panels with plates thicker then $\frac{3}{8}$ inch was almost always Mode (ii), i.e., the buckling of the longitudinal stiffeners between adjacent rigid transverse beams. For this mode the high water mark for allowable stress was the outright adoption of half the critical buckling stress, but with the added limitation that the proper safety factor was to be maintained against yield of the whole section. It is important to point out that in these check calculations the effective lengths of the compression panels was taken to be the full length between transverse stiffeners, which we agree is the only safe course of action in our present state of knowledge.

Some idea of the extent of the increase above BS153 values can be seen in the table below. The range of slenderness ratios quoted covers the majority of cases on the actual panels and the BS153 values are those for the permanent service condition.

Table showing allowable stresses in stiffened panels.

l/r	BS153	$\frac{\sigma cr}{2}$	Increase per cent.
50	10·9	13·5*	24
60	10·1	13·5*	34
70	9·0	13·1	46
80	7·9	10·0	27
90	6·8	7·9	16

* These values are limited by over-all yield at safety factor 1·7. Values are in tons per sq. in. and relate to a HYS panel.

The highest percentage increase is seen to be at $l/r = 70$, for lower values the yield stress condition intervenes.

A slenderness ratio of 70 is typical of the panels so that if the BS153 formula is correct the use of $\frac{\sigma cr}{2}$ with its 46 per cent. increase of stress means a very real incursion into the safety margin. For example, the BS153 values are based on a load factor of 1·7 so that, taking $l/r = 70$, the collapse stress according to BS153 would be $1{\cdot}7 \times 9{\cdot}0 = 15{\cdot}3$ tons per sq. in. If the allowable stress is taken as 13·1 tons per sq. in., then it would appear that the real safety factor is $15{\cdot}3/13{\cdot}1 = 1{\cdot}17$ instead of 1·70 as required.

It is difficult to trace the time sequence through most of this checking work since the sheets are undated. The date of the adoption of the $\frac{\sigma cr}{2}$ rule is however identified by a comment on the calculation sheet where the rule is set out :—" Suggested to Dr. Kerensky 27–28th July—agreed as reasonable ". From that time on the rule appears to have been used in determining whether reinforcing was needed in any box and if so, how much.

The final summary sheet in the calculations setting out which boxes need strengthening, stated clearly that in some cases the strengthening was required for the permanent service condition and was not solely the result of erection requirements. Among the areas identified in this way is the bottom flange of box 8.

At that stage of the investigation, early in August, it was clear that some boxes would have to be strengthened and Hindshaw was advised accordingly. Later, as the results from the computer runs gave more reliable values of stresses throughout the structure, it was found necessary to add to the number of boxes affected. Hindshaw protested against the orders for strengthening coming to him in " bits and pieces " because, as he pointed out, it made it much harder for him to keep up the morale of the men and their confidence in the safety of the work.

In all, 160 tons of extra stiffening was provided at that time. Of this, some was fitted to boxes already erected but the majority to boxes still on the ground. The chief items of stiffening were :—

(i) Doubling up of the longitudinal stiffeners in the bottom flanges of boxes 12, 16, 17, 19 and 21. This was done by welding in 6 in. x $3\frac{1}{2}$ in. x $\frac{3}{8}$ in. angles midway between existing bulb flat stiffeners.

(ii) Substitution of 6 in. x $3\frac{1}{2}$ in. x $\frac{3}{8}$ in. angle splices for the 4 in. $\times$ $\frac{1}{2}$ in. flat K-plates for bottom flange splice and for the splice of the lowest stiffener on the webs at splices 13–14 to 20–21 inclusive for both east and west sides, and for splices 7–8, 8–9 and 9–10 on west side.

(iii) Increase by 50 per cent. of bolts in transverse seam of lower flange at splices 13–14 to 20–21 inclusive.

(iv) Addition of 15-in. channels as further transverse stiffeners in boxes 8, 9, 17, 20 and 21. The channels were fitted so as to break down the span of the longitudinal stiffeners from 10 ft. 6 in. to 5 ft. 3 in.

(v) Additional longitudinal and transverse stiffeners to the outer sloping webs of boxes listed in (iv).

(vi) Increase by 50 per cent. of bolts in top flange splice 17–18.

(vii) Addition of reinforcing plate around opening for main tower in box 17.

(viii) Increase by 100 per cent. of bolts connecting main diaphragm over pier 11 to outer sloping webs.

(ix) Massive strengthening of tranverse diaphragms at boxes 8 and 12. In box 8 sixteen vertical channels 15 in. x 4 in. were welded to the existing diaphragm plate and some new horizontal stiffening provided.

(x) Concreting in of K-plates on the already erected east span 14–15 for bottom splice 7–8, 8–9 and 9–10.

We agree that this stiffening was vitally necessary, although none of it directly affected the section which collapsed.

Before Crossley left London in early August to return to Melbourne, he wrote a second report on the situation at that time. He comments that most of the regions of over-stress have been tackled but points out that the flange stresses in the final dead load condition were still not accurately known. His notes on the state of the diaphragms at piers 11 and 14 are of particular interest. He states that they were found to have—

> " very high stresses over the bearing and no capacity to take eccentric loading. Also the critical stresses in the diaphragm over the bearing are too low. Stiffening has been added to rectify these faults but the compressive stress over the bearing during erection will still be close to yield and with any non-uniformity of bearing loading will exceed yield."

In other words, despite the massive extra stiffening the diaphragms were still unsatisfactory. Bearing in mind that the Milford Haven failure was traced to the under-design of a transverse diaphragm and that stress analysis for such a member is full of imponderables, we urge that particular attention should be given to checking these elements to ensure that yielding will not take place either during erection or in service.

4.2.1 Miscellaneous Effects.

BS153, and all other codes on the design of bridges, call for a proper consideration of the effects of wind, temperature and a number of other miscellaneous effects.

Freeman when describing to the Commission the process of design used for the West Gate Bridge, stated that stresses induced by wind and temperature were among the effects considered.

FF & P have not produced any calculations to show that they ever assessed the wind and temperature effects on the box girders, and indeed under examination they never claimed to have made any such calculations.

These secondary effects remained therefore, as unknown factors eroding the already slender margins of safety. Their potential influence is particularly important in the erection conditions, when the safety factor, to start with, is only 1·31.

Five secondary effects which could possibly be significant are—

(i) Wind ;

(ii) Temperature ;

(iii) Stresses permanently induced by erection forces ;

(iv) Stresses induced by settlement of supports ;

(v) Stresses caused by creep effects in the concrete of the composite deck.

The wind force on the bridge during erection has been estimated by Maunsell, London to cause stresses at box 17 as high as 2·1 tons per sq. in.

During the erection of the half spans 10–11 and 14–15, the wind stresses could also have been relatively high because the half girders would have not only a larger co-efficient of drag than the more streamlined full section, but also a very much smaller section modulus for bending in the horizontal plane. It is estimated that wind induced stresses in the half span would have been of the order 1 ton per sq. in. for the specified wind speed.

The main temperature effect is caused by differential temperatures between the upper and lower flanges of the box. The bridge is most sensitive to this effect during the erection stages before the concrete deck is placed.

Under solar radiation the separate spans would all want to hog upwards and if they were pin-ended this action would not be accompanied by induced stresses, other than minor effects due to local non-uniformity of temperature. The continuity of the girder, however, causes bending moments to be induced at the supports and these act to cause compression in the upper flange throughout the entire span. The magnitude of the stresses induced depends on the temperature differential, but at a rough estimate could amount to about 1 ton per sq. in.

The method of erecting spans 10–11 and 14–15 in two halves meant that unless the two halves were made with exactly the same camber, permanently locked-up stresses would be induced when bringing the two halves to the same level. Similarly locked-up stresses would be induced if there had been any errors in setting out the horizontal alignments of either half girder.

For the conditions actually encountered on the bridge, these permanent stresses induced during erection amounted to approximately 1 ton per sq. in.

Other erection-induced stresses would occur if the cantilevers were not set out to the correct profile, so that their reactions on to the piers were other than assumed in the calculations. There was some evidence that this had already happened when cantilevering out from pier 14 towards the temporary prop on the east side.

It is unreasonable to assume that there will be absolutely no settlement in the future, in any of the bridge piers. It is wise therefore, to know what sort of stresses are likely to be induced in the structure by settlements of various magnitudes. Maunsell, London conducted some analyses along these lines and found stresses in the box girder of up to about 1 ton per sq. in. induced by a settlement of 6 inches at either pier 11 or pier 12.

The concrete deck has been assumed to work in a composite manner with the steel, so long as the concrete remains uncracked ; although it is noted that for one section the concrete was assumed still uncracked under a tensile stress of over 600 lb. per sq. in. The creep of concrete under stress would gradually change the stress distribution, and this effect too could become significant.

It will be seen that while none of the secondary effects are large they might in combination cause a dangerous erosion of the safety margin. It is not suggested that all the effects could act together to cause a cumulative effect at any one point, but it is suggested that certain combinations are statistically probable and should have been investigated.

4.2.2. Shear Lag.

The classical theory of bending is based on certain assumptions which result in the theoretical stresses at any section of the beam being linearly related to the bending moment at that section. The bending stress at a given section is, by the classical theory, the same for all points equidistant from the neutral axis of bending. This simple theory was used by both FF & P and WSC in making their calculations on the stresses in the box girder.

When large concentrated loads, such as the reactions at piers or cable anchorages, act on the girder, the simple beam theory is not strictly applicable because of the shear distortions which take place locally around such points of load concentration.

The modification to the classical beam stresses caused by this effect is called shear lag. The box girder is peculiarly sensitive to shear lag effects because of the generally thin plates, with their consequent high shear stresses and the fact that some parts of the flange are far removed from the nearest web.

FF & P made no calculations of shear lag effects until after the bridge had collapsed. Then in presenting to the Commission calculations on the stresses in span 10–11, prior to failure, they included what was claimed to be a shear lag calculation. Neither Roberts nor Brown could explain this calculation to us and it was certainly not self-explanatory. Curiously the calculation is directed only to finding the shear lag at the centre of span 10–11 where we would have thought its value was negligible, because not only are the shears low at that section, but so too is the local intensity of loading. FF & P's calculations led them to the conclusion that the shear lag effect at mid-span was such as to increase the bending stresses in the flange immediately over the inner and outer webs by $7\frac{1}{2}$ per cent. and to decrease the flange stress for points mid-way between webs by the same percentage. When it was pointed out to Roberts that if the shear lag really was $7\frac{1}{2}$ per cent. at mid-span it must be even greater close to the supports, he expressed the opinion hat shear lag, was greatest at mid-span and lowest at the ends. He was twice asked to reconsider that answer, but consistently maintained his opinion. Roberts also stated that he did not think it necessary to calculate shear lag effects.

The detailed check analyses conducted by Maunsell, London, after the failure, included shear lag calculations for several of the areas where concentrated loads were applied. They used the finite element method of analysis to demonstrate shear lag effects of up to 80 per cent., that is, the theoretical stress is 80 per cent. greater than that given by the classical theory as used by FF & P.

In places Maunsell, London's analysis showed peak stresses caused by shear lag in excess of the yield stress and there were areas of up to about 30 sq. ft. where, according to Maunsell, London, the stress was never less than 90 per cent. of the yield stress. Such concentrations can hardly be called local.

It appears evident to us that a sophisticated structure such as a trapezoidal box girder with internal webs cannot be safely designed without taking into account the shear lag effects. This is particularly so when the safety factors are as low as those used on the West Gate Bridge.

FF & P have shown us a copy of a document for use within their organization entitled " Outline Guide for the Design of Thin Walled Box Girders ", (4th draft). This document, dated 30th November, 1970, tentatively suggests that shear lag effects should be assessed, using the method specified in DIN 1073. They recommend that for compression flanges, the total stress including shear lag should not exceed the values given in BS153 (Table 3B) by more than 20 per cent. nor the values for the allowable stress in buckling by more than 30 per cent. There are a number of places in the box girder of the West Gate Bridge which, we believe, would fail to meet this requirement even after the post Milford Haven stiffening has been added.

4.2.3. Distribution of Load between the Inner and Outer Webs.

The distribution of the total shear loads between inner and outer webs is not a statically determinate problem and a reasonably complex analysis is needed to get a meaningful distribution.

FF & P for most of their work assumed that the inner and outer webs shared the shear equally although they gave no reason for this estimate. In the post Milford Haven checks they continued to use a 50/50 distribution, but introduced also two other possible distributions, one with the outer webs taking 59 per cent. of the total shear and the other with them taking only 42 per cent. Both of these latter distributions were embarrassing because, if true, they would have involved over-stress in the bolted connections between the webs and the load bearing diaphragm. In the 59 per cent. distribution, the over-stress would have been at the outer webs while with the 42 per cent. distribution, it would have been at the inner web. FF & P, without giving any reasons stated in the calculations that " Truth probably between " the 50/50 and the 59/41 distributions.

According to the analysis made by Maunsell, London the " truth " lies nearer to 40/60, that is the outer web taking less than half the total shear. The Maunsell, London answer certainly appears to be more rational, particularly when viewed from the point of view of compatibility of the shear deflections in the four webs.

The analyses made by FF & P to get the 59/41 distribution are extraordinarily crude and appear to be quite illogical. Several methods have been used, a typical example being the one headed "Distributing Load by Statics". For this it was assumed that the moment of the vertical component of the force in the outer web, taken about the inner web, was equal to the moment of the reaction of one of the main bearings about the same point. This assumption to us is meaningless. Why pick the inner web as the point of zero moment, and why ignore the horizontal component of the force in the sloping outer web ? Brown when questioned on this analysis agreed that it made no sense to him either.

The attitude of both Brown and Kerensky to this problem appears to be that it is just too hard to solve analytically, and that an intelligent guess based on " engineering judgment " is the preferred solution. Brown indeed admitted to just that proposition. He even rejected the 59/41 distribution favoured in the later FF & P calculations and returned to the 50/50 division. When it was pointed out that FF & P had themselves used the 59/41 division for design purposes on four separate occasions, Brown said this did not make him change his peronal opinion. He later added that he thought it unlikely that any amount of calculations would make him change his mind.

Kerensky referring to the same problem said " It is a very difficult problem and whichever answer you tell me, I do not believe it ".

We agree with Kerensky that it was a difficult problem but we think that the degree of uncertainty could have been reduced from the extremes of FF & P's various analyses, 59/41 to 42/58. If that degree of uncertainty existed we hold that the use of a safety factor as low as 1·31 during erection was wholly unjustified.

Kerensky said that his own approach would have been to take a 50/50 distribution and then add on a percentage to the loads in both webs to make sure he covered all possibilities. In the absence of a proper analysis, this would be reasonable, provided the added percentage was big enough. Unfortunately it was not followed by FF & P's designers.

It is of note that neither Brown nor Kerensky agreed with what was done in the FF & P calculations and this raises again the question of who was in charge of the design, particularly after July, 1970.

4.2.4. THE K-PLATES

A mathematical analysis of the behaviour of the K-plates is complex, even if limited to the plate itself and its immediate connections. When the analysis is extended to include the buckling of a panel containing K-plate splices the work becomes extremely involved. Some simplifying assumptions are necessary to get a practical answer on the behaviour, and the extent of these assumptions makes quantitative values unreliable.

Hardenberg who carried out extensive calculations after the disaster states—

> " The stress pattern in the joint is complex and no analysis can claim to be precise."

Grassl who also worked on this problem concluded—

> " The complex nature of this joint evades accurate analysis, even when the method of finite elements is pursued within reasonable limits. In particular, stressing beyond yield requires fresh assumptions regarding the system and the materials."

We agree with both these opinions. It is obvious that the K-plates will be subjected to very high stresses because of the eccentricity of the connection, and while there is such an uncertainty in the analysis we ourselves would have rejected the detail altogether, but hold that, if used at all, the safety factor must be suitably increased to allow for the " factor of ignorance ".

In fact, neither FF & P nor WSC made any special allowance for the safety margin when considering the K-plates.

Hardenberg defended the detail on the grounds that it would be " difficult to devise a splice which is simpler to fabricate and erect "—he does not go on to add—" and more likely to buckle ". He stated that, after checking that the local slenderness ratio of the K-plate itself was less than the slenderness ratio for the panel as a whole, WSC thereafter gave little further attention to the detail until after the collapse. Hardenberg reminded us that the whole picture of the allowable stresses in the panels was " somewhat enigmatic " due presumably to Brown's failure to provide figures, and this " tended to put problems like K-plates in the background ".

Hardenberg's analytical values for the failure stresses for panels containing K-plates are of interest, but must be viewed in the light of his own statement on their limitations. He concluded that for the inner panel of the upper flange at the 4–5 splice, the critical stress was 13·6 tons per sq. in., and for the outer panel 14·5 tons per sq. in.

FF & P witnesses, Roberts, Brown and Kerensky, all defended the K-plate as " satisfactory ". They conceded that the detail was less than 100 per cent. efficient but maintained that it was adequate to carry the loads actually imposed. This, if true, would mean that the actual stresses in the panels were somewhat less than the allowable stresses for unspliced panels. FF & P's own calculations make it abundantly clear that in many panels the actual calculated stresses are not less but greater than the allowable values, thus completely invalidating the case of the admittedly weak K-plates.

Kerensky at first maintained that the K-plates were " adequate ", but later conceded that for some situations ½-in. thick plate was inadequate. He rejected the suggestion that an angle section would have been a better splice cover, on the ground that " it would not be paintable ". Despite this, FF & P had already authorized the substitution of angle splice members in place of K-plates for about half of the bottom flange splices.

Roberts claimed " these K-plates were carefully designed and they were quite adequate for the job ". Asked if he had thought of doing model tests, Roberts stated that this was unnecessary since " it can be done by calculation ". Roberts' calculation turned out to be the simple division of the estimated load by the area of the K-plate. Not only did Roberts not take into account the obvious eccentricity of the K-plate with respect to the bulb flats which were spliced, he even denied that this had anything to do with the stress situation. Under intensive questioning Roberts finally agreed that his analysis was a " crude approximation ".

Brown claimed that he personally had made an analysis of the stresses in the K-plate, but this too turned out to be a simple dividing of load by area. Brown appeared to take the view that anything more sophisticated in the way of analysis was not with the realm of " practical engineering ". Like Roberts, he dismisses the eccentricity effect as of no practical consequence. When reminded of the experimental evidence from the tests of Murray, Stevens and Grassl that high bending stresses existed in the K-plate due to eccentricity, Brown said he was surprised. While maintaining that the simple approach was the only practical method, Brown would not agree that the safety factor for the K-plates should be increased to cover the uncertainties of analysis.

Neither Brown nor Roberts made clear what they would have considered an appropriate allowable stress, although Roberts talked about using " a certain effective free-ended length ".

The only design calculations we have been able to find on the K-plates used an equivalent length factor of 0·85 and took the buckling length as that between the centres of the last holes in the bulb flats to be spliced, normally about 15 inches.

In a table prepared by FF & P after the Milford Haven collapse, the value of the actual mean compressive stress in the K-plates at each splice was compared with the full value of the Euler critical stress, based on $0{\cdot}85l$ as defined above. The ratio, actual stress divided by Euler stress, was examined as if it were a safety factor. Some of the factors are given as low as 1·05, but a later insertion has multiplied most of the low values by 1·24 to bring them up to a minimum of 1·30. No explanation is offered for the 24 per cent. increase, the same value being used for widely different parts of the bridge.

An outstanding case occurs, in the table, for the 8–9 splice on the bottom flange when the extra long K-plates were used. There the " factor " is given as 0·76 and no attempt is made to boost it by 24 per cent. In other words, the calculations show that this K-plate is loaded above its Euler critical load. Roberts, when questioned about this, admitted that in this particular plate the stress " had already gone too far ". Brown, on the other hand, repudiated the calculations of the FF & P designers and said the basis of the calculation was unsound. Brown claimed that the K-plates should have been designed on an equivalent length of $0{\cdot}7l$, the buckling length being taken as the clear distance between the facing ends of the bulb flat stiffeners, generally about 12 inches. He agreed that he would then use the BS153 formula to get the allowable stress.

Just why Brown should take the ends of the stiffener is not clear, because on buckling, the K-plate deflects away from the stiffeners, so that their ends can be of no particular significance.

Taking Brown's method on its face value the equivalent length of the long K-plates at 8–9 would be $0{\cdot}7 \times 22\frac{3}{4}$ inches = 16 inches, giving a slenderness ratio of 111 and an allowable stress of 5·0 tons per sq. in. The actual stress in the K-plate was stated in the FF & P table to be 7·6 tons per sq. in., but this latter figure appears to be much too low. It is quite inconsistent with that given by FF & P in the dead load and live load analysis where the stress in the flange at the 8–9 splice under service conditions is given as around 13 tons per sq. in. Even using the lower value of the table it is clear that there is more than 50 per cent. over-stress in the service condition, even after allowing for Brown's version of the length.

Brown and Kerensky made the point that the actual K-plates used were in fact some 7 per cent. thicker than the specified size because of rolling margins. They suggested that the yield stress would also, in all probability, have been above the specified minimum and that it was reasonable to take these extras into account. We cannot agree that this is a safe way to design any structure.

In January, 1971, after the Commission had commenced, FF & P prepared further calculations based on their idea of the stress distribution in the K-plates. They calculated the centroid of a typical strip of upper flange plate, 21 inches wide, with its attached bulb flat stiffener, and repeated the operation for a section consisting of the 21-in. plate and a K-plate. They argued that the eccentricity of the line of thrust was that due to the very small shift of centroids between the two sections analysed. As a result of some unjustifiable manipulation with the eccentricity, as defined by FF & P, they deduced that the eccentricity of the line of thrust in the K-plate was 0·0000594 inch, ignoring the obvious offset of the order 0·5 inch. How anyone could be so stupid as to accept 0·0000594 inch as a rational answer is beyond our comprehension. It is alarming that anyone with so little insight into structural behaviour could have been engaged, apparently unchecked, on the calculations for West Gate Bridge.

What makes this irresponsible calculation all the more surprising is that it was in no sense part of the design calculations and FF & P were under no obligation to produce it. The only conceivable object in handing over post-failure calculations was to explain to the Commission why, in FF & P's view, the stresses were satisfactory. As mentioned elsewhere these post-failure calculations contain many other errors both in arithmetic and engineering principle. They exemplify the casual and careless manner with which FF & P treated the calculations throughout.

In our view the weakness of the K-plates was an important factor in the ultimate tragedy. At no time did FF & P give proper regard to the safety of these critical elements.

4.3.1. Stresses at 4–5 Splice Span 10–11.

Most of the matters raised in the preceding sections deal with stress calculations for parts not directly involved in the collapse of span 10–11. This has been necessary in order to report on the way in which calculations were undertaken. Any direct criticism of the calculations for the upper flange at the 4–5 joint in span 10–11 is not possible because no design calculations were presented by FF & P justifying the stresses at that point.

After the collapse a number of analyses were made and it is now generally conceded that according to the simple beam theory the stresses in the deck of box 4 north of the 10–11 span were very close to—

Values in tons per sq. in.

	Inside Edge	*Outside Edge*
(i) With north half span not connected to south half span	10·2*	4·6
(ii) As above but with kentledge added ..	12·5*	5·7
(iii) Two half spans pulled together, kentledge removed	9·0	7·3

* The two highest values were agreed to as correct by Brown and we concur.

The crippling stress for the ⅜-in. plate itself, buckling between stiffeners, is given by the usual Timoshenko formula as 15·0 tons per sq. in., although the inclusion of stresses due to welding would probably reduce this somewhat. The transverse splice would also tend to give a further reduction in the crippling stress.

It is clear from the figures given above that, when kentledge was added, the margin of safety was small. In their post-collapse calculations FF & P sought only to demonstrate why buckling of the inner upper panel should not have taken place, they made no claim that the approved safety factor of 1·31 had been maintained.

So far as the buckling of the longitudinally stiffened panel is concerned, any calculations are rendered of doubtful value by the uncertain behaviour of the splice, including as it does the weak K-plate. The working stress given by the BS153 formula, for the buckling of a 10 ft. 6 in. panel, is 9·0 tons per sq. in. Since the BS formula is based on a load factor of 1·7, it follows that the corresponding collapse stress would be 15·3 tons per sq. in. This value is, however, for a panel free from any splice.

As set out in Section 6.1.1. the Roderick tests showed that with otherwise identical panels the presence of the splice reduced the failure load by about 16 per cent. There are reasons for supposing that the reduction on the bridge might have been even greater, but assuming the Roderick reduction, the predicted collapse stress would be reduced from 15·3 to 12·8 tons per sq. in., thus bringing it down to very near the existing theoretical stress.

Hardenberg obtained a theoretical value of 13·6 tons per sq. in. for the failure stress of an inner upper panel with a splice. This is based on a sophisticated analysis which extends into the plasto-elastic range. It does, however, include a number of assumptions which must introduce an element of uncertainty.

Grassl also made calculations but these were based partly on experimental data and again included many simplifying assumptions. He concluded that the 10 ft. 6 in. panel without a splice would fail at 13·9 tons per sq. in. and that the presence of the splice would reduce this to 9·7 tons per sq. in.

FF & P in a statement issued in January, 1971, gave their considered opinion that the failure stress for the inner upper panel was 12·7 tons per sq. in.

Brown has argued that some end fixity should be taken for the longitudinally stiffened panels buckling between transverse beams, thus giving an increase in the theoretical buckling stress. We cannot agree that, in the present state of knowledge, any such increase is justified.

Various test results on panels are reported in Section 6.1.1. While criticism can be levelled against almost all the tests, in varying degrees of severity, it would appear from the tests that the failure stress of the inner upper panel with a splice is little, if anything, above the stress which theoretically existed in such panel in the bridge, when kentledge was added.

In summary it can be seen that failure of the inner upper panel was a possible consequence of adding kentledge either by local plate buckling or by buckling of the longitudinally stiffened panels. This conclusion is quite independent of whether there was any mechanical interference by the diaphragm as suggested in Section 2.2.3.

Without any doubt the collapse of span 10–11 on 15th October, 1970, was precipitated by the removal of bolts in the 4–5 splice. FF & P have attempted to focus attention on this single action claiming that it alone was sufficient to cause the failure. We feel that the severity of the stress concentration caused by removal of the bolts may have been over emphasized. Both Grassl and Stevens have used the finite element method to assess the stress concentration, but in both cases they assumed the inner upper plate was flat and carrying its full theoretical load immediately prior to undoing the bolts. The real situation was very different. The inner panel containing the splice was already so badly buckled that it must have shed a major part of its load and thereby created a stress concentration of its own in the region where the buckle was arrested, that is near to the intersection of the upper flange and the inner web. It is not possible to perform any precise analysis of the situation at that time because the nature and magnitude of the buckle are not sufficiently known. An approximate analysis suggests that locally the stresses must have been very close to critical, particularly bearing in mind that the inner web and its connecting bracket were mild steel.

This highly critical stage was passed when the two half spans were drawn together horizontally, and the stresses were later again reduced when the kentledge was removed. The simple beam theory stresses at that stage are given in the last line of the table above, but the values are of little relevance to the situation then prevailing because the damaged north half span was no longer behaving in the manner defined by simple beam theory.

In our view the margin of safety at this stage, although higher than when the kentledge caused buckling, was still low. The act of undoing the bolts was rather more a "last straw" than a " bolt from the blue ".

We believe that from 6th September, when the buckle was detected, right through until 15th October, the contractors and consultants were not justified in assuming they were dealing with a stable situation. They could no longer be certain that the proper and approved safety factors were being maintained and should have taken immediate steps to put any question of safety beyond reasonable doubt. This might have been done by adding reinforcement to the buckled panel or by erecting a temporary prop to relieve the damaged span from some of its load.

A contractor more familiar with the erection of large steel bridges might have recognized the danger signs more readily. The consultant engineers on site do not appear to have taken the matter sufficiently seriously, while the consultants in London did not consider the matter at all, because they knew nothing of either kentledge or buckles until after the bridge had fallen.

4.4.1. Conclusions on Design and Calculations.

The matters which have been set out in Parts 3 and 4 relate mainly to the design and calculations of the flanges of the box girders for spans 10–11 and 14–15. The design of other elements in these two spans, such as bolted joints, webs, diaphragms and bearings appear to contain similar errors or omissions which would have resulted in over-stressing of the structure. There is reason to believe that other parts of the bridge such as the concrete deck, the towers and the cables may be similarly affected.

Despite all the evidence brought before the Commission on design inadequacies and calculation errors, FF & P maintained throughout that they were in no way to blame. Kerensky, in reply to a question by Mr. Kaye agreed that there was " nothing that the partners of Freeman, Fox did or failed to do which in any way caused or contributed to the collapse of the bridge ".

Roberts, Brown and Kerensky all swore that they believed the design to be sound. Roberts and Brown went further and expressed the view that the strengthening added in August and September, 1970, was probably unnecessary and was the result of an over-reaction to the Milford Haven disaster. Roberts expressed this when he said the stiffening was " perhaps terribly unnecessary but it was a case of safety first at any price—almost ". When questioned as to whether safety first was his policy Roberts replied " It is not our policy but it was the atmosphere after the Milford Haven collapse—rather artificial ".

Freeman, in November, 1970, had telexed Birkett, no doubt intending that his statement should be seen by the Commission, stating—

> " Freeman Fox and Partners have now completed their design check and are satisfied that design is correct and sound so that bridge will perform all its functions safely in service and is also correct and sound for safe erection."

He goes on to explain that he means the design as then amended by the post-Milford Haven strengthening.

This appears to us to have been a most irresponsible statement, particularly in view of FF & P's own check calculations which still showed areas of considerable over-stress, despite the post-Milford Haven stiffening. Over-stress which the calculations clearly showed, would have occurred for both the service and erection conditions.

Freeman wrote again as late as 15th January, 1971, this time to the Authority giving FF & P's explanation of why span 10–11 collapsed. He said—

> " We have carefully checked the design of the span for the erection conditions to which it was subjected and can find no fault or omission that could in any way have caused or been indirectly responsible for the collapse."

We reject these claims of non-culpability by FF & P and find it hard to believe that those who made such claims can themselves seriously believe them to be true.

We assert that a basic cause of the tragedy at West Gate was the design inadequacies which led to the safety margins being much too low, and certainly lower than the specified values.

As to the partial failure, which occurred when kentledge was added, we think it probable that there was some mechanical interference which led to the initiation of a bulge, but contend that had the safety factor been adequate the K-plates would not then have buckled, and there would never have been a necessity to attempt the unbolting of splice 4–5.

We emphasize once again the folly of undoing the bolts at the 4–5 splice, but point out that after this had been done the span hung in a precarious situation for nearly an hour before it collapsed, suggesting that a small increase of safety margin might have made all the difference.

We find that FF & P approached the design of West Gate Bridge in a disorganized and unsystematic manner and without any real guidance being given to the engineers doing the work by senior men such as Roberts or Brown.

The calculations contain a great many errors of arithmetic and of engineering principle and these have gone unchallenged until this Commission.

It is doubtful if FF & P had any effective internal checking system. They certainly failed to give any adequate check to WSC's calculations although apparently prepared to approve them.

We find that FF & P made assumptions about the behaviour of box girders which extended beyond the range of engineering knowledge. Examples of this are the manner of treating allowable stresses in panels, the division of loads between webs, the behaviour of double and treble bolted joints using grip bolts and the extent to which load shedding can be safely allowed as one element becomes over-stressed and shares its load into nearby structure. In many cases the FF & P assumptions would appear to be unjustified and liable to cause serious over-stressing of the parts affected.

We cannot agree with the senior FF & P witnesses that the design, as it now is, is unquestionably sound and we stress the importance of having a thorough and truly independent check made before work is recommenced.

It is for all these reasons that we have formed the conclusion set out in Section 1.1.1. that the primary cause of the collapse of the West Gate Bridge was that " FF & P failed altogether to give a proper and careful regard to the process of structural design ".

PART 5. THE REPORTS.

5.1.1. THE MAUNSELL, LONDON, REPORT.

In August 1970, Wilson was so disturbed about the safety of the structure that he called for an independent check to be made of the calculations. The consulting engineers were not too happy about having their work submitted to external scrutiny and suggested that the check should be done by G. Maunsell and Partners in London. In theory, the firm of Maunsell, London, was independent of the Australian Maunsell, although Birkett was a partner in both firms. Wilson, perhaps unwisely, allowed himself to accept this suggestion. Instead of negotiating directly with Maunsell, London, Wilson commissioned the joint consulting engineers to have the work done, thus leaving it free for them to appoint Maunsell, London. In this way, Wilson failed to secure direct contact with the checkers. This isolation was very real and when later Wilson wrote directly to London, his letters purposely remained unanswered. Explaining this, Maunsell, London, wrote on 28th September to Fernie of Maunsell in Melbourne—

> " I do not propose acknowledging either of the letters from Wilson and shall be glad if you will provide any response as appropriate. We are aware here of the correct relation this firm has with the Authority."

The same letter refers to the First Interim Report on the design check and states that it had been " read out to Bill Brown before being sent off to you and he has been given a copy ". So much for independence !

It would appear that Maunsell, London, approached the task of checking a FF & P design in the same spirit that most engineers would have had towards anything connected with FF & P. It was inconceivable that there could be anything seriously wrong with it.

In their First Interim Report dated 25th September, 1970, they reported some regions of high stress around bearing diaphragms and anchorages but they were reasonably confident that the stresses would turn out to be acceptable. They also noted that the K-plates were a weak element but comment that they understood this matter had already been overcome by concreting-in the weak plates.

The note on K-plates refers to a large flow of correspondence between Maunsell, London, and Maunsell, Melbourne when it was discovered by the former that the K-plates were likely to be unstable.

By the time the Second Interim Report was issued on 22nd October, span 10–11 had already collapsed. The report was non-committal, saying nothing about stresses or safety, merely reporting what work had been completed and what was still in hand.

On 11th November, Baxter, the managing partner of Maunsell, London, issued a statement presumably intended to be read by the Commission, which, after explaining that they were doing the check for the joint consulting engineers, continued—

> " . . . Calculations to date entitle them to statet hat spans 10–11, if rebuilt to the present design by appropriate erection methods, will be satisfactory in all stages of steel erection up to concreting as planned—nothing in the calculations to date has indicated that this or any other part of the bridge as planned will not be satisfactory at any stage or in use."

The next Interim Report (No. 3) dated 25th November, 1970, started off by making it clear that the special statement of the 11th November had been issued at the specific request of the consulting engineers and although only two weeks had elapsed between the dates of the special statement and the third report, the situation had already changed. The report goes on—

> " We have now carried our own calculations further . . . As a result of these further calculations we advise that we now believe that certain details of the bridge will require modification . . . "

There followed details of several of the areas affected, including one where Maunsell, London, say the shear stress may be as much as 12·5 tons per sq. in. compared with their estimate of the allowable stress for that point of 7·5 tons per sq. in.

The next report was undated but from other evidence appears to have been issued on or about 18th December, 1970 ; it was headed " Interim Memorandum of the Analysis of End Spans ". This memorandum showed clearly that any illusions Maunsell, London, may have once had on the infallibility of FF & P had been shattered. They reported—

> " There are a number of areas which appear to us to be highly stressed and we consider that some modifications to the structure are required . . .

The following are the items which we consider to be inadequate :—

1. Concrete outer decks in shear at edge of spine box.
2. Spine box inner webs in box Nos. 1, 6, 7 and 8.
3. Spine box inner web to top flange connecting plates in box Nos. 3, 4 and 5.
4. Spine box outer cross-beams in shear near inner web.
5. Spine box centre cross-beam at splice.
6. End webs under expansion joint support beam.
7. Expansion joints upport beam in horizontal bending between spine box webs.
8. End connection bolts of front diaphragm of box No. 8 anchorage.
9. Longitudinal splices in web plate in shear.
10. Bottom flange splices at joint 2–3 due to eccentricity of plates.
11. Bottom flange splices in tension due to reduction in gross effective area at bolts.
12. Top flange in compression buckling in box Nos. 3, 4 and 5.
13. Top flange in tension in box No. 8 in middle cell.
14. Diaphragm at pier Nos. 11 and 14 due to compression stresses, shear stresses and bolt slip ".

All the above items relate only to spans 10–11 and 14–15. Item 12 is of particular interest, and in commenting on this item, the report, after stating that the stresses were too high in relation to buckling between transverse beams, went on to add—

> " The lack of experimental evidence for buckling of this type of panel does not warrant a departure from the usual Perry-Robertson values of BS153 ".

So far as item 11 is concerned, the report gave early results from special tests which were carried out to investigate the behaviour of the double row grip bolted splices used on the bridge. No previous knowledge was available for such joints and it would appear from the report that FF & P's assumption on how the joint would behave was unsound.

The final report issued by Maunsell, London, in January, 1971, was a most extensive document. The comments on spans 10–11 and 14–15 were much the same as in the fourth report, but more details were given. The report dealt with the whole bridge, item by item and drew attention to highly stressed areas which " require attention ". The areas included—

1. Concrete Deck.
2. Steel Cantilevers.
3. Cross Beams.
4. Flanges.
5. Webs.
6. Diaphragms.
7. Expansion Joint End Details.
8. Cable Anchorages.
9. Cables.
10. Towers.
11. Bearings.

Of all the items considered, only the design of the crash barriers emerged without any critical comment.

Richmond, the Maunsell, London, engineer largely responsible for the check calculation, came to Melbourne to give evidence before the Commission. It was astonishing to find that during his cross-examination by counsel for FF & P, no serious attack was levelled at any of the many criticisms, some of them serious, made in the report.

Kerensky dismissed the over-stressing claimed by Richmond as " small, local and transient ", but then he admitted he had not read the report. An examination of the Maunsell, London, report, whether it is right or wrong, certainly shows that the over-stress claimed was not small, being in places near 100 per cent. It was not local, because in some cases areas of many square feet were over-stressed and finally, it was not transient because in many cases over-stressing was said to occur under dead load only.

Faced with some of these facts, Kerensky conceded that " small, local and transient " would need modifying " in about two places " and went on to define one such detail which he said they were still trying to solve. His evidence was given on 27th April, 1971, months after the FF & P statement that the design was completely sound.

After the issue of the final report, it appeared that FF & P, with the approval of the Authority, had discussions with Maunsell, London, to see if matters of conflict between them could be resolved. Kerensky said that FF & P had " disagreed with a good many points and Maunsells themselves agree with our disagreement." He indicated that a preliminary report on their discussions had been given to the Authority but added " We as partners are not in agreement yet ". A copy of this report to the Authority would have been very useful to the Commission in assessing the validity of Richmond's statement about inadequacies throughout the structure, but no copy was offered to us.

The commissioners made it clear that they were anxious to see any statement on the final report which could be agreed to by the parties concerned, that is by FF & P, Maunsell and Maunsell, London, although they thought it desirable that Richmond himself should also be in agreement. Counsel for FF & P was told by the Commission that if he desired further to protect his client's interest by submitting such a joint statement, it would be accepted. No such submission was received. We were informed that it had not been found possible to do so for " legal difficulties ", but we do not understand what that means.

We have not been able to assess the validity of the Maunsell, London, report ourselves because the supporting calculations and computer outputs were not submitted to us.

We are left therefore, in the absence of any substantiated criticism, to assume that the report is essentially correct and hence that FF & P's design is inadequate in many places throughout the bridge.

5.2.1. THE RODERICK REPORT.

Professor J. W. Roderick was retained by the Authority to carry out tests and to report upon the likely technical reasons for the failure of span 10–11.

The tests which Roderick carried out are summarized in Part 6 of this Report. His analysis of the stresses was based in part on the structural behaviour observed during the tests and it is, to that extent, conditional on the test panels being truly representative of the panels on the bridge. The test panels were, however, fabricated by WSC and so should have been closely similar to actual panels.

The tests showed that the stress in the bulb flat stiffener was higher than the stress in the adjacent plate, and while the ratio of the stresses at low loads was about $1 \cdot 15 : 1$, at loads near to collapse it reached $2 : 1$. It was this characteristic which Roderick built into his analysis.

In his interim report of March, 1971, Roderick stated on the result of his tests, that when the stress in the stiffener reached $16 \cdot 5 \pm 1 \cdot 3$ tons per sq. in., the K-plates collapsed. According to his full analysis the stress in the most highly loaded stiffener would not exceed $12 \cdot 5$ tons per sq. in. even when kentledge was added. Roderick concluded therefore that some other " accidental " factor must have been present to cause the buckles of the inner upper flange at that time.

Unfortunately Roderick had underestimated the self weight of the boxes so that his self weight bending moment was about 15 per cent. too low. When made aware of this he revised his values and in the second interim report dated May, 1971, he estimated that the highest stressed stiffener carried $14 \cdot 7$ tons per sq. in. for the conditions with kentledge. This Roderick agreed " comes much closer " to the buckling stress for the K-plate. The accidental extra needed to trigger off instability, he said, was equivalent to about $\frac{1}{4}$-in. " upward movement of the panel ", which we take to mean about $\frac{1}{4}$-in. initial out of flatness.

Bearing in mind the limitations of Roderick's tests, particularly the facts that the panels were only 6 ft. 1 in. long and had a somewhat uncertain end fixity, we believe that the results of any analyses based on the tests must be open to some error.

Roderick himself states that the most important conclusion to be drawn is that the panels " are particularly sensitive to small changes in eccentricity of loading ". It follows that real eccentricities on the bridge, such as the $\frac{1}{4}$-in. out-of-flatness mentioned above, may well have provided that small extra factor needed to bring about instability.

We conclude that Roderick's work tends to confirm our own view that the probability of compression instability in the inner upper panel of box 4 north was a predictable consequence of adding kentledge.

5.3.1. THE GRASSL REPORT.

Dipl. Ing Hans Grassl, an engineer with wide experience on box girder bridges in Europe was retained by JHC to investigate the causes of the collapse and report thereon. Grassl, a consulting engineer, with offices throughout Germany, was appointed " Prüfingenieur " by the West German Government in 1948. This office is awarded to independent engineers possessing particular qualifications. Their duty is to check structural calculations for all types of structures, a legal requirement in Western Germany. Since 1948, Grassl has taken an active part in the design or construction, of a number of large cable stayed box girder bridges.

Grassl and his colleagues undertook a most exhaustive study of the West Gate collapse and his report, issued only four months after his commission, contained many hundreds of pages of calculations and discussion. It included the results of tests made at Grassl's request by Prof. Dr. -Ing. R. Barbré of the Technical University of Braunschweig. These latter results are summarized in Part 6.

In making a mathematical analysis of the buckling behaviour, Grassl, like Roderick, based his work partly on the observed behaviour during tests. He investigated, in particular, the buckling of a panel containing a splice and undertook calculations which traced the behaviour of the K-plates through into the plastic range. Grassl concluded that the failure stress of the inner upper panel with a splice, was 9·7 tons per sq. in. Some doubt exists in our minds as to the validity of some of the assumptions made in this analysis and we think the failure stress he calculated for the panel was probably somewhat low.

In some other parts of his work Grassl was proceeding on incorrect data as to the sectional properties. Conclusions in these parts are suspect.

Grassl was highly critical of the splice detail, particularly the K-plates. Among his conclusions are the following :—

> " Inner flange buckling would not have occurred under the kentledge if the girder had been provided with full strength transverse splices."

> " Had the half girder been designed to the factors of safety given in BS153, as prescribed in the specification, no buckling would have occurred."

> " The use of mild steel for the inner web and the stopping of the vertical stiffeners at about 12 inches below the top flange accelerated the buckling behaviour of the inner web when the transverse splice was opened."

So far as the unbolting of the splice was concerned Grassl reported—

> " . . . . With the splice designed as weakly as it was, it should, under no circumstances, have been opened, before it had been relieved of load by the action of a cantilever load."

PART 6. THE TESTS.

6.1.1. Tests on Upper Flange Panels.

Physical testing on typical panels of upper flange plating were carried out after the collapse in order to supplement the information obtained by mathematical analysis.

The results of analysis are necessarily restricted in their reliability because of the simplifying assumptions which must be made to deal with so complex a problem. Test results are in the same way limited in their reliability because of the physical impossibility of reproducing in a test all the conditions which apply in a panel when it is part of a bridge. Besides which, the panels in the bridge would themselves differ in such matters as initial imperfection and locked up stresses so that, at best, a test can only try to represent one particular set of conditions.

Tests were carried out independently by five authorities—

(i) Prof. N. W. Murray at Monash University, tests made at the request of the Commission ;

(ii) Prof. L. K. Stevens at Melbourne University, tests made at the request of the Commission ;

(iii) Prof. J. W. Roderick at Sydney University, tests made for the Authority ;

(iv) Prof. R. Barbré at Technical University of Braunschweig, test made in conjunction with Grassl, for JHC ;

(v) Dr. M. S. G. Cullimore at Bristol University, tests made for FF & P.

A summary of all tests made on panels of upper flange plating is set out on Table 1 where it will be seen that the test specimens used differ over a wide range in both length and breadth.

Counsel for FF & P in seeking to discredit the tests of Murray and Grassl said that the manner in which these particular tests were planned led one to conclude that Murray and Grassl had " determined prior to conducting their tests, that the K-plates used on span 10–11 were inadequate and thereafter set about the task of demonstrating that fact ".

We think this is directly opposite to the truth. Murray and Grassl were alone in taking the panel as it was and loading it to see what happened. Stevens and Roderick were forced to cut down the length of the test panel because of limitations in their testing equipment, but both made it clear that they thereby narrowed the test down to a study of the behaviour of the plate or of the splice itself, as distinct from the splice as part of a longer stiffened panel. Roderick was asked whether " the piece to be tested should simulate as closely as possible the piece on the bridge ? " . He replied, " I would not agree with you entirely. We avoided doing tests which might involve any condition of elastic instability because we did not want the things we were looking for masked by elastic instability—in this way we felt we were getting the true compression load-carrying capacity of the splice ". Cullimore went so far as to apply transverse constraints to the edges of the panels he tested, thereby preventing any buckling of the over-all panel. His tests, therefore, relate to the splice and not the stiffened panel.

Various criticisms have been levelled at every one of the tests, some of which we regard as trivial, others, without doubt, would have influenced the test result. Among the criticisms raised were—

(*a*) The use of flame cutting to prepare the test piece, and the failure to machine off the heat affected edges. This was chiefly a criticism of Murray and Stevens who used panels salvaged from the wreck, and of Barbré, who also left flame-cut edges. We cannot regard this criticism too seriously when we note that flame cutting was permitted for the panels on the bridge itself.

(*b*) Uncertainties in the calibration of the loading rig. This was again levelled at Murray who at the time of making his reports had only calibrated one of the five identical jacks used. The other four jacks were subsequently calibrated and we are satisfied that errors from that source were negligible.

(*c*) Inadequate stiffness of the load spreading beam fixed to the ends of the specimen. This affected the tests of Stevens and Roderick who used a single jack to apply the load across a wide specimen. Without doubt the central stiffener was loaded more heavily than outside ones and this may have caused premature failure.

PLACE	REF.	SECTION (Dimensions in inches)	LENGTHS & CONSTRAINTS (Dimensions in inches)	SPLICE	FAILURE STRESS T/□"	COMMENTS
MONASH (Murray)	A			Yes	10·3	
,,	B			Yes	9·1	Panel had longitudinal seam with 5" x 3½" x ¼" Angle stiffener.
,,	G			Yes	8·2	Panel had fabrication error involving overlong K-plates.
,,	H	Edge constraint		No	17·7	Outside edges of panel constrained against local buckling - Failure gradual.
MELBOURNE (Stevens)	K	48 3/16 for K, 49 for L. From wrecked span.	36 for test K, 27 for tests M & N. 102 for K, 72 for L, M & N. Flat ends top & bottom except for N which is pinned both ends.	Yes	14·8 *	
,,	L			No	16·5 *	Gradual failure.
,,	M	From wrecked span.		Yes	4·0 *	K-plates removed.
,,	N			Yes	0·3 *	K-plates removed.
BRAUNSCHWEIG (Barbré-Grassl)	A1	Made in Germany from nearest metric sizes.		Yes	12·4 *	Failure in middle bay.
,,	A2			Yes	12·8 *	Failure in middle bay.
,,	A3		As above.	Yes	7·7 *	K-plates removed. Failure in middle bay.
,,	B		As above.	No	17·0 *	Failure in 126 in. span. Transverse buckle at outstanding tip of Bulb Flat stiffener.
SYDNEY (Roderick)	P1	Made specially for test by W. S. C.	Splice, when fitted, at mid span. Flat ends used for all tests except P3, which had pin ends.	No	14·8	Repeat tests.
,,	P2			No	14·8	Repeat tests.
,,	P3			No	(9·6) X (8·1) Y	Test X with load concentric with centroid. Test Y with transverse eccentricity of 0·5 in. Neither test to collapse.
,,	S1			Yes	12·4	Repeat tests.
,,	S2			Yes	12·4	Repeat tests.
BRISTOL (Cullimore)	1	Plate edge constraints for tests 1, 2 & 3. Specially made for test.	Flat end. 40½ for 1 & 2, 126 for 3 & 4. Hemispherical joint. Splice at mid span for tests 1 & 2. Splice 2'-9" from end for tests 3 & 4.	Yes	16·3	Failure incomplete.
,,	2			Yes	20·7	Gradual failure.
,,	3			Yes	(15·7)	Test stopped.
,,	4			Yes	15·7	No edge constraints, failure involved torsion - gradual.

TABLE 1. SUMMARY OF TESTS ON UPPER FLANGE PANELS

NOTES: All panels failed suddenly except where noted.
* Stresses marked thus are based on actual section areas, all others are based on nominal areas.

(*d*) Non-coincidence of the applied load with the centroid of the section both horizontally and vertically. All workers took great care to avoid unintentional eccentricity of loading, but Murray's tests show some eccentricity did occur, probably because the specimens he used were asymmetric and contained wide outstanding plate edges whose contribution to the section was uncertain. The eccentricities caused by non-coincidence would have had some influence on Murray's test results.

(*e*) Thickness of members used in test piece not representative of those used in the bridge. This is chiefly a criticism of the Barbré tests for which the nearest metric sizes were used for some of the test material. For example, the K-plates on the Barbré tests were about 0·48 inch thick compared with about 0·53 inch used on the bridge. For collapse modes involving the failure of K-plates this may have caused test values perhaps 15–20 per cent. too low.

(*f*) Initial flatness being unrepresentative of that in the bridge panels. This criticism was levelled at several of the tests but the measurements made on the panels would suggest that it cannot be sustained.

(*g*) Friction grip bolts not checked for tightness. This again is levelled at Murray who was using panels from the wreck. Stevens, using similar panels, reported that he had found it necessary to tighten some bolts. There is, however, no evidence of bolt slip having taken place in any of Murray's tests.

(*h*) Instability of the projecting edge of the plate. The plate edge projection varied from as low as 2¾ inches in the Stevens' tests, to 21 inches in one of Murray's tests. There is no doubt that the large projecting width would become unstable at an early stage in loading and thus the failure load of the panel calculated on gross area, would be too low. Conversely, the very small projection in the Stevens' tests is equally not representative of the real situation and his collapse loads are in consequence likely to be somewhat too high.

(*j*) Uncertainty of end restraint condition. This applies to all tests although the Braunschweig tests are far superior to all others in this respect. Roderick acknowledged the importance of getting proper end restriant and his specimen P3 was tested to show the effect of changing the end eccentricity. He concluded that the panels were " particularly sensitive to small changes in eccentricity of loading ".

The evidence from the tests appears to be so conflicting that it is not easy to draw any positive conclusions. One interesting observation can be made on the test results for panels not containing a splice. The Monash test H gave 17·7 tons per sq. in., the Melbourne test L gave 16·5 tons per sq. in. and the Braunschweig test B gave 17·0 tons per sq. in., thus showing a reasonable agreement. The Sydney values for the same test however, were only 14·8 tons per sq. in.

For the normal panels containing a splice the disagreement is wider, ranging from 20·7 tons per sq. in. for Cullimore's 4-ft. long test piece down to only 10·3 tons per sq. in. on Murray's test panel A. It would appear that there was a length effect involved, which caused the failure stress to decrease with increasing equivalent length of test panel. The average of the six most relevant tests for the spliced panel (Monash A, Melbourne K, Braunschweig A_1 and A_2 and Sydney S_1 and S_2) comes out at 12·5 tons per sq. in., but this may not have much meaning in view of the different test conditions. This value would be for the outer upper panels, presumably the value for the inner upper panels would be somewhat lower.

The Monash panel B must be regarded as a special case because it contained an angle stiffener at the plate overlap ; furthermore, this angle stiffener was not continuous through the transverse ribs and was spliced with two K-plates each 3 in. x ¼ in. These latter would clearly have failed at a low load, thereafter leaving a very complex stress situation in the panel. These unusually weak K-plates would, no doubt, have been a factor causing the low test result, but this detail was, nevertheless, used on the bridge and would have caused a similar weakness there.

Monash panel G is another special case because the panel used contained a fabrication error resulting in the K-plates being 21¼ inches long between centres of bolt hole instead of 16 inches. This probably resulted in a failure load some 30–50 per cent. lower than would have occurred with a normal K-plate. It must be pointed out, however, that this so-called " freak " panel was in fact built into the bridge, although not, as it happens, at a section which would have been highly stressed.

The panels Melbourne M and N, Braunschweig A_3 and Sydney P_3 must also all be regarded as special cases.

It is understood that further testing is still in progress and this may lead to a better understanding of the behaviour of the panels.

6.1.2. TESTS ON DIAGONAL BRACE.

Cullimore carried out tests on the strength and rigidity of the special diagonal brace, first with the end plate as drawn and then with the bolt hole in the end plate displaced by 2 inches. (See Appendix E.) Unfortunately his test models do not represent the actual structure in a number of respects, the major differences being that the bent plate transverse ribs were made of HYS on the model, but were MS on the bridge and the holes on the flanges of these members were not in the same location as on the bridge.

The test results for both strength and rigidity would probably have been reduced if the models had been correctly made. The difference between the " correct " and " as made " end plates would also have been reduced.

The way in which the two tests were reported makes it difficult to draw direct comparisons ; it would appear, however, that the ultimate load of the brace as intended was about 15 tons, while the one with the incorrect end plate was about 10 tons, the corresponding spring stiffnesses at zero load were about 130 tons per inch and 80 tons per inch. The spring stiffness dropped rapidly so that by the time the load was 5 tons the values were down to about 22 tons per inch and 11 tons per inch respectively, the over-all extensions then being 0·13 inch and 0·20 inch for the two cases.

PART 7. THE MATERIALS.

7.1.1. The Steel.

The majority of the steel used for the box girders was a low alloy high yield steel specified to comply with ASA151 (1966), but with the modifications set out below. The steel so modified was designated LY 50 (Lower Yarra 50 kips per sq. in. yield).

Modifications—

(i) The minimum yield stress for plates and sections up to and including ¾-in. thickness shall be 22½ tons per sq. in.

(ii) The percentage of carbon as shown by plate analysis shall not exceed 0·23 per cent.

(iii) All material shall have a notch ductility corresponding to Charpy impact values of 15 ft. lb. at –5° C.

The reference to ¾-in. thickness was made at a time when it was thought that the main flange material would be limited to ¾-in. plate. Subsequent design modifications increased the maximum flange plate thickness to ⅞-inch, but no change was made in the steel specification to cover this thicker plate.

Test certificates show that for the ⅞-in. plate the average yield stress was 23·9 tons per sq. in., but individual specimens gave values as low as 22·5 tons per sq. in. For the thinner plates the mean yield stress was higher, reaching 26·2 tons per sq. in. for the ⅜-in. plate.

Extensive testing was done on four preliminary heats at the Port Kembla works of Australian Iron & Steel Pty. Ltd., to ensure that a suitable steel could be produced.

Very great care, involving a most extensive testing programme, was taken to see that no sub-standard material found its way into the structure.

It was not anticipated that any pre-heating prior to welding would be needed, partly because of the controlled low carbon equivalent and partly because of the relatively thin sections. Nevertheless, it was specified that no welding should be carried out on LY 50 steel at temperatures below 15° C (59° F).

Following the collapse of span 10–11 samples of steel were taken from various parts of the fallen steelwork and subjected to physical and chemical tests. Those tests made from parts stated on the drawings to be high yield steel gave entirely satisfactory results.

From the evidence we have examined we are satisfied that the LY 50 steel supplied for the bridge did meet the specification.

For some parts mild steel, generally to ND II quality and of Australian origin, was used in place of LY 50. Mild steel was not used on the box flanges or webs for plates thicker than ⅜ inch.

Bulb flats, which were used for much of the plate stiffening were imported form U.K. because this section is not rolled in Australia.

Bulb flats of both high yield and mild steel were used. The steel specified for the high yield bulb flats was to BS968, while the mild steel was to BS15.

Tests made on sections of bulb flat cut from the wreck showed that the material was in general satisfactory ; although it was noted that some of the bulb flats to BS968 had ill-defined yield points. Some samples had yield stresses as high as 34·4 tons per sq. in., which was about 50 per cent. in excess of specification.

After the Milford Haven collapse (see 2.2.1.) a considerable weight of extra stiffening was added to the West Gate Bridge. Because of the urgency this extra steel mostly in angle and channel sections, was obtained from stockists around Melbourne.

The steel obtained was nominally mild steel. It has not been possible to get reliable certificates for much of it.

On the west span 10–11 little extra stiffening had been fitted and we have not had specimens tested from the wreck. FF & P have stated that some of this extra steel has a yield stress as high as 20 tons per sq. in. If this is so, and the material is supposed to be mild steel, it is possible that some other properties, such as ductility and notch toughness, may have been adversely affected.

It is true that extra stiffening was added to plating which was normally under compression stress and so the change of brittle fracture in this material is remote. Nevertheless, we feel that the relaxation of the tight quality control used for all other steel in the bridge was unwise.

7.1.2. Concrete.

Concrete for the piers and pile caps was provided by Pioneer Concrete (Vic.) Pty. Ltd.

Test cylinders made at the time showed that the concrete was up to specifications. Core samples taken from the fallen pier 11 confirmed that this was so.

The specified characteristic strength of the concrete was 4,500 lb. per sq. in. Eighteen cores were taken from the wreck and after adjusting for the non-standard height : diameter ratio gave results as follows :—

Minimum for any one core	5,350 lb. per sq. in.
Maximum for any one core	6,850 lb. per sq. in.
Mean for all cores	5,970 lb. per sq. in.

We are satisfied that the concrete was of good quality and met the requirements of the specification.

At the time of the failure no concrete had been placed on any part of the deck, either on the east or the west sides of the bridge.

7.1.3. HIGH STRENGTH FRICTION GRIP BOLTS.

The special friction grip bolts were provided by McPhersons Ltd. of Melbourne. A special waisted shank was specified to give the bolts greater ductility.

Almost half a million bolts were to be used overall in the bridge.

We are satisfied that the bolts performed properly in the structure.

It is true that when span 10–11 fell a number of bolts sheared and others pulled out the plate edge. Under these conditions, however, the bolts are subjected to stresses altogether of a different order from those in normal service. The fact that so few bolts failed in the collapse speaks highly of their capacity to absorb excessive load.

7.2.1. CONCLUSIONS ON MATERIALS.

We are satisfied that the failure of span 10–11 cannot be attributed either wholly or in part to any defect in any of the materials used. This includes the various grades of steel, the concrete and the friction grip bolts.

With the single exception of the steel added for the post Milford Haven stiffening, we are satisfied that all materials used comply with the requirements of the specification. So far as this added steel is concerned, we do not have enough evidence to decide whether it complies or not. None of this added steel was, however, involved in the failure at the 4–5 joint on span 10–11.

PART 8. THE COMPETENCE AND PERFORMANCE OF THE PARTIES AND THEIR RELATIONSHIP DURING CONSTRUCTION.

8.1.1. GENERAL.

The various firms and companies assembled by the Authority for the West Gate Bridge enterprise, were all of the highest reputation, well established, and had demonstrated their ability by the successful undertaking of important projects. The senior personnel of each party should have been accustomed to working in conjunction with other organizations and familiar with the problems inevitably arising in such circumstances.

It is all the more astonishing therefore, to find on this project the confusion, lack of co-operation and antagonism that developed not only between each of the parties and the Authority, but between the parties themselves.

Some degree of dissension may well arise on a project of such magnitude, involving the need for co-operation among a number of people, but the discord at West Gate must have exceeded that of most other projects.

Fundamental to the unhappy situation were two factors which so vitally affected the relationships of all the parties, that it is desirable to deal first with them. These were the removal of WSC from the greater part of Contract S, and the selection of JHC as substitute contractor.

8.1.2. THE REMOVAL OF WSC FROM CONTRACT S.

From the very beginning of work on the project, the Authority and the joint consultants were concerned with the lack of progress by WSC. The work became further behind schedule as time passed, and other contracts were progressing satisfactorily.

As early as September, 1968, some difficulty was apparent, and during the last quarter of that year the Authority became seriously concerned at the lack of progress. Throughout 1969, there was a series of conferences between the joint consultants, the Authority, WSC and its parent company in the Netherlands, Werkspoor and of course many letters and telex messages all directed to the problem of the failure of WSC to achieve anything like a reasonable rate of production.

On 17th February, 1970, the Authority invoked the powers provided in Clause 47.1 of Contract S and served on WSC a notice alleging failure to " proceed with due diligence and expedition ", and calling upon WSC to show cause why the sanctions provided by Clause 47.1 should not be exercised (see 1.2.4.).

The service of this notice was advised by Birkett. It was done with the full knowledge of FF & P although at first, the London partners advised a month's postponement to give WSC a further opportunity for improvement in performance. In the long run however, they too agreed to the action taken, after Brown had seen for himself the true state of affairs in Melbourne.

The notice set out " Particulars of Default and Neglect ", all of which are within the general allegation of lack of diligence, but there was no complaint in regard to the quality of such work as WSC had completed. In substance, the complaint of the Authority was of inordinate delay by WSC. The company replied by a counter-notice dated 2nd March, 1970, refuting the allegation of lack of diligence and expedition and making countercharges. The dispute was ultimately resolved, the terms of the compromise being incorporated in the agreement of 13th March, 1970. By the agreement a financial settlement was arranged. It also provided for each party to withdraw its notice, and in substance each withdrew its claims against the other. It contained clauses requiring WSC to make available to the new contractor, by that time expected to be JHC, certain equipment and the services and expert knowledge of senior WSC engineers, and also to use its best endeavours to co-operate with the new contractor.

Although this matter of the dispute between the Authority and WSC forms part of the sequence of events leading to the collapse of span 10–11, we concluded that the time spent in a close and detailed investigation of the charges and counter-charges would not be justified. We were influenced by the fact that the dispute had been settled between the parties, and we did not seek to investigate such issues in any detail. Nevertheless, a great deal of evidence, both oral and documentary, was ultimately placed before us which dealt with this controversy. As a result we now find ourselves sufficiently well informed on the matter to make some general observations on the main issues, which we feel it is proper and necessary to do. These findings may be summarized as follows :—

WSC had from the very beginning of work on Contract S, lagged behind other contractors and by the end of 1969, were seven months behind on the original schedule, and even two months late on a schedule greatly modified to suit WSC's position.

The Authority, supported fully by Birkett, continually urged and entreated WSC to accelerate the rate of progress, and take such steps as were necessary to catch up with the programme.

At the hearing before us, WSC witnesses gave three main reasons for the delay. These were, labour disputes, the unavailability of steel at the time when it was required, and the failure of FF & P to produce design calculations and other data, when required, or at all.

We are quite satisfied that there was no delay or embarrassment caused by the unavailability of steel, and this was not relied upon by counsel in his final submission.

No doubt one of the main causes of WSC's failure to make any headway during the erection of span 14–15, was that they were plagued with industrial stoppages, many of which were demarcation disputes between the Metal Trades Federation and the Builders Labourers' Union. Other disputes appear to have arisen from quite trivial causes. For this unhappy state of affairs, the workers cannot be held blameless, but neither can WSC. The management on the site appears to have been entirely lacking in any ability to control the labour situation. Moreover, there was a good deal of evidence, which we feel disposed to accept that the company's organization was poor. James, for example, said that more lower-level supervision was required, that foremen were " thin on the ground ", and that the order of work should have been more closely laid down.

However, there were no labour disputes causing any serious disruption prior to commencement of erection of span 14–15, or after WSC ceased to be the erection contractor. The cause of the greater proportion of the delay was allegedly due to FF & P against whom WSC made a number of complaints.

First, in common with all other parties, WSC experienced the greatest difficulty in obtaining replies to their communications, which were rarely answered promptly, frequently answered after interminable delays and repeated reminders, and often not answered at all. This habit of FF & P, provided excuse for delay by WSC and doubtless had a bad effect on the morale of the contractors' staff.

The second aspect is the more serious, in fact that the required information on some very important matters was deliberately withheld. Typical of this latter aspect was the controversy over what became known as Document K, the details of which are discussed fully in Section 1. 2. 4.

The refusal of FF & P to supply to WSC the data which had been promised, gravely disrupted the contractors programme and initiated the friction that developed between the two organizations.

It would be unnecessarily tedious to set out all the instances of which the foregoing is but one example, but we have perused the exhibits referred to us by counsel and are satisfied that there were many such instances which cumulatively created great difficulty for WSC. In some cases where calculations were or should have been, in the hands of FF & P, and were not produced, the WSC engineers gave up in despair, and set about their own calculations at the cost of much time and unnecessary labour.

There was also continual controversy between WSC and FF & P in respect of notices of claims made by the former under Clause 35.2 of Contract S for extension of time, because of delays allegedly the responsibility of the engineers. We have not considered these items in great detail but there are some which appear to us to have been unreasonably rejected by the consultants. One instance was the matter of welding wire.

On 2nd December, 1968, the resident engineer prescribed Oerlikon wire, but on 17th December, changed this to Rockweld wire. After tests lasting some two months FF & P decided to return to Oerlikon, which was found to be satisfactory. All this indecision represented lost time to WSC.

In addition to these matters WSC alleged numerous drawing errors and ambiguities, and many alterations in design. To most of these allegations the engineers made a blanket denial. To have examined all of these would have taken an intolerable time, and we were not asked to do so, and cannot therefore make any firm finding on individual items. However, such evidence as we heard does induce us to agree that part of the WSC complaint was justified.

An important factor in considering WSC's attitude to the changeover was that Contract S was a lump sum contract containing a clause binding the contractor to pay by way of liquidated damages \$7,200 per day of delay, to a maximum of \$350,000. By 17th February, 1970, when the Authority served its notice under Clause 47.1, this sum had been far exceeded. We are satisfied that in the controversy between WSC on the one hand and the Authority and the consultants on the other, there is something to be said for the contractor, but we have no doubt that a good deal of the criticism directed to the contractor by the Authority was justified.

In June, 1969, eight months before the service of the notice by the Authority, there occurred an exchange of letters between Birkett and Schut, which was of considerable significance. Birkett's letter was dated the 3rd June. In it he voiced the strongest criticism of WSC organization, and told Schut that he was not in full control of his company's operations on the site. To this Schut replied on 5th June, that he " did not basically disagree ", admitted preoccupation with other matters, and while anticipating some improvement, said that " I cannot and will not promise dramatic improvement ".

At the beginning of February, 1970, the Authority's concern over WSC's lack of progress had become alarm. When advised by Birkett that a notice under Section 47.1 should be served, it appeared the Authority had little alternative. It may be that serving the notice, which did not automatically terminate the contract, was intended merely to bring the unsatisfactory situation to a head, but in fact WSC served its counter notice, and in ensuing discussions must have indicated that it was far from unwilling to relinquish the contract, unless a drastic revision of the situation was made by the Authority in order to rescue the company from its difficulties.

The company was losing money, and looked like losing a great deal more. The speed with which the dispute was settled indicates strongly that both sides were anxious to end the situation.

While the Authority and Birkett were discussing the situation in Melbourne and deciding that the service of a notice was necessary, FF & P partners in London had a discussion with Schroeder, and as a result, urged delay of a month before any more drastic action, to give Schroeder a chance to reorganize and improve performance. This advice was not accepted in Melbourne.

We think it a pity that some further effort was not made to arrange for WSC to continue with the contract, even if this involved financial concessions to the contractor.

After careful consideration, we conclude that in the circumstances existing in February, 1970, the Authority cannot be blamed for accepting the advice of Birkett to terminate WSC's contract and make new arrangements. What we do criticize is that the need to take this step was ever permitted to arise. The main responsibility for this unfortunate situation is probably shared between WSC and FF & P but the Authority must take some share of the blame.

8.1.3. The Selection of JHC as Substitute Contractor for Erection of Steel Spans.

Although for convenience we have considered under separate sections, the questions of the release of WSC from part of Contract S, and the appointment of JHC as substitute contractor, the two matters are very much connected.

We believe that if the Authority's only choice had been to keep WSC or appoint a new contractor unconnected with the project, there would have been much greater reluctance to release WSC. The fact that JHC was available and willing to take over the erection, albeit on special terms, undoubtedly influenced the decision to make the change. It follows that in judging the wisdom of releasing WSC from Contract S, much depends on the view taken of whether the choice of JHC was reasonable and correct.

At one stage, it was suggested to us that the propriety of the selection and appointment of JHC as substitute contractor was not a matter for our determination. We are plainly of the contrary opinion because this decision was an important link in the chain which led to the ultimate disaster, and was a major factor influencing the conduct of the other parties.

In seeking a possible substitute contractor, the Authority gave some thought to the second tenderer, which was a consortium consisting of Redpath Dorman Long Ltd., Johns & Waygood Ltd., and JHC. The consortium, formed purely for the purpose of tendering for the steel work of West Gate, had dissolved when the contract was awarded elsewhere. It was said that RDL were approached but displayed no interest in taking over the contract. The evidence on the matter is vague and contradictory.

Apparently the only approach that was made to RDL was through Johns & Waygood, and the information as to lack of interest was at best second-hand. Having regard to the subsequent history of the project since October, 1970, we have considerable doubt whether the inquiry was made in such a form as to be at all likely to arouse any interest.

In the absence of RDL from the field of choice, it was probably the natural thing to approach JHC, and there were certain advantages in doing so. JHC were on the location already and familiar with the overall project, and prepared to take over the work immediately. Their site management had demonstrated efficiency in the carrying out of their own contracts, and in particular had shown an ability to manage and achieve co-operation with employees and union officials, in marked contrast to WSC. On the other hand, JHC's experience with the assembly and erection of steel work was limited. No operation of the size and complexity of the West Gate Bridge steel structure had ever been undertaken by that company. Consequently the ability of their personnel to handle a project of such magnitude remained untested and conjectural.

There was a conflict of evidence as to whether JHC—which in reality means Mr. C. V. Holland—was eager to obtain the contract, merely willing to accept it, or reluctant to undertake the task, and persuaded to do so more or less as a public duty.

In the long run, the answer to this question may not be important. We accept the evidence of Holland that he was very conscious of the difficulties of taking over a contract in the " depressed state " of Contract S, particularly in the light of the severe industrial troubles suffered by WSC, and that while he was prepared to consider taking the contract for " community, professional, commercial and prestige reasons ", it was not something " to be rushed into ". Nevertheless, we feel that from at least December, 1969, JHC had conveyed to the Authority their interest in taking over the contract.

The expertise of the Holland group lay more with concrete than steel, and its management recognized that the nature of the design of the steelwork in this bridge was a field in which it had no experience. Because of this, JHC was prepared to undertake the work only on the basis of a " labour management contract ", that is to say that JHC would be responsible for the physical task of erecting the steelwork, but would have no responsibility for engineering decisions relating to final or erection stresses in the bridge.

It follows therefore, that basic to all discussions between the Authority and JHC, and to the ultimate agreement reached, were two factors recognized by both parties. First that JHC were undertaking to complete a contract already partly executed by WSC, and which was then in serious trouble. Secondly, that JHC were inexperienced in working to a design like West Gate.

From these two factors it followed that JHC would require—

1. Some strengthening of their engineering staff. An attempt was made to meet this requirement by WSC agreeing to second to JHC some experienced engineers and foremen. Messrs. Spee, Schott, Lund, Atkinson, Hart and Grist, all previously WSC employees were transferred to the JHC organization for Contract E. Further, by agreement between the Authority and WSC the services of Hardenberg and Van Veldhuizen were retained in an advisory capacity, and by the terms of Contract E, JHC were required to seek advice from them.
2. Some greater assistance from the joint consulting engineers in the way of advice and supervision, beyond that normally expected from a consulting engineer.
3. Some limitation of the responsibility and liability usually assumed by a contractor under any normal contract.

This third requirement arose from three separate considerations. First, there was the fact that part of the work had already been done by WSC and the fabrication of boxes would still be in the hands of WSC. Secondly, the relative inexperience of JHC influenced them to seek, and the Authority to grant, some limitation of the usual contractor's responsibility. Thirdly, it was necessary to protect JHC, as a matter of fairness, from some of the legal liability which was imposed on WSC under Contract S, because Contract E was a " cost plus " contract, and the liability imposed by Contract S was therefore not appropriate.

By 18th February, 1970, it had been agreed between JHC and the Authority that they would negotiate a suitable contract and on 16th March, a " Letter of Authority " was sent to JHC and they took over the work on 17th or 18th March.

The terms of Contract E were not finally settled until 10th July, 1970. In the meantime, the legal advisers of both parties experienced considerable difficulty in arriving at a mutually acceptable form of contract.

A number of alternative drafts submitted by the Authority's solicitors were rejected by the company's advisers.

It was suggested during cross-examination of JHC witnesses that having reached a fairly invulnerable position, with the work actually commenced, the attitude of JHC hardened perceptibly, and they insisted upon a more favourable form of contract than had been envisaged originally. The evidence on this is conflicting, but we are not persuaded that this was true. It seems to us that the JHC position had been made plain from the beginning and that the ensuing argument as to the form of contract was simply one of semantics.

Wilson, in evidence, said that he had always accepted that " very little responsibility could be placed " on JHC, and that this had been discussed with the consultants. He added that " we were all well aware of it while negotiations were taking place ".

Be that as it may, JHC certainly insisted on the inclusion in Contract E of Clause 19 (see Appendix C and Section 8.1.5.)

We have given serious thought to the question whether the Authority was in error in appointing JHC as substitute contractor for Contract E. Assuming the need to release WSC and find a new contractor, the Authority may well have felt that there was no practical alternative to JHC.

It was too much to expect the Authority to have gone through the process of calling for further tenders from other possible contractors, having regard to the time element which by early 1970, had become of major importance. We feel, however, that it would have been wiser to have made some real attempt to obtain the services of a contractor more experienced in steelwork than JHC, and in particular, one who had the expertise and confidence to have accepted a contract in the normal form and undertaken the usual responsibilities of a contractor. We can understand however, the Authority taking the view that this presented far more problems than the appointment of JHC.

The Authority no doubt considered that any lack of experience on the part of JHC would be compensated for by the general efficiency and management capability already demonstrated. Also it may well have been considered that the arrangements made for assistance from WSC and for increased responsibility of FF & P created a situation approximating to that which would have existed had the original tender of the consortium been accepted, and JHC been operating as part of that consortium.

In considering this matter, we have tried to put out of our minds the knowledge of what in fact happened between March and October, 1970, and have put ourselves as best we can in the position of the Authority at the time the decision was made. We take full account of the urgent situation in which the Authority found itself and the financial and other pressures upon it to expedite the project.

In the climate of urgency which prevailed, the willingness of JHC to undertake the work immediately, without the inevitable interruption which would occur if any other contractor was appointed, must have been an over-riding consideration for the Authority.

In the circumstances the appointment of JHC appeared the obvious solution to the problem, and the provision agreed upon for the support and co-operation of WSC and FF & P appeared to offer a practical arrangement with every chance of success. It is quite impossible to suggest that the Authority could or should have foreseen that this arrangement would break down.

For all these reasons we conclude that the decision to appoint JHC was justified by the circumstances then existing, and should not lead to any censure of the Authority.

8.1.4. Increased Responsibility of FF & P under Contract E.

There can be no doubt that the problem created by the limitation on the responsibility and liability of JHC and the increased responsibility of FF & P became a major cause of serious trouble and difficulty. The failure clearly to define the roles of the FF & P staff and JHC engineers led to a confusion that was disastrous.

It had a profound effect upon the relationship of the consulting engineers and the contractor, and indeed had some quite substantial effect on the relationship of the two consultants themselves —Maunsell and FF & P.

The FF & P organization held the view, expressed forcefully by Roberts in his evidence, that virtually all responsibility during the erection of the bridge rested with the contractor under the normal contract. This was the principle that they were used to working under and the new situation which they believed placed all responsibility for technical engineering decisions on the consulting engineers was thoroughly disturbing to them, and led directly to disharmony between Hindshaw and the contractor's engineers. Hindshaw's communications to his superiors in London disclose his bewilderment and his attempts to rationalize the situation. His work-to-rule practice and somewhat pin-pricking attitude to the JHC staff created from them a natural reaction. This unhappy state of affairs had an inevitable effect on the work of construction and created a climate in which the probability of error in judgment was greatly increased.

At times it appeared that JHC staff were seeking to thrust too much responsibility on to FF & P men, Hindshaw in particular. At other times they resented attempts at what they regarded as interference by the FF & P men. Similarly, Hindshaw's attitude varied from rejection of responsibility on the one hand, to attempted dictation on the other. FF & P engineers on site were on occasions prepared to undertake more responsibility than JHC were willing to concede.

Simpson said in evidence that his instructions from London before coming to Australia were that he would be " working in effect as contractor's engineer, controlling labour and directing operations as required ". He said also that Hindshaw had the same instructions, but added that, on his arrival it was apparent that JHC would not agree to this. Simpson said that he and Hindshaw ultimately concluded that their role was that of a normal resident engineer, except that procedures would be scrutinized more closely and assistance provided for their preparation.

This may have been their view at one stage, but it is apparent that Hindshaw, and his subordinates departed from this role on many occasions.

All this confusion and difficulty could and should have been quite easily avoided, had the increased responsibility of FF & P been clearly defined in the first place.

It is the greatest pity that nobody seems to have thought of the desirability of amending the contract between the joint consultants and the Authority, contemporaneously with the execution of Contract E, in such a way as to define the consultant's responsibility.

As it was, neither the FF & P site staff nor the JHC engineers knew where they stood, nor for that matter did the management of either body. The Authority and Maunsells were equally unsure. The views on the responsibility of JHC expressed by Wilson, Brown, Birkett and Kerensky are entirely irreconcilable and quite at variance with those expressed by the Holland witnesses.

The situation of doubt and misunderstanding had extremely serious consequences, as it created the circumstances in which the actions which were the immediate cause of the failure and collapse of span 10–11 were able to take place.

Why this unhappy uncertainty as to the limits of duty and responsibility between contractor and engineer should have been allowed to exist at all and why it should have been permitted to continue for many months, is beyond comprehension. The primary blame we feel is upon the

Authority, but plainly FF & P and JHC themselves, should have insisted on their function and responsibility being clearly defined, and are therefore only a little less blameworthy. As Maunsell were well aware of the situation, and had a direct interest and duty in the matter, they too must take some share of the censure.

8.1.5. The Effect of Clause 19 of Contract E.

This clause is set out in full in Appendix C.

It is neither necessary nor desirable that we should attempt to determine its legal effect, certainly not so far as it may affect the rights of the Authority and JHC—a task which may fall to some other tribunal.

However, it is clear that the inclusion of Clause 19 in Contract E, a contract to which FF & P were not parties, had no effect whatever on their legal liability. This was governed at all times by the terms of their own contract with the Authority (Exhibit 5). Their liability was the same under Contract E as it had been under Contract S. The increased responsibility attaching to FF & P when JHC became the contractor arose from an agreement made with the Authority.

Some time about 8th March, 1970, Wilson had received assurances from the joint consultants, as represented by Birkett and Kerensky, that in the event of JHC being appointed, they would " stand by the Authority in providing the extra staff and assuming the extra responsibilities that would be entailed in any such action being taken ".

As was pointed out in final addresses by both counsel for the Authority and for Maunsell the word " responsibility " was used in the correspondence and minutes exhibited, and in oral evidence in several different senses. It was used often as being synonymous with " job " or " function ". It was also used at times as an equivalent for legal liability, and on other occasions in a rather vague way as meaning a moral, as distinct from a legal, duty. The inability of some of the witnesses and personnel engaged in the project to appreciate this distinction led to some confusion.

It was clear enough to Freeman and Kerensky that what had changed was not the legal liability of FF & P but the nature of their task. It had become more onerous, more difficult to carry out satisfactorily. Whether Roberts ever appreciated this is doubtful. Certainly Hindshaw was completely confused on the matter. He believed that JHC had been relieved of any responsibility and that the entire burden had been thrown on to him—a situation which would have been bad enough had he been armed with the complementary authority. As it was he saw himself with all responsibility but no appropriate power to control the operation, which presented itself to him as quite an intolerable situation. That Hindshaw's position was difficult is perfectly true. To an extent he was right in feeling that he was given responsibility divorced from the needful authority. His mistake was in thinking that this unhappy state of affairs resulted from Clause 19. In fact, the increased responsibility of FF & P under Contract E arose not from Clause 19, but from the firm's agreement with the Authority to assume additional duties.

A perusal of Exhibit 267, a file of correspondence and draft documents, shows clearly the origin of Clause 19, and explains its presence.

It was recognized by all those involved in the negotiations that it was necessary to protect JHC from complete liability for rectification of faults as required by Contract S. Birkett's first draft provided for the exclusion of liability except in circumstances of " serious " negligence. Later this was changed to " gross " negligence, and this was ultimately accepted subject to the definition in Clause 19. What is quite plain is that this was never contemplated as changing in any way FF & P's responsibility—much less their legal liability.

When on the 30th July, 1970, Maunsell wrote to the Authority a letter containing the text of a telex from FF & P wherein the question of responsibility was raised, the Authority replied by a letter of 12th August in which the position is fairly summarized—

> " The Authority wishes to impress upon its consulting engineers that it expects and demands from them in the discharge of their duties the skill, care and diligence contemplated by the common law and expressed in the provisions of the consultants agreement of July, 1967.
>
> So far as this skill, care and diligence is concerned, generally speaking, the arrangements that the Authority makes with its contractors relating to responsibility should be regarded by the joint consulting engineers as immaterial."

8.2.1. The Authority in Relation to Other Parties.

It is important to appreciate what was the role of the Authority as constituted by its Memorandum and Articles, and its governing Statute. It is not a bridge-building instrumentality, and it was not intended that it should itself build the bridge, relying upon its own engineering staff.

It set about the task in the normal manner of any other " building owner ", by first selecting consulting engineers, seeking from them a design and specifications and relying on their advice in the selection of contractors. Nevertheless, it did not adhere strictly to this role. Among the directors of the Authority, and its employees were men of some engineering experience. In particular, the general manager of the Authority, Mr. C. A. Wilson, was himself a professional engineer with considerable experience of bridge design. He had for many years been a senior design engineer in the Bridge Division of the Country Roads Board of Victoria, and in that capacity had become involved in the planning, but not in the actual design, of the King's Bridge in Melbourne. Failure of the King's Bridge by brittle fracture of high yield steel had a lasting influence on Wilson ; although he was anxious not to get involved in the professional engineering details of the West Gate Bridge design, he nevertheless insisted that the designer should take particular precautions to ensure that brittle failure did not occur.

Without doubt these pressures on the designers influenced the development of the design, and in particular led to the extensive use of grip bolts for all site connections. Doubts about the ability of Australian fabricators to deal with the complex welding problems of an orthotropic steel deck which could take the traffic directly, led to the abandonment of this form of deck in favour of a reinforced concrete slab made to work in a composite manner with the upper steel flange plate.

Not unnaturally, Wilson expected a firm with the world-wide reputation of FF & P to produce an economical solution to the design of the bridge, but he placed no particular financial limits on the cost. There was certainly no pressure by the Authority to achieve cheese-paring economies, particularly if by so doing any risk to the safety of the structure was involved.

Apart from the influence that Wilson exerted upon the original design, he also from time to time intervened in the work of construction. It appears from the correspondence produced to us that in the early stages this intervention by Wilson was resented to some extent by the consultants. We ourselves make no criticism of Wilson for this, indeed, to have stood by without interfering when he observed undesirable or unwise features of the work would have been not only impossible for a man of his temperament, but a matter of considerable criticism had he failed to make use of his talents and extensive experience in the Authority's interest. Our criticism is that Wilson did not do more in the way of inspecting the project, as the Authority was entitled to do under the terms of its contract with the consulting engineers, especially when it began to show signs of trouble. It is doubtful that he ever visited the west side at any relevant time. He certainly did not between the raising of the two halves of span 10–11 and the collapse. After JHC took over the assembly and erection of the steel boxes, Wilson in his own interests should have kept a close observation on the work. Had he done so and made some forthright comments to the engineers the serious situation which eventuated might have been avoided.

Practically every one of the other parties found themselves in dispute with the Authority at some stage or other. This in itself would suggest something radically wrong with the approach of the Authority's officers—which means Wilson. However, apart from the matters just discussed, it is only fair to say that little emerged from the evidence to suggest, much less to substantiate, any such failure in attitude to the other parties.

In the dispute with WSC culminating in the change of contractors, the Authority should have appreciated that FF & P were failing in their responsibility toward the contractor, and should have taken a stronger line and insisted on the engineers co-operating fully with WSC. This aspect apart, the troubles which led to the " change-over " were not attributable to the Authority.

From time to time, beginning quite early in the history of the project, the Authority had occasion to make drastic criticism of FF & P even to the point of threatening to " terminate their consultancy ". While the early correspondence disclosed as much criticism of Wilson by the joint consultants, as there was criticism by Wilson of them, we feel that the fault in the main lay with the consultants, although Wilson, at times, may have been difficult. In one of Hindshaw's personal reports to London he described Wilson as being extremely antagonistic to FF & P claiming that the failure of WSC was due to lack of care and diligence by FF & P and threatening to sue them for $1,000,000. A perusal of the early correspondence shows that from the beginning down to a week or so before the disaster there was a continuous complaint by Wilson and resentment by FF & P.

It is impossible not to sympathize with Wilson, even if he sometimes displayed ill-temper and perhaps some intemperance of language, which was the result of frustration, at times amounting to despair. He was in a most unenviable position, with responsibility for a great and complex project which almost from the first encountered difficulties. Some degree of acerbity was well justified. Both the FF & P and JHC engineers on the site appear to have regarded Wilson as something of a " bogey man ", particularly Hindshaw, who claimed that Wilson had " torn strips off him " on one occasion, and plainly feared a repetition.

Having regard to the lack of progress, as well as many other defaults and errors made by the engineers and contractors the attitude of the Authority was quite understandable, and any complaints and criticism made of other parties stems from their faults and not from any unreasonableness of the Authority. Any client who sees his costly project being mishandled is entitled to complain in stringent terms, and on the whole we feel the Authority exercised considerable restraint.

There is one other aspect as to which the Authority merits some criticism. Fundamental to the whole sorry situation was the constant sense of urgency and pressure to complete the construction within specified times. No one can blame the Authority for a desire to keep its contractors up to schedule. The financial consequences of any delay were serious to an organization working on borrowed capital—and a degree of pressure to reach completion on time is understandable and even praiseworthy. Nevertheless, the determination to keep the work moving at all costs was so extreme as to engender an atmosphere in which speed was the all-important consideration. In a number of instances the burning desire for speed resulted in quick, ill-considered decisions which brought about trouble, difficulty and delay, whereas time spent in careful thought, and the adoption of what appeared to be the slower method of procedure would have, in the long run, meant an earlier, and happier result. We are satisfied that this climate of urgency and pressure tended to lower morale, and in fact directly caused some of the more serious errors of judgment upon which we have had occasion to comment.

It was put to us on behalf of JHC that the Authority was culpable in failing to disclose at the time of the negotiations for Contract E, that it entertained serious doubts as to the safety of the bridge design. By that time, Wilson was aware of the views of Hardenberg, that the structure could have been unsafe had not the extra strengthening suggested by WSC been adopted. He was still concerned about the adequacy of the design, to the point where he had sought a complete re-checking by FF & P. We consider that, when seeking to engage any contractor for the task of completing the erection, the fair and honest course would have been to have disclosed any doubts as to the adequacy of the design.

In this case, the new contractor was about to enter upon the work without the necessary information enabling him to make calculations of safety factors. Furthermore, the form of the contract did not require him to be responsible for such calculations—it was assumed that for these matters he must look to the consulting engineers and WSC. In these circumstances we feel that the Authority had at least a clear moral obligation to inform JHC fully of the doubts that were currently entertained and the failure to do so is very rightly a matter of serious complaint by JHC.

8.2.2. FF & P as Designers of the Bridge.

The design of the steel spans of the West Gate Bridge is of course a matter basic to the whole inquiry. The technical aspects have been dealt with in some detail, but it is also necessary to make some observations of a more general nature. A careful review of the evidence compels us to some conclusions which reflect little credit on the firm's performance as designers of this bridge. There can be no doubt that, in the past, Roberts established his outstanding ability as a designer of large bridges. At the time when West Gate Bridge was being designed, Roberts was nearly 70 years old, and, as his evidence shows, more than a little detached from modern structural theory and computer technology, both of which are desirable for a proper understanding of the stress analysis of a cable stayed box girder bridge. So far as the stress analysis is concerned and the subsequent proportioning of the sizes of members, the evidence shows that Roberts played little, if any, part in either directing or checking the work. By claiming to be the designer of the bridge, Roberts may have conveyed to the Authority and the contractors that the all-important work of stress analysis was in his, supposedly, capable and experienced hands. We feel that Roberts made an error of judgment in allowing himself to be put in this position of " designer " when he was no longer controlling the processes of design. Roberts made it clear that he entrusted the actual design to Brown, who had at his disposal a team of engineers to undertake the necessary calculations. We feel that Roberts' trust in Brown's ability to guide and direct the work in an effective manner, so as to result in a safe and efficient structure, was misplaced.

Brown, in his own evidence, stated that it was not his policy to give specific directions to the team working under him, neither was it his policy to have checked the individual work of the designers. He claimed that any control by the leader of the group would inhibit the freedom and initiative of the assistant engineers. He claimed also that checking of individual calculations was not necessary as there would be a multiplicity of parallel calculations leading to the same answer and that he could judge the rationality of that answer. The result was a disorganized agglomeration of calculations which contained many errors both in fundamental engineering principles and in simple arithmetic. Faced with some of these errors, while giving evidence, Brown repudiated the work of the assistant engineers, claiming that the calculations were done at a time when he was not responsible for the design. He freely admitted that some of the calculations were in error and stated that he would not have done them himself in that manner. Brown, himself, says that he was not involved in calculations for the bridge after July, 1970. Kerensky, on the other hand, says that after July the design was in the joint hands of Roberts, Brown and Kerensky. Kerensky stated that he did not at any time direct the details of calculation ; Roberts, by 1970, was retired, so either Brown was directing calculations from July, 1970 or no one was.

Some of the main assumptions on which Brown based his assessment of allowable stresses in panels of compression plating appear to us to be unjustified and such as to lead to a false and perhaps dangerously high estimate of allowable stress. Brown based his assumptions on the confident belief that all the longitudinally stiffened panels of upper flange plating were dished slightly downward on the bridge. He failed to instruct his own inspectors and the resident engineer that it was necessary to make sure that this condition was achieved. In fact, careful measurements made on the still standing steelwork on the east side show that the condition was not achieved. Expert witness was given, by others, that even if Brown's unrealized ideal had been achieved, his basic assumptions could still have led to a dangerous over-estimation of the allowable stresses. Brown admitted that the way in which the West Gate Bridge design had been handled was somewhat less effective than would be normal. He, himself, at the time, was involved in serveral other major projects for big bridges. We cannot accept Brown's involvement elsewhere as any excuse for the slipshod way in which the West Gate Bridge was designed.

Brown failed to give proper leadership and direction to his team ; he did not appear to be familiar with the details of much of the work which was in fact done. His own ideas on the structural behaviour of some of the elements in the West Gate Bridge are unsupported by current practice and contrary to modern theories of structural behaviour. He appears to have isolated himself from rational analysis and to have made decisions largely on an intuitive basis. We can only regard this as a highly dangerous situation, particularly, when dealing with a structure so complex as a cable stayed box girder bridge.

8.2.3. FF & P Performance on Site.

That the Authority should have found itself in conflict at one time or another with its consultants and contractors is perhaps unremarkable. It is far more surprising that a firm of the wide experience of FF & P should have been in constant disagreement, not only with the Authority, but with every one of the other parties, including Maunsell. Even the FF & P staff on the site had occasion to complain to the London office, chiefly on the ground that their urgent requests for instruction or information were answered only after long delays and repeated requests, or simply ignored.

One matter of controversy between FF & P and the Authority was the far from unreasonable demand by the latter for the presence in Melbourne of a senior partner of the firm. The Authority felt that FF & P should have had a senior member of the firm permanently in Melbourne while the steel erection was going on. Wilson was concerned that while the resident engineer and his staff were competent to carry out the routine work involved in building the bridge, they were not versed in the complexities of the design and so would not be in a position to handle with complete confidence any unusual condition which might arise. Time was to prove him right.

As we have already noted, early in 1970 the situation between FF & P and Wilson became so strained that Wilson threatened to terminate FF & P's appointment unless a more senior engineer was sent out.

At about this time JHC were taking over Contract E, with the consequent increase in responsibility for FF & P, and in any case the whole tempo of the steel erection was building up. After repeated requests, FF & P still did not send out a senior man who could handle an unusual situation where knowledge of the calculations would be essential. They did send out three more men, Hindshaw, Simpson and Ward.

The FF & P personnel on the site before this change were —

D. F. McIntosh	Resident Engineer.
P. J. F. Crossley	Deputy Resident Engineer.
R. K. Grieve N. Crook }	Assistant Engineers on Loan from Maunsell.

After April, 1970, McIntosh was made resident engineer for the fabrication and sub-assembly work only. He returned to U.K. in September, 1970.

During the hearing some pointed comments were made that FF & P had failed to call McIntosh as a witness. He was not in Australia at any time after September, 1970, but was still in the employ of FF & P. We feel quite certain that had his evidence tended to assist the cause of his employers, he would have been available as a witness. No explanation whatever was given for the failure to call him.

From the correspondence and the records which he left behind him, we take the view that McIntosh was a very sound engineer, who performed his work thoroughly and efficiently. Except for a somewhat vague statement that he had become *persona non grata* with Birkett and Wilson, we were offered no explanation as to why he was removed from his position as resident engineer.

We feel strongly that the evidence of McIntosh would have been of assistance to the Commission. The failure to call him does not permit us to draw any specific inference against FF & P but in issues where his evidence was important we feel entitled to take his absence into account.

The new team on the bridge after April, 1970 was—

J. Hindshaw	.. .. ..	Resident Engineer.
P. J. F. Crossley	.. ..	Deputy Resident Engineer.
C. V. J. Simpson	.. ..	Assistant Engineer on east side.
D. Ward	.. .. ..	Assistant Engineer on west side.

During the hearing there was much discussion as to the capacity and experience of these engineers, and we therefore regard it as necessary to set out their qualifications and experience.

Jack Hindshaw was born in England in 1922 and obtained his Diploma of Structural Engineering at the Constantine Technical College and became an M.I.C.E. From 1942–46 he served with the Royal Engineers in India and Burma and then was with Dorman Long & Co. mainly at Middlesbrough for fourteen years, working in their design office, shops and on sites. From 1960 onwards he worked for three different companies as agent or resident engineer mainly on concrete bridges, and with the Lancashire County Council as senior assistant bridge engineer, organizing the maintenance of various steel bridges. Prior to his appointment as resident engineer on the West Gate Bridge, he appears to have had little if any experience on site on the erection of any major steel bridge, either with consulting engineers or contractors.

P. J. F. Crossley, born in England in 1933, studied at Trinity Hall, Cambridge from 1954–57 and took his B.A. degree. Since then he served with FF & P mainly in their design office, although he spent a few years on the resident engineer's staff on the Forth Road Bridge (1960–62) and the Auckland Harbor Bridge Extension (1965–68). From 1968 until the collapse he was deputy resident engineer on the West Gate Bridge, working generally on the east side.

C. V. J. Simpson, graduated from Sheffield University with an Honours Degree in Civil Engineering in 1963. He then joined FF & P working in the design office under Brown on the Severn, Wye, Auckland Harbor, West Gate and other bridges. In 1967 he was awarded a Culmann Travelling Fellowship and studied cable stayed box girder bridges for two years in Germany and seven other countries in Western Europe. Since February, 1969 he was seconded to the contractor's staff on the Erskine Bridge and in April, 1970 took up his duties as section engineer for the east side on the West Gate Bridge.

David Ward, obtained the Higher National Certificate with endorsements in Civil Engineering and joined FF & P early in 1967 as junior engineer on the Auckland Harbor Bridge Extension, mainly on the foundation work and approach roads. He was 29 years old when in April, 1970 he took up his duties as section engineer on the west side of the West Gate Bridge. In 1971 Ward became a M.I.C.E. Neither he nor his opposite number Tracy, the JHC engineer on the west side had had any significant experience of steel bridge erection on site.

The resident engineer's office was on the east bank so that on that side there was a relatively strong team, Hindshaw, Crossley and Simpson.

Ward kept in almost daily touch with his superiors on the east bank but it was rare for Hindshaw to go over to the west side and see for himself what was going on. There is evidence that on one occasion seven weeks elapsed without Hindshaw visiting the west side and, according to Tracy's diary, even then he "got no further than the office".

We deprecate the situation created by the disposal of FF & P engineers on the east and west sides. The act of leaving Ward with his limited experience and authority in charge of the day-to-day work on the west side was in our opinion, unwise, unfair to Ward himself and not fulfilling the obligation of the consultants to the Authority. We are disturbed that Hindshaw as resident engineer did not visit the west side more frequently and make a personal check of all important parts of the work at regular intervals. The failure by FF & P to have a senior man permanently on the site was an important mistake, with serious consequences. The occasional visits of Roberts, Brown and Kerensky for a few days or a week at a time did nothing to remedy this omission.

Apart from matters of design, the most serious fault of FF & P so far as concerns the actual supervision of the work was a two-fold failure. Having failed to provide a senior engineer capable of fundamental professional decisions, the London partners left the relatively junior engineers on the site without sufficient communciation.

The presence of a partner on the site was said to be unnecessary as the juniors on the site were competent to handle day-to-day problems, and any more serious questions which arose could readily be referred to London for decision. In theory this was a reasonable plan. In fact it broke down because the advice or decision from London was just not forthcoming. There were very many letters and telex messages in the exhibits which explain that the partner dealing with the particular aspect was not available.

If the resident engineer had been properly and fully briefed as to his duties on site ; if he had been required to submit regular full fortnightly reports to London, setting out the progress made, any delays and reasons for them, condition of steelwork, lines, levels, &c., labour problems, any unexpected events ; and if each report had been replied to at once by a senior engineer in London, charged with responsibility for the contract—all the errors that were committed might well have been avoided.

Over and over again the site staff were left without proper instructions. This we find was a serious dereliction of the engineers' duty to their client.

8.2.4. THE JOINT CONSULTANTS.

Upon the appointment of Maunsell and FF & P as joint consultants, it was agreed between the two that Maunsell would be responsible for all contracts other than for the steel spans, FF & P to be responsible for the design of the steelwork, and to supply engineering staff to supervise and control construction. In respect of the steelwork, Maunsell were to act as spokesman for the joint consultants in Melbourne, and were to control all administrative matters while FF & P controlled the technical side. Thus the resident engineer for the steelwork reported to and was under the authority of Birkett, the senior Maunsell partner, for administrative purposes, but had direct access to London on technical matters. The scrutiny and approval of the contractor's working drawings and erection scheme drawings was the responsibility of FF & P in London. The necessary technical conferences were to be held with Werkspoor in London or Utrecht. No formal contract between the two consultants was ever executed.

Unhappily, the apparently inevitable disharmony and distrust which developed between the parties concerned in the bridge construction occurred as much between the two joint consultants as between any other two parties. The difference was that the Maunsell–FF & P dissension remained below the surface for the most part, until, at any rate this inquiry revealed it.

The origin of the dichotomy is hard to find. It probably arose from two causes. Firstly, Maunsell were subjected as much as, or more than, anyone else to the FF & P habit of not answering letters or telex messages and generally being the opposite to forthcoming with information quite legitimately sought. Secondly, we received a strong impression that there was a fundamental mutual distrust because of professional rivalry. Whatever be the reason for the disenchantment, there is no doubt that they both strongly criticized each other, though in the main such expressions were kept within the bounds of each organization.

Their difficulties were not helped by a tendency on the part of Birkett to hunt with the Authority and run with FF & P. An example of this occurs in connection with the pressure by the Authority on FF & P to have a senior partner permanently on site. Birkett, in discussion with the Authority appeared to agree with the request and, at least implicitly undertook to use his influence with FF & P to induce their compliance. However, in communicating with London he took the opposite view, not only agreeing that the proposal was unnecessary, but positively discouraging it, and claiming to have advised the Authority against it. In fact, Maunsell did not want a senior FF & P partner permanently in Melbourne.

One of the circumstances of some significance was the rather odd way in which Hindshaw communicated with Kerensky in London. Apart altogether from the meagre official reports he made, which went to the Authority, Maunsell and FF & P, he also wrote in his own handwriting a series of personal reports, of which six have been preserved, which went direct to Kerensky, and usually no further, although some were passed on to Brown. In addition to these written reports it was Hindshaw's custom to telephone Kerensky frequently. While on the face of it there was nothing very sinister in this reporting confidentially to a partner of his own firm, it is plain enough that Hindshaw was deliberately by-passing Maunsell. In some of the reports he claims bitterly of the actions of the Maunsell personnel. Typical of these complaints is an instance where he alleged that James, Fernie and Birkett spent so much time on the site as to make his presence almost unnecessary, and that they regarded resident engineers as " very small pawns ". He also added that the Maunsell people " were no friends of FF & P ". When questioned about this matter all three Maunsell men expressed surprise at Hindshaw's statements and completely rejected his allegations.

The knowledge that Hindshaw was in constant touch with London by phone probably led Birkett on two occasions to forbid Hindshaw to speak to London. On each of these occasions Hindshaw wanted to discuss responsibility with London, which was an administrative rather than a technical matter and so came within Birkett's jurisdiction and thereby entitled him to insist that he must not be by-passed in any discussions.

There are many other incidents revealing an unhappy and uneasy relationship between the joint consultants which it would be merely tedious to record. Sufficient to say that this disagreement added to the climate of confusion which prevailed throughout the period of construction.

8.2.5. JHC PERFORMANCE ON SITE.

When JHC took over Contract E their site team was as follows :—

T. R. Nixon	Project Manager.
T. V. Burbury	Section Engineer for east side.
W. F. Tracy	Section Engineer for west side.
J. S. Holland	Transport Engineer.

In addition, I. H. Miller served as assistant project manager from September, 1970.

J. Riggall acted as substitute for Tracy from September 7–21, when Tracy was on leave. Riggall was also section engineer on night shift in October, 1970.

T. R. Nixon, graduated in 1951 as B.C.E., Melbourne University and worked for six and a half years with the C.R.B. In 1958 he joined JHC as a construction engineer and worked as project manager on a number of contracts for roads and roadway bridges, &c. In 1963 he became Victorian branch manager and then served in positions of increasing responsibility for JHC, becoming general manager for the Southern Zone. From March to October, 1970 he was relieved of his duties as general manager and became project manager of Contract E.

It appears from his diary that Nixon was still required to attend numbers of meetings in Sydney, Hobart and elsewhere and also engineering symposia and other meetings. These frequent absences were a technical breach of JHC's contract. Although he was on site most days his main pre-occupation at West Gate was with the industrial troubles, in spite of the assistance of L. A. Plaxton as industrial officer.

The technical problems were left largely to Burbury and Tracy to settle with their opposite numbers Simpson and Ward. In view of the relative inexperience of these young section engineers, we feel that there was a big gap left through JHC's failure to have any senior engineer at all familiar with the erection of this kind of steelwork, spending full time on site directing the section engineers. This may well be the reason why so many of the faults on the east side, such as the failure to equalize the cambers of the half spans 14–15 before erection, were blindly followed on the west.

It had been intended that JHC should have the services of Hardenberg and Van Veldhuizen to fill this vacuum but in practice JHC seemed unenthusiastic about this idea and referred to them less and less. They also dispensed with the services of the two WSC advisers, Schut and Spee.

I. H. Miller was transferred by JHC from Contract C to Contract E as assistant project manager a few weeks before the collapse. JHC thus appeared to have realized there was a vacuum to be filled but Miller was not on Contract E long enough to make any significant impression.

T. V. Burbury, graduated in 1963 from the University of Tasmania with the Degree of B.E. Hons. He had worked on the pre-stressed concrete Tasman Bridge and with the Department of Main Roads, Sydney and joined JHC in April, 1968, working on Contract C until he was transferred to Contract E in March, 1970. He appears to have had little if any experience in the erection of structural steelwork.

W. F. Tracy graduated in 1964 from the University of Melbourne with the Degree of B.C.E. After five years experience mainly on water and sewerage schemes, dams, canals, &c., he joined JHC in March, 1968 on Contract E. In April, 1970 he was appointed section engineer for erection of steelwork on the west side of West Gate Bridge. He too had had little if any previous experience of steel erection.

J. S. Holland served as section engineer in charge of the transport of completed steel boxes from the fabricating shops after they were taken over from WSC, to wherever they were to be assembled on site.

There was lamentably poor output by the labour force under JHC, particularly on the west side. This was no doubt due in part to the industrial disputes which beset JHC from the commencement of their work on Contract E and which are dealt with in Section 9.1.2. However, in our opinion, some part of the trouble was caused by the inadequacy of the JHC staff, who were lacking in steel erection experience sufficient for the work they had to do on the contract.

8.2.6. FF & P AND JHC.

From the time JHC began work on the steel spans, there were constant arguments between the JHC management and the FF & P organization as well as between the consultants' engineers and the contractors' engineers on the site.

The more important of these are dealt with elsewhere in other sections.

It is however, necessary to mention the continual disputes between Hindshaw and the JHC engineers. Hindshaw's constant complaint was that the Holland men consistently departed from the agreed procedure, failed to seek advice from himself or his staff as they were required to do, and when advice was tendered, frequently ignored it. When pressed for details of these allegations during the hearing, the FF & P witnesses listed some eleven instances of such conduct by the JHC staff. Most of these we feel were trivial.

Typical of this sort of thing is the incident of box 12 south on the east side. Due to stop-work meetings, a late start was made on this operation, so that at the end of the day, it was not completed and the partially erected box had to be safely secured before leaving it. Burbury proposed a method for doing this, but Hindshaw would not give his approval and he and Simpson put forward counter proposals. Burbury persisted in his argument for about fifteen minutes but finally seeing he was getting nowhere said, in effect—

" Just tell me and we will do it whichever way you instruct ."

Hindshaw, having the issue and its attendant responsibility thrown back at him replied—

" Very well, do it whichever way you please."

He then left the bridge leaving Simpson and Burbury to work out a solution.

In July, 1970, Hindshaw made an attempt to rationalize the unusual relationship between consultant and contractor by suggesting that he and his team be absorbed by JHC for the duration of the job. In this way, Hindshaw and his team could have actively directed the work in conjunction with JHC engineers which they could not do while acting as resident engineers.

JHC thought this an unworkable and unsatisfactory arrangement, with which we agree. They rejected the proposal.

Hindshaw's reaction was to insist that JHC worked strictly to the agreed procedure as in the manual. There is no doubt that this unresolved question of responsibility was the source of much friction between Hindshaw and Nixon.

Hindshaw was not getting much help with his problem from his own principals, although there was talk of Kerensky coming out in November, 1970, when the problem would be resolved.

Hindshaw had many discussions on the problem with Birkett who was effectively Hindshaw's superior in the joint consulting engineer's organization. Birkett was in any case prepared to instruct Hindshaw on non-technical matters, what he was to do and what he was not to do.

The question of responsibility was still unresolved on 15th October, when the span 10–11 collapsed. Leaving this question unresolved considerably reduced the effectiveness of the resident engineer, Hindshaw, and endangered the whole project by tending to create an atmosphere in which co-operation between the engineers for the consultants and engineers for the contractor was difficult.

Several witnesses spoke of a state bordering on chaos prevailing on the west side. There is an entry in Hindshaw's diary to the same effect.

Morale was bad and the direction and organization were largely ineffective. There are reports of large groups of men wandering aimlessly around in the middle of the morning with nothing to do ; of other men attempting to perform impossible operations when trying to bring parts into line for bolting up.

Bolting up on that side appears to have been done without system, putting in bolts where they would go, with little regard for how later ones could be placed. Reaming of bolts was in consequence widespread. In some cases, the over-enlarging of the holes by reaming was so bad that special washer plates had to be provided under the bolt heads.

8.2.7. WSC—Competence in Fabrication of Boxes.

In order to fabricate the steelwork WSC built and equipped a fabrication shop on the bridge site. The shop was designed exclusively for the production of the panels for the West Gate Bridge and was to be demolished on completion of the contract. At the time the span 10–11 fell all panels had been fabricated for the entire bridge and the shop was " running down " ; the only work still in progress at the time was associated with the stiffening programme and the manufacture of non-standard cover plates and minor items.

Although the shop was not actually in production at the time the members of the Commission were able to inspect it, we were impressed by the well conceived layout of the plant and by the general quality of the equipment.

From our inspection of finished panels by that time built into the boxes, we concluded that the quality of fabrication was of high standard. Dimensional errors over the first and last holes in a panel appeared to be within tolerable limits. There were some inevitable departures from flatness of the panels but in general these distortions were acceptable although FF & P had not specified any quantitative permitted tolerances for flatness. Some mistakes in fabrication were made and at least one of these was not detected until after the bridge had fallen ; this was on the west side at inner upper panel 7 north where all five bulb flat stiffeners had been cut short by about 5¼ inches at one end necessitating extra long K-plates to bridge the gap. This was the panel which Prof. Murray used in his test G.

WSC remained in charge of all fabrication and sub-assembly work throughout, so that once JHC had taken over Contract E for the erection it was necessary to define a handing-over stage at which the boxes passed out of the control of WSC and became the responsibility of JHC. A close examination was made by JHC of all boxes at this " handing-over " stage. We are not altogether satisfied that the inspection was completely effective. There is reason to believe that at the time of handing over, some boxes were in an imperfect condition. One frequent fault was that the outer sloping web was kinked at the longitudinal welded butt, about two-thirds of the way up the web. There is some evidence to show that the unpainted surfaces at places where friction grip bolts were to be used had not always been kept clean. Another complaint was that loose material had been left in the boxes which might create a safety hazard during erection.

PART 9. INDUSTRIAL RELATIONS AND SAFETY.

9.1.1. Safety Precautions.

The safety of the structure during erection is considered elsewhere in this Report. This section is concerned with the safety of construction plant and of the personnel employed by JHC on Contract E from the time they took over steel erection in March, 1970.

(*a*) *Safety of Contractor's Plant and Equipment—Span* 10–11.

According to the evidence it appears that plant and equipment were regularly serviced and maintained. The lift was serviced weekly and was subject to annual checks by the Department of Labour and Industry. There is some evidence however, that the lift, which had capacity for fifteen men, used on occasion to be overloaded. Later its allowable capacity was reduced by Tracy to only ten men.

The stiff-legged derrick taken over from WSC was modified to conform with the D.L.I. Regulations. The temporary prop was modified to bring it in accordance with appropriate S.A.A. codes. The proposed erection tower climbing frame was planned on JHC's insistence to be constructed in accordance with the appropriate regulations. As regards power supply, advice was obtained from the local electrical authority on what was permissible in the reticulated electrcial power inside the steel boxes. Leak detector trip switches were installed on these reticulated systems.

A shortcoming of which the men complained was the failure to provide semi-circular guards over the access ladders to the bolting-up gantries beneath span 10–11. The men refused to use the gantries with the access ladders unguarded and, as a consequence, little if any bolting-up of the bottom flange longitudinal splice was done, except at the extreme ends, before the collapse. The evidence is somewhat conflicting on this point. It was fortunate that JHC gave priority to the extra stiffening needed in box 8, including cable anchorage connections which might well have been left till later. The available boilermakers were employed on this stiffening work and few if any were available for bolting-up the longitudinal splices.

(*b*) *Safety of Personnel.*

In his statement to the Commission, Nixon claimed that JHC's senior management had throughout its history emphasized to its staff the importance of safety awareness. In some twenty years of construction activities, JHC had never had a fatal accident on a bridge construction job before the West Gate tragedy.

His statement continues—

" For a number of years a safety officer, who co-ordinates safety training, has promulgated safety statistics and assisted in stimulating safety thinking on construction sites. A Safety Manual was prepared and is distributed to all of Holland's construction engineers.

In order to assist the construction staff to take proper note of the various aspects of the Safety Manual, lectures on safety are given from time to time, and safety statistics are prepared and promulgated. At the end of each quarter, site staff are given details of the safety result of the quarter just ended.

The company now has operating a Safety Award system in which every project seeks to win the Safety Award of the year made by the company ; this award is made on the basis of the best statistical safety record for the projects completed that year. Over the last few years the 'frequency rate' average throughout the projects in the company has decreased from 200 for year ending 30.6.63 to 86 for year ending 30.6.70.

(*a*) For work performed by Hollands on all the Westgate Bridge contracts, the following are the appropriate statistics up to 15th October, 1970 :—

Manhours worked	1,246,524
* Frequency Severity Index	8·04

N.B.—These figures exclude two accidents which occurred to employees whilst on their way home.

The Snowy Mountains Hydro-Electric Authority have for a number of years awarded their contractors a bonus payment for safe working.

For the purpose of participating in this bonus payment, the contractor must achieve a Frequency Severity Index less than 12·0.

(*b*) For Contract E, prior to 15th October, 1970, the relevant statistics are—

Manhours worked	110,187
Frequency Severity Index	7·20

Eleven lost-time accidents are included in the above figure, of which seven occurred on the west side and four on the east side ".

* (A statistical measure of the frequency and severity of accidents).

JHC appointed full-time first-aid officers on site—one on each side of the river. Each of these operated from a properly equipped first-aid centre where safety notices and safety posters were promulgated.

Three men were also employed full time to clean the sheds, to arrange lunches and to keep the site tidy and ensure that all access ways were kept clear of obstruction.

Nixon said that in the last year or so three foremen and three engineers from the staff on Contract E had attended an external safety course. During the last month or two before the collapse regular safety committee meetings were held on site. These were chaired by the section engineers and attended by representatives from each union and the men.

There was no full-time safety officer employed on site. Nixon felt that there was a danger if that was done, that the rest of the staff might not pay proper attention to safety but might leave it all to the safety officer. He regarded it as his responsibility to see that adequate precautions were taken. This is somewhat in conformity with the practice adopted on recent major bridge contracts in the U.K., where the contractor's management appointed a number of their senior engineers and foremen as safety supervisors, in addition to their other duties, with very good results.

The wearing of safety helmets on site was a condition of employment. The unions and most men co-operated in seeing that this condition was observed, but Enness, one of FF & P's inspectors stated that although two of JHC's men on the east side would not wear safety helmets, they were nevertheless allowed to continue to work on the job. In view of the fact that the wearing of safety helmets was a condition of employment we consider that refusal to do so should not have been permitted.

Safety nets were available and had been used on Contract C but Nixon said there had been no need for them on Contract E. Safety belts and harnesses were also available for any men who might require them.

For many years it has been accepted practice, especially on major bridges, to locate men's huts, such as mess sheds, stores, offices, toilets, &c., on the steelwork, i.e., on the deck or towers of a bridge, so as to have them as readily available for the men as possible and save long journeys between the actual place of work and the huts. In this way the huts are located where nothing can fall on them and they should be safe.

It is less usual to locate huts beneath the steelwork, principally because of the risk of objects falling on to them. On a small congested site it may be difficult to avoid ; but it should not be done on any account unless adequate precautions are taken. No one ever imagines that the steelwork of the bridge is going to collapse and that is not a factor that is taken into account in the positioning of the huts. The tragedy in this case was that there were men in the huts below the bridge when it fell and this contributed to the high death toll.

After the two half spans 10–11 had been lifted to the top of the piers, 5 mess sheds, 3 offices, 2 stores, a changing shed and toilets were put up on top of the deck. Three changing sheds, 2 stores and a time clock, ablutions hut and first-aid hut were located beneath the span near pier 10 where they could be reached on dry ground.

Of the huts beneath the span, the ablutions hut and the first-aid hut were close to the centre line ; and two of the changing sheds, the hydraulic store and the time clock were near to the northern edge of the span.

There was a gap about 7 inches wide on the longitudinal centre line between the steelwork of the two half spans. Nixon stated that for the length over land, about one-third of the span, this was " reasonably well covered in ". However, according to Halsall, a JHC rigger, a 12-ft. long scaffold plank fell from the top deck right through the gap and landed about 12 feet away from the first-aid shed. He said that the gap was only covered in by an 8-ft. long piece of plywood on either side of each diaphragm. This would be necessary because men had to get through the access manhole in the diaphragm which was on the centre line over the gap. Halsall said that the handle of a Tangye jack and a few nuts and bolts also fell through to the ground. There was no evidence of any objects having fallen over the outside edges of span 10–11 where there were toe boards and handrails fixed.

In our opinion, the longitudinal gap between the two half spans should have been securely covered throughout its length over land, particularly in view of the fact that there were a first-aid shed and other huts beneath the span. As a general rule, huts should never be located beneath steelwork on which work is proceeding ; if this should be unavoidable on a very congested site, then the huts and their access must be completely protected from falling objects.

9.1.2. Industrial Relations.

When this Commission first convened on 28th October, 1970, Mr. Marks, Q.C. made an application for leave to appear on behalf of the Victorian Trades Hall Council, which represented the seven unions whose members were working on the contract at the time of the collapse.

After being granted leave to appear, Mr. Marks made an application for the Chairman of the Commission to request an extension of the terms of reference to include matters involving the safety of the men who were working there at the time.

The Commission undertook to consider the application and upon the commencement of evidence on 5th November, 1970, ruled that the terms of reference as they existed were wide enough to cover the matters raised by Mr. Marks but agreed that the application could be renewed if considered necessary.

In fact, although given leave to appear, no representative of the Victorian Trades Hall Council appeared after the preliminary sitting. Consequently the Commission did not have the assistance of counsel representing the unions, nor was any witness called who appeared in the character of union representative. In these circumstances we are conscious that on the matter of industrial relations we have only heard one side. Nevertheless the uncontradicted evidence placed before us enables us to make some findings as to which we are entirely satisfied.

(*a*) WSC.

On the fabrication and shop assembly of the steel boxes for West Gate Bridge, WSC employed men belonging to unions of the Metal Trades Federation, i.e.—

Boilermakers and Blacksmiths Society of Australia.
Federated Iron Workers Association of Australia.
Amalgamated Engineering Union.
Australian Society of Engineers.
Federated Engine Drivers and Firemens Association.

In the early stages of the contract when fabrication only was proceeding, WSC had virtually no labour problems. Once they started steel erection on site, however, employing the Metal Trades Unions as before, they ran into severe trouble, of a kind that they had never encountered before during twenty years' work in Australia. Continual demarcation and other disputes developed, the work performed was appallingly slow and the situation continued to deteriorate both in the shop and on site.

It was as though every opportunity was grasped to delay the completion of the job.

The view was expressed that this state of affairs was due to certain elements who wished it to appear that the Metal Trades Union members were not capable of satisfactorily carrying out the work of steel bridge construction. It was certainly argued that WSC would have to employ members of the Builders Labourers Union for the performance of certain aspects of steelwork erection. After the hearing of a demarcation dispute in March, 1969, a ruling was given that WSC could employ any type of labour as thought fit. However, this ruling did not prevent the continuance of delays caused by labour problems.

These industrial troubles continued right up until the changeover in March, 1970, and after that WSC's labour problems suddenly ceased. At that time many of the men they had employed on the West Gate site were transferred to other projects. Thirty-four men were retrenched and paid a week's wages in lieu of notice. WSC carried out no more steel erection, which had been handed over to JHC, but continued only on the fabrication and sub-assembly of boxes. It is interesting to note that their production in the shop, once the labour troubles stopped, increased twofold. There were no more strikes. This was not due to any change in WSC's system of work or supervision, but apparently simply because the cause for the earlier delays, i.e., the fact that WSC had been using metal trades men for steelwork erection on site, had stopped.

(*b*) JHC.

When they started work after the changeover, it was naturally JHC's intention to employ members of the Builders Labourers Union, whom they knew and with whom they had always worked before, for the steelwork erection. Nixon said he would like to have started with a small nucleus of tried and trusted men and to build up from there. Contractors generally prefer to employ members of as few different unions as possible because that reduces the risk of disputes and stoppages due to inter-union rivalry, or demarcation issues. It also means that all the men are working or none of them are and prevents the job from being held up by a few essential men, such as crane drivers, being on strike and no one else.

A major difficulty had arisen, however, over the 34 men dismissed by WSC, 22 of whom had not yet found other work. A site meeting was held on 18th March, 1970 with the Metal Trades Union to whom these men belonged. This meeting is reported in James' diary. It was attended by Wilson, Nixon, James, Hill, A.E.U.; O'Neall, Boilermakers ; Hollowell, Ironworkers ; McDonald, F.E.D.F.A. ; Morgan, A.S.E. ; together with job delegates from these unions.

At this meeting considerable pressure was put on Nixon to engage these 22 men at once amongst the 100 or so JHC would need for the work, WSC had apparently said that the 22 men were satisfactory ; although JHC must have gravely doubted this, in view of the fact that WSC were disposing of them and it certainly was not James' view.

Nixon was threatened that if the 22 men were not taken on at once by JHC, the unions would put a black ban on all the work, not only on Contract E but also on Contract C.

After a short adjournment, the unions agreed with Wilson's suggestion that the Authority would pay an extra week's " severence pay " to all the men concerned and JHC would inform them by Monday, 23rd March what they were going to do. The outcome was that JHC took on the 22 men. Soon after that Nixon received an ultimatum from the Builders Labourers Union to say that erection had to be done by their union otherwise the job would stop. It is not surprising that under these impossible conditions JHC suffered from continual labour trouble, strikes, stoppages, go-slows, disputes, absenteeism and very little work was done. Appendix F gives a note of the stoppages from 13.4.70 to 14.8.70. In addition, there were fifteen stoppages in September, 1970, and others in October. This degree of labour disruption was quite strange for JHC because for many years they had enjoyed good labour relations and on Contracts C and F there was very little industrial trouble and not much delay.

There is no doubt that if JHC had refused to take on these men they would certainly have been faced with many weeks of stalemate, a situation which neither they nor the Authority wanted. In retrospect, however, the latter course may well have been the better choice. It would have enabled JHC to start the steel erection work in a manner of their own choosing and not under dictation.

Faced with this situation Nixon had to spend a great deal of his time dealing with labour disputes. He was assisted in this by Mr. L. A. Plaxton, an industrial officer from Metal Trades Employers Association who was seconded to JHC for this project.

JHC worked a 56-hour week on Contract E, made up of four ten-hour days and two eight-hour days. Sunday was a holiday and on Saturdays the men were paid for the first four hours at time and a half and thereafter at double time. The weekly gross wage for metal trades men on a 56-hour week varied from $134.31 to $112.31 according to the trade. In addition, a man might earn about $5.00 height money. Men on night shift usually worked 48 hours instead of 56 hours, but the rate was increased by 25 per cent.

Nixon gave evidence that tradesmen under the builders trades, as distinct from the metal trades, would receive a slightly higher wage, because they are on hourly hire as distinct from weekly hire.

The total number of men employed by JHC in October, 1970, was about 120, of whom 100 worked on the daily shift ; the other 20 worked a night shift on the west side. Of the day shift men, 40 were employed on the west side and 40 on the east. The remaining 20 formed a ground crew engaged on the transport of boxes, and worked mainly on the east side, but to some extent on both.

Some extraordinary facts emerged from the evidence given by Nixon. For example, when the men were " cabined-up " in their huts and could not work outside because it was raining they were paid the full rate. There was thus no incentive to make them walk a few yards in the rain to enable them to work under cover inside the boxes. On other contracts a reduced rate is paid for hours when men are necessarily " cabined-up " and no work under cover can be found for them.

Another example was the very high rate of absenteeism of which Nixon complained. He said that JHC attempted to stop this by not permitting any man to work overtime unless he had worked for the full week without being away at all. A resolution was passed by the men, however, " All in or none in ". This meant that if JHC said to one man " You were not here during the week and you cannot therefore work on Saturday " the whole lot of them went off.

Nixon stated subsequently that although 28 per cent. of the available time was lost through industrial disturbances, the effective time lost was far greater having regard to loss in morale and general lack of application and enthusiasm which resulted from such an industrial climate. In an endeavour to improve progress JHC took on more men and started a night shift.

In spite of a signed agreement with the unions and men setting out agreed rates of pay there was agitation for a $20 a week increase in wages. Nixon did not feel this was justified but finally after consultation with the Authority and the consulting engineers, a type of " termination allowance ", payable on completion of the job, was put to the men and incorporated in an agreement.

This " termination allowance " of $50 per month would be payable on completion of the contract, but it was subject to certain conditions relating to good behaviour. If the men, for example, had an unauthorized stop-work meeting for that month they received nothing. If they were absent for more than a few hours without very good reason they would forfeit the " termination allowance " for that month.

By early October, 1970, after six and a half months of Contract E, Nixon felt that JHC had " turned the corner ". The original 22 men taken on after the initial wrangle, had been reduced to ten and some good and willing men anxious to get on with the job had been recruited. Moreover, the " termination allowance " had been finalized in a way that gave JHC a strong hand in dealing with future troublemakers. There were three successive days' work without an industrial stoppage and Nixon anticipated that the industrial climate would thereafter show significant and continuing improvement.

The six months of industrial sabotage on the job, however, had a crippling effect on the construction of span 10–11, from which it never recovered. With the working time cut to 50 per cent. by industrial disputes, absenteeism, go-slows, and bad weather, it is doubtful whether even as much as a quarter of the work was done that would have been achieved given a reasonable industrial climate.

The men refused to use the two gantries beneath span 10–11 until the ladder access to them was improved. This was not done and the gantries remained unused. This made it impossible to complete the longitudinal splice in the bottom flange, except in the end boxes. Only half the upper longitudinal flange splice was completed. This considerably reduced the strength of the span and its resistance to failure, compared with its strength if these upper and lower joints had both been completed. The action of the trade unions and the men and JHC's failure properly to control the labour retarded the work and undoubtedly contributed to the weakness of the span at the relevant time and so to the ultimate collapse.

It is widely accepted that the essential requirements for good labour relations are mutual trust, confidence and respect as between management, trade unions and men. Once this relationship is established, all concerned will work as a team and first-class production can be achieved. Without it, little if any progress can be made.

By their actions in compelling JHC to engage men in whom they had no confidence and to run the job in a manner not of JHC's choosing, the trade unions and men must accept their share of responsibility for the tragedy that ensued.

PART 10. RESPONSIBILITY OF THE PARTIES.

10.1.1. General.

The disaster which occurred at noon on the 15th October, 1970 and the tragedy of the 35 deaths was utterly unnecessary. That it should have been allowed to happen was inexcusable. There was no sudden onslaught of natural forces, no unexpected failure of new or untested material.

The reasons for the collapse of span 10–11 are to be found in the acts and omissions of those entrusted with building a bridge of a new and highly sophisticated design.

The various companies who supplied the materials used were not shown to be in any way at fault, and must be held blameless. However, among those engaged upon the design and construction of the steel spans there were mistakes, miscalculations, errors of judgment, failure of communication and sheer inefficiency. In greater or less degree, the Authority itself, the designers, the contractors, even the labour engaged in the work, must all take some part of the blame. Error begat error, and the events which led to the disaster moved with the inevitability of a Greek tragedy.

In our consideration of the responsibility of the various parties, we have been careful to avoid as much as humanly possible, the effect of hindsight. We have tried to judge the conduct of those engaged in the project as best we can in the circumstances facing them at the time of the act or decision which we are considering, without allowing our judgment to be affected by our knowledge of subsequent events. In judging the various issues we have not applied any high standard of perfection, but have taken as our yardstick what we believe to be a standard of reasonable competence to be expected from men holding themselves out as competent professionals. It is by the application of this moderate test that we have found it necessary to make a number of criticisms of those engaged on the project.

Engineers engaged on the design of major bridges cannot stand still. It is part of their duty, not only to their clients but to the community as a whole, to advance, to develop new concepts of design, to adopt new methods of calculation such as the computer and to encourage the production and use of improved materials, as high tensile steels and pre-stressed concrete.

These advances have led to ever-increasing spans of bridges, to lighter and more economical structures, and novel methods of design to assist in combating the forces of nature. FF & P have undoubtedly been in the lead in the design of great bridges, as witness the entirely new conceptions in their design of the Severn Bridge—achievements which won them the distinction, against international competition, of being selected to design the long span Bosphorus Bridge to connect Europe with Asia.

It is however necessary to emphasize that when leading designers are working as pioneers, only just within the bounds of the engineer's knowledge, some slight misjudgment, or failure to appreciate every aspect of a new problem may prove disastrous and bring tragic and fatal results. Under these conditions, it is more than ever essential to employ really adequate margins of safety and to ensure that they are not eroded by various unexpected and accidental factors, including of course, imponderables and human fallibility.

Within less than twelve months there occurred three failures of box girder bridges in course of erection, the Fourth Danube Bridge, Vienna in November, 1969 ; the Milford Haven Bridge in June, 1970 and the West Gate Bridge in October, 1970. In these bridges, factors of safety of from 25 per cent. to 31 per cent. were intended during erection. In each case, however, this margin of safety was eroded by a number of accidental factors which were not foreseen—including under-estimate of dead load, locked-up welding stresses, effects of eccentricity and distortion, temperature differentials, fabrication and erection errors, and the like.

We understand from Grassl's evidence that the factor of safety has now been increased to 50 per cent. for future bridges of this type in Austria. This appears to us to be a wise precaution until more is known about the recondite complexities of this type of design.

10.1.2. The Authority.

The Authority approached its task with full understanding that it had undertaken to construct an important link in Melbourne's road system, which was expected to be a landmark and an attractive feature of the city's environment.

Conceived as a structure of world standard, there was no thought of cutting down on effectiveness or appearance by way of any false economy. The intention was to engage the best professional advice, the best workmanship and to use only first-class materials. With the failure of King's Bridge in 1962 still fresh in everyone's memory, there was a determination to avoid the mistakes then made and the Authority chose its professional advisers, and with their assistance its contractors, with great and anxious care.

The project as a whole fulfilled the promise of its beginning, but in respect to the vital steel spans the bright beginning faded to a pitiful result. For this, as will have appeared from earlier sections, the Authority must share some of the blame.

We are satisfied that the decision to accept the advice of the joint consultants and relieve WSC from the erection portion of Contract S was reasonable at the time it was made, but we believe the necessity for this decision could have been avoided. (8.1.2.)

Once it became necessary to select a substitute contractor, the Authotity was greatly influenced by the urgency to complete a contract which was badly behind schedule, with as little further delay as possible. In the circumstances then existing, we conclude that the appointment of JHC was reasonable. (8.1.3.).

It was plain to the Authority that JHC was inexperienced in the type of steel construction required by the contract, and lacked the necessary special expertise, and that for the reasons set out in Section 8.1.3., it was only possible to employ JHC on a basis which limited that company's liability and responsibility. An attempt was made to mitigate the effect of this situation, by taking steps to see that the inexperience of JHC was counter-balanced by FF & P accepting an increased responsibility and by an effective strengthening of JHC's professional staff by the addition of experienced engineers, but this was by no means successful.

Having agreed with FF & P that this firm should assume additional responsibilities and duties upon the appointment of JHC, the Authority fell into grave error by failing clearly to define the respective areas of responsibility and function of the two organizations. This omission led to a situation in which neither the consultants nor the contractors were likely to give of their best efforts, and in fact failed to do so. (8.1.4.).

At the time of negotiation with JHC to undertake the work of Contract E the Authority already entertained serious doubts as to the safety of the bridge design, and particularly as to adequacy of the structure during the process of erection. It is our view that in fairness to JHC, a disclosure of these matters should have been made to the new contractor. (8.2.1.).

The contract between the Authority and the joint consultants provided for inspection by the Authority. We feel that this right of the client should have been exercised more frequently and more sensibly. Regular and careful inspection by the Authority with consequent comments to the joint consultants might well have had very beneficial results. (8.2.1.).

Closely related to the failure to make adequate inspection is a failure to insist upon proper detailed periodical reports from the joint consultants. A perusal of the reports in fact furnished reveals that they were perfunctory and not reasonably informative. Some very important and significant matters that should have been included in these reports were conspicuously missing. As examples, no reference can be found to the difficulties encountered arising from differences in camber of the two halves of span 14–15 or span 10–11, or to the bulge in either span, or to the use of kentledge on span 10–11. While the primary fault was that of the joint consultants, we feel that the inadequacy of the reports was so obvious that much better ones should have been insisted upon by the Authority. As it was, some incidents of serious import were never known to the Authority, or were only discovered fortuitously and too late for any effective action.

It may be that the original estimate of the time to be allowed for completion of the project was not long enough, thus enforcing a schedule which was too tight. Be that as it may, the delays caused by inadequacies of the contractors soon created a situation where a constant feeling of pressure to speed the lagging programme became oppressively evident.

This pressure situation was increased enormously by the constant stoppages by the unions and the men, arising often enough from unjustified causes. The atmosphere of urgency was not the fault of the Authority, but it is unhappily true, that at times it, as well as the other parties, permitted judgment to be influenced by the prevailing sense of pressure, resulting in ill-considered decisions and to the kind of mistakes which all too often arise from hasty actions. (8.2.1.).

10.1.3. WSC.

This company, backed by its parent companies, Werkspoor & Wescon, was thought to have a substantial reservoir of professional skill, experience and expertise, and its performance was expected to be first class on this project. It may be that those back in the Netherlands with whom the ultimate control rested, did not give to this contract the care and attention it deserved, but whatever the reason, we are compelled to conclude that the group's performance on West Gate, in many respects, fell far short of ordinary competence.

It is necessary to go back to the WSC tender to find what was to prove the most important factor leading to the failure of span 10–11. As discussed in Section 3.3.1., unlike the Australian practice, many English and Continental designers prefer to leave the erection method entirely to the discretion of the contractors. We take leave to doubt whether this practice is so universal as suggested. In any event, what may be suitable for Eurpoe, where there are no doubt many contractors of long experience, does not appear desirable for Australian conditions. Be that as it may, FF & P adopted this practice in their tender documents, in marked contrast to Maunsell's tender documents for the other bridge contracts, where the erection method was prescribed in detail. The tenderer was thus left at large to decide its own erection method, and for spans 10–11 and 14–15 WSC chose the method described in 2.1.1.—that is, of assembling each span on the ground in two halves divided longitudinally, and subsequently raising each half separately to the top of the piers where they would be joined. The adoption of this unusual method could only be justified if the greatest care and thought was thereafter applied to the process of erection. In this regard both the performance of WSC and the supervision of the joint consultants failed to attain a sufficiently high standard of care.

The performance of WSC after the work commenced, has been adverted to in Sections 8.1.2 and 8.2.7. It is plain enough that the company failed to place in control of the operation anybody with the ability to manage the labour situation. It is true that the union members behaved irresponsibly and much of the industrial strife was unavoidable by WSC, but we are firmly of the opinion that far more could have been done by its management to improve the situation.

The inordinate delay in the work of Contract S was not entirley the fault of the company and admittedly it encountered difficulties of various kinds not all of its own making. We nevertheless believe that a more efficient management could have done a great deal to mitigate the situation. We believe that the organization, particularly as to lower-echelon supervision, was inadequate. There were insufficient foremen, and the organization of the work was inefficient.

We are satisfied that during assembly of the boxes for span 14–15 WSC failed to adopt proper procedures to ensure that the differences in camber between the north and south half girders were reduced to a manageable minimum. (2.1.3.).

Basic to the difficulties experienced in having too great a camber difference, was the failure of the contractors to take adequate steps to measure the camber during assembly on the ground. Although Hardenberg claimed that the measurements were made when there was no difference in temperature between top and bottom flanges, either at dawn or on a day when there was an overcast sky, other evidence did not support this. We believe that errors in the measuring of camber did occur due to temperature effects, which were not properly taken into consideration.

Essential to a successful result, using the method of erecting in half boxes, was that the boxes be adequately stiffened on the outer edges of the flange plates. Such stiffening was completely inadequate on the first half span of 14–15, with the result that buckles formed in the steelwork. This consequence should have been foreseen and adequate stiffening provided. This is but one example of the absence of that special care and foresight which should have been exercised in the use of this method of erection.

Another matter on which we feel bound to make some criticism of WSC—more particularly of Hardenberg—relates to the request by JHC for information on erection stresses. The background to this matter is that WSC had the only relevant calculations in Melbourne. Early in September, 1970, Rugless had asked Hardenberg whether he could verify the expected stresses when cantilevering box 12 on the east side, bearing in mind that there was a buckle in box 9.

Hardenberg was embarrassed because he felt that he was being asked for information behind the back of Hindshaw—though why it could not have been given with Hindshaw's full knowledge is hard to understand in the light of the contractual obligations on JHC to seek, and WSC to supply, advice of this nature. Hardenberg put Rugless off by saying he would talk with Nixon. This he did, and asked Nixon to write to him.

On 14th September, 1970, Nixon wrote to Hardenberg " in his capacity as adviser on erection method and technique ", seeking his opinion on " the adequacy of the structure in its present condition to properly withstand erection stresses ". Nixon also wrote to the joint consultants.

On the 16th, Holland himself, at a site meeting, made a forthright request for assurances on safety margins generally. As a result these were given to him by the joint consultants.

Hardenberg replied to Nixon's letter, also on 16th September to the effect that he had examined the condition of the erected steelwork in particular the outer web of the south side, and had concluded that " the structure is quite adequate to allow erection of box 12 ".

The effect of Hardenberg's letter, combined with the assurance of the joint consultants, was to give JHC a false sense of security. Had Hardenberg refused the assurances sought from him JHC might well have declined to proceed further with the erection.

Hardenberg himself had doubts concerning various aspects of the design which he had raised with Hindshaw and Crossley, suggesting that they have these doubts resolved by London. In the light of these doubts, and of what we now know of the calculations of stresses made up to that time, and of what the condition of the structure in fact was, we are satisfied that Hardenberg had no justification for the reassuring opinion he expressed.

10.1.4. Maunsell.

The Australian firm of Maunsell & Partners, which grew from the English firm of G. Maunsell & Partners, has continued to enjoy the reputation for ability and efficiency originally attached to the parent firm.

Maunsell have acted successfully as consultants in many important engineering projects in Australia, which were primarily reinforced concrete structures. Maunsell's designs for the foundation of the West Gate Bridge and for the approach spans were highly satisfactory and Contracts F and C were executed efficiently, to the satisfaction of the Authority and within reasonable time.

Maunsell were ready with advice to the Authority on the subject of the removal of WSC from part of Contract S, but in giving advice, failed to appreciate that to some extent errors and omissions by FF & P contributed to the delay on the part of WSC. This failure to apprehend the true position was no doubt due in some measure to the arrangement between the joint consultants whereby FF & P were responsible for the supervision as well as the design of the steelwork, which only came within Maunsell's jurisdiction for purposes of administration. However, we believe Birkett should have realized the shortcomings of FF & P in their dealings with WSC, and made further efforts to persuade FF & P to adopt a more co-operative attitude, both by providing the promised calculations and in being more sympathetic towards any legitimate claims for extension of time. Had this been done early enough and some financial adjustment offered to WSC, the need to change contractors might have been avoided.

Birkett was concerned to quite a considerable extent with the decision of the Authority to appoint JHC as substitute contractors. The observations we have made relative to this decision of the Authority must therefore, apply to Maunsell in their capacity as its advisers. No doubt the considerations to which we have already adverted, led Maunsell to advise in favour of the proposal, and for the reasons we have given in judging the Authority's action, we accept that the advice tendered was in good faith and reasonable. (8.1.3.).

We appreciate the difficult position in which Birkett particularly, and the Maunsell staff in general, found themselves, as being the intermediary between FF & P on the one hand and the Authority, WSC and JHC on the other. At times, there must have been a conflict of loyalties, which in the main was resolved fairly enough, although we do not depart from our observations in Section 8.2.4. to the effect that there was some distrust by Maunsell of FF & P considered as possible professional rivals, and a tendency on Birkett's part to present one face to the Authority and another to FF & P.

We do criticize Maunsell, in relation to the request by Wilson that FF & P provide a senior officer, a partner or someone almost on that level, permanently to supervise the work on site. This is something FF & P should have done and insofar as Birkett encouraged Roberts and the other London partners to resist this very reasonable request, we feel that the duty toward the Authority was disregarded, because of the unwillingness of Maunsell to have such a senior FF & P man in Australia.

We are not prepared to accept Hindshaw's allegations that Birkett, James and Fernie were too much on the site, or that they regarded Hindshaw and his assistants with disdain. We accept the denial of all three on this aspect. After considering carefully the allegations of Hindshaw against Maunsell's engineers who were concerned with the site management, we ultimately concluded that whatever Birkett, Fernie and James did was wrong in Hindshaw's eyes. He complained both that they left him without adequate support and that they were too concerned to supervise and direct him. In the tragic circumstances that Hindshaw has perished in the disaster, and we were therefore without the assistance of his evidence, we hesitate to make any findings on this aspect, but can at least say we are far from persuaded that these allegations had any real substance.

There having been very little direct conflict of evidence in this inquiry, we have not found it either necessary or desirable to say anything of the credibility of individual witnesses, except in one or two instances. We feel however, that it is only fair to say that we found James a particularly impressive witness, and where his evidence conflicts with others, we are disposed to accept his account. James' diary, which was never intended for the light of day and was therefore frank to the point of embarrassment, we found to be a most useful document from which we derived a great deal of information.

Hindshaw's allegations that Birkett had forbidden and prevented him from telephoning his head office in London, in substance admitted by Birkett, we at first found rather an extraordinary attitude, but Birkett's explanation of this matter was reasonable and satisfactory to us. (8.2.4.).

By about September, 1970, serious doubts as to the adequacy of the structure of the bridge as regards stresses during erection, and the safety factor generally, had arisen in the minds of Wilson and of the JHC engineers. The matter had been raised by the Authority, by Nixon in a letter of the 16th September, and by Holland personally at a site meeting on that date. On the 18th September, a letter was written on Maunsell paper and sent to JHC although unsigned and headed " First Draft ".

In this document there is an unequivocal statement as follows :—

" 1. *Safety and Adequacy of Design.*

Following the change in site management and the collapse of a bridge in Milford Haven, Freeman, Fox and ourselves have re-examined the Lower Yarra design and have strengthened certain panels in order to increase safety margins and to provide a " belt and braces " approach to critical members during erection. We are therefore able to give a categoric assurance that provided the laid-down procedures are properly followed the bridge design is fully adequate during all stages of erection. It should be noted however, that this is probably the biggest bridge of its type in the world, and as such its successful execution does require skill and care during building. However, I am fully confident that the site staff headed on our side by Jack Hindshaw and on yours by Trevor Nixon are fully equal to the task."

Again as in the case of Hardenberg (Section 10.1.3.) we say that such a categorical statement was not justified. No sufficient calculations had been carried out to support any such firm opinion by the joint consultants, and the structure was not in fact " fully adequate during all stages of erection ". As this assurance to JHC was also in substance given to the Authority, and was for use among other things, to pacify and allay the suspicions of the labour unions and their members, on the matter of working safety, we find the undertaking of the joint consultants to have been given without any proper foundation for the expressed belief. This we can only categorize as improper, and in breach of their duty to the Authority, the contractor, and the workmen engaged on the project.

10.1.5. JHC.

As in the case of other parties, the JHC group bears a first-class reputation as an efficient organization, and has competed in the Australian construction industry for some years with marked success, being responsible for a number of important structures, mainly in reinforced concrete. It had experience in structural steel to a limited extent only. The work performed as the contractor for Contracts F and C on the West Gate project was entirely satisfactory and although these contracts fell somewhat behind schedule this was not a serious delay.

The relations between JHC and the labour unions had been good on these contracts, although there had been some difficulties. The company management had an excellent reputation for good labour relations, and were rightly proud of the extremely good record for safety.

The performance of JHC on Contracts F and C undoubtedly impressed the Authority, as it might well have done, and this was a principal factor in deciding to offer to JHC that part of Contract S from which WSC had been relieved. We have discussed the wisdom of this decision by the Authority (8.1.3.) and it now remains to look at the reverse side of the coin and consider whether JHC should have sought or accepted the contract.

That JHC lacked the special expertise necessary for the work involved in this contract, was manifestly plain to the Authority and the consultants, and indeed was clearly stated by Holland himself without equivocation. Doubtless, the company management quite honestly believed that their general experience as contractors for important structures was amply sufficient to enable them to deal successfully with this project, so long as the expertise which they lacked would be provided to them from some other source. This must also have been the view of the Authority and of the joint consultants, because the contract ultimately offered to JHC was a " labour and management " contract. This imposed responsibility for the physical task of assembly and erection of the boxes, plus the control of labour, but did not require the company to rely upon its own personnel for the sophisticated calculations and highly technical decisions necessary to ensure the safe and satisfactory completion of the contract.

Looking at the matter from the viewpoint of the JHC management, having regard to the limitations of function agreed upon, and without being influenced by hindsight, we cannot see any reason to blame JHC for deciding at the time to accept what must have appeared as a challenge reasonably within their capacity.

Having accepted the reasonableness of JHC undertaking Contract E, we must emphasize that this view is based on the assumption that they would in fact take advantage of the arrangement making available the advice of senior officers of WSC, and would accept the services of WSC personnel incorporated into their staff, as they were contractually bound to do.

We take the view that as the work progressed, the JHC management became over-confident. They ceased to seek or follow advice from Hardenberg and dispensed with the services of Spee and Schot. Hindshaw's words—" they knew it all ", appear to sum up, not unfairly, the JHC attitude.

This course of conduct resulted in the breakdown of the arrangement originally envisaged by the Authority and the joint consultants and led to errors which could have been avoided had the experience of the WSC personnel been fully utilized.

Once the work on Contract E was under way, it soon became apparent that JHC, despite their reputation for ability to handle labour problems, were subject to the same troubles as had beset WSC. Indeed, in the months between 17th March and the collapse, there were as many, or more stoppages by the men as had occurred prior to the takeover. This was undoubtedly due in great measure to the fact that JHC was compelled to take on to its work force some men from the WSC payroll.

Relieved of the erection part of Contract S, WSC found itself with a number of men who were no longer required. Some of these were found other jobs on WSC projects, but 22 of the less satisfactory workers, including many troublemakers, were made available to JHC. For reasons which appear in Section 9.1.2., the JHC management were virtually forced to employ these men on Contract E. In the result, the industrial troubles which had plagued WSC continued. Nixon, in evidence, claimed that during the next seven months, a partial elimination of the undesirable elements among the men had taken place, and that just prior to 15th October, the situation had improved to a stage where the abnormal strike tendency had been overcome, and the future prospects for trouble-free industrial relations looked promising. We see no reason why this evidence should not be accepted. Due to strikes, " go slow ", and similar troubles, the work of Contract E lagged just as badly, if not worse, than had the work of Contract S under WSC management. While labour problems accounted for most of this delay, we are satisfied that the JHC organization must also bear a share of responsibility.

The JHC site staff proved to be inadequate. We have noted elsewhere Nixon's frequent absences, and his section engineers were comparatively junior and inexperienced officers who lacked the capacity to control the work satisfactorily, in particular, the difficult labour situation, or even to find the answers to day-to-day problems. There should have been some deputy to Nixon to fill the gap between the project manager and the section engineers. This was recognized by JHC, but too late. Miller was appointed to just this position but only three weeks before 15th October, 1970.

Again following the pattern set by WSC, the JHC engineers failed to take adequate steps to ensure that the camber of the half spans were within acceptable limits on span 10–11. The difference in camber could and should have been 1 inch or less. This unfortunate error led to the same problems and difficulties which had been encountered on span 14–15, but this time with disastrous results.

When it was found that the difference in camber between the two half spans was of the order of 4 inches, which was, or was believed to be, a greater difference than could be adjusted by the method provided in the Procedure Manual, it was the JHC engineers who suggested the use of kentledge, which they thought to be effective and speedier than the use of jacks. This turned out to be the gravest of errors as appears from Section 2.1.5. Our condemnation of the adoption of kentledge is not based merely on our knowledge of subsequent events. At the time of the decision, Nixon and Tracy had no calculations available giving them any knowledge of the stresses likely to be created.

JHC seeks to avoid the responsibility for this mistake because Hindshaw's approval had been obtained, although given reluctantly.

However this does not exonerate Nixon and Tracy. They conceived the idea and persuaded Hindshaw to agree, which he should not have done. The responsibility for this tragic error must be shared by both organizations.

We have considered how far the JHC site staff departed from the Procedure Manual and believe that there was little serious divergence from the prescribed procedures, except on occasions when the consent of the resident engineer or his deputy was obtained. Nevertheless, we feel that the Holland men must take some share of the blame for the constant bickering and argument which occurred between them and the FF & P engineers.

We are satisfied that at least on the west side the JHC organization was quite unsatisfactory. The bolting-up of splices was unsystematic and inefficient (Section 8.2.5.). The section engineers allowed the incorrectly fabricated diagonal braces to be installed. They failed to ensure that all K-plates were installed and in some cases, removed them after they had been fixed.

It is also a matter for criticism, that the JHC engineers sited various huts, including the first-aid hut and the time clock shelter, beneath the longitudinal splice of span 10–11 while the half spans were still in course of erection. There is evidence of objects falling from the gap between the unjoined half spans, to the imminent danger of men who perforce had to venture under the bridge work. Although it is said that, unlike the east side, there was a lack of space in which to place these huts on the west side, we are satisfied that some safer and more suitable arrangement could have been made, and the siting of these huts where they were, was an error of judgment. At the least, the gap between the two half spans should have been securely covered, over land, and the failure to do so was a breach of duty.

The responsibility for the final act which triggered the collapse of span 10–11, namely the removal of bolts from the 4–5 splice—is a matter that has given us great difficulty. After a most careful scrutiny of the whole of the evidence, we are satisfied that the JHC engineers, while they did not actively oppose the unbolting as a means of getting rid of the " buckle ", were wise enough to entertain very serious apprehension about doing this at the stage in erection of the half spans which had been reached. The apparently successful operation on the east side had of course, been carried out after a degree of cantilevering, and when the two half spans of 14–15 had been almost completely joined longitudinally.

It had been agreed to adopt a similar procedure on the west side at a similar stage in erection. Whether the decision to bring forward the operation was Ward's, Hindshaw's or both, it was certainly not a JHC decision. Tracy obviously had the gravest doubts of its wisdom, to the point where he insisted on written instructions from Ward, which was most unusual. The actual work of unbolting was done under Ward's direct supervision, which as Ward told us, was at his insistence because it was " a completely non-standard operation ". The normal practice was that even if instructions emanated from Hindshaw or Ward, the orders were given by Tracy through the foreman to the men. In this instance, Ward gave these orders and watched them being carried out. Tracy merely stood by—and at one stage, actually moved away from the 4–5 splice leaving Ward solely in charge. When, for a short period, Ward and Tracy were both absent, Enness assumed control, so that the JHC engineers had little to do with the operation.

The whole incident was a complete departure from the practice which had previously been followed.

There is no evidence that anyone of the JHC site staff, other than Tracy, knew of the decision to proceed on 15th October.

From Tracy's point of view of course, the responsibility for the removal of the bolts had been assumed by Ward. Thus is it difficult to lay any very serious blame on Tracy, for falling in with what amounted to a direct order from the engineers, in a field which was within their province. It may be that Tracy should have made further resistance, have reported the matter to Nixon and if necessary carried the matter to top-level authority, but this is counsel of perfection, and his failure to do so is understandable. For these reasons, we cannot attach any part of the responsibility for this tragic act to JHC or its personnel. (2.1.6. and 2.1.7.).

10.1.6. FF & P.

Of all the parties involved in the West Gate Bridge project, none enjoyed a higher reputation than FF & P. For half a century they had been in the forefront of the world's leading bridge designers, and had successfully designed and supervised the erection of many major bridges including several of the longest spans.

In a way this extraordinarily high reputation was the source of some trouble on West Gate. It seems to us that some at least of the other parties were dazzled by the firm's reputation to the point of uncritical acceptance of its design and advice.

We have already adverted in detail to the FF & P design for West Gate and need not repeat the criticisms made in Parts 3 and 4, where it will be noted that the design was in many respects inadequate.

Apart from criticism related to the original design the firm is further censurable for failures and errors of judgment occurring during the period preceding construction, and during the period from commencement of the work up to the collapse of span 10–11.

The matters of criticism which we now consider should be divided into two categories, firstly, mistakes of commission and omission of the partnership itself, and secondly the errors of the FF & P employees on the site, for which the firm is vicariously responsible. Some matters of course fall into both categories, as when some action for which we criticize the site staff was later confirmed or adopted by the London principals.

As these matters have all been discussed previously in some detail, it is sufficient in this section, to do little more than list them, where they constitute errors for which we regard the firm as censurable.

Under the heading of direct failure of the partnership itself we observe the following :—

The firm accepted the proposal in the tender of WSC to erect span 10–11 in two longitudinal halves, without ensuring that all difficulties had been foreseen and forestalled by contractors. The firm apparently failed to appreciate that more than ordinary care was required of a contractor employing this unusual method, and that there should be a correspondingly increased intensity of supervision by the consulting engineers. In fact, no such special supervision was provided. (2.1.1.).

Having on the 12th March, 1968, agreed to supply to WSC a set of bridge design calculations, the firm neglected, or refused to make these available, thereby causing serious disruption to the programme arranged by WSC. (1.2.4. and 4.1.1.).

When in September, 1970, the Authority, JHC, and for that matter the unions employed on the work, had all become seriously concerned as to the safety and adequacy of the structure and alarmed as the result of the Milford Haven collapse, FF & P, through Hindshaw, and in conjunction with Maunsell, gave assurances as to the adequacy of the design which were unsupported by any relevant or sufficient calculations. The criticism which we have directed to Hardenberg and to Maunsell on this matter, apply with greater force to FF & P, the designers of the steel spans and the consultant directly responsible for supervising their erection. (10.1.3. and 10.1.4.).

The subject of the calculations made, or which should have been made, by FF & P are fully dealt with in Part 4 of this Report. It is sufficient to state here that such calculations as were supplied to the Commission demonstrate complete inadequacy. Perhaps the most significant matter is the absence of some vital figures, for which no satisfactory explanation was ever forthcoming. We were left in the unhappy position of being unable, on the evidence, to decide whether these calculations were never made, lost, or simply not supplied to us despite repeated requests. FF & P can take no comfort from any of these explanations. (4.1.2.).

When selecting engineers to supervise the work on site, FF & P chose men who were doubtless competent within their experience, but as exemplified by Hindshaw and Ward had had little if any experience on site in the erection of any major steel bridge.

Having appointed relatively inexperienced engineers they failed to brief them adequately as to their duties and to support them by arranging constant and efficient communication with the London office. There was a failure to insist upon proper information flowing from the site to London. Full detailed reports should have been required to keep London informed on site difficulties. Conversely, when the site engineers required information or advice from London, they had the greatest difficulty in obtaining it. (8.2.3.).

In our view the Authority was right in demanding that a FF & P partner should have been permanently on site in Melbourne.

Failing this, efficient and adequate communications should have been maintained between the site staff and the principals in London.

There was a failure to give any adequate check on the stresses set up during the various stages of erection. Even where contractors are responsible under the terms of a contract—it is essential that consultants satisfy themselves independently that work is being done safely—because men's lives are at stake.

In addition to the above matters relating directly to the firm's work as consulting engineers, the conduct of the partnership is further censurable in a more general way.

We find for example, that the organization of the London office was most inefficient resulting in failure to answer letters and telex messages even of the most urgent character, to the great inconvenience of all other parties, and at times, in circumstances where the failure to reply with needed information created serious delay and expense, involving in some cases the risk of danger.

Furthermore, we are compelled to find that when, after the Milford Haven collapse, the partnership decided that the steel spans must be strengthened to eliminate weaknesses exposed by the failure of the steelwork on Milford Haven, the measures adopted for this purpose were confused with those taken concurrently to strengthen the structure because of proposed accelerated concreting programme, and as a result the Authority and other parties were in fact misled, whether intentionally or not. It may also be stated that the insistance that the two programmes were inseparable was persisted in during the early stages of our inquiry. (See 2.2.1.).

Adverting now to the actions of the FF & P site staff, which merit criticism we mention the more outstanding matters which were—

They authorized lifting of half spans to top of piers although cambers had not been equalized within reasonable limits, say 1 inch or less.

Hindshaw agreed to use of kentledge instead of jacks, to correct camber, either as had been done on span 14–15 or as laid down in Procedure Manual.

The responsibility for the final act of removing the bolts from the 4–5 splice of span 10–11, has been fully discussed in Section 2.1.6., where we consider the problem of Hindshaw's knowledge of Ward's action, and in Section 10.1.5., where we state our reasons for finding JHC to be without responsibility in the matter. It is here sufficient to say that Ward authorized removal of site bolts which triggered the final collapse before the two half spans were properly joined together and before any cantilevering had been done. (2.1.6.).

There was a failure to take regular check levels on steelwork as necessary in early morning, before levels were affected by temperature. It is essential that these checks should be carried out both by the contractors and the consultants.

For all the above reasons, we are compelled to conclude that FF & P bear a heavy burden of responsibility for the failure of the bridge. While we have found it necessary to make some criticism of all the other parties, justice to them requires us to state unequivocally that the greater part of the blame must be attributed to FF & P.

10.2.1. The Labour Unions.

Throughout the construction and erection of the steel spans there was an extraordinary amount of industrial trouble, including strikes caused by demarcation disputes between unions and for other reasons. There were also numerous stop-work meetings and much absenteeism.

These matters are fully discussed in Section 9.1.2. It is sufficient in this section to observe that the great amount of time lost caused extreme distruption of the programme of WSC and later of JHC. As a result the work of constructing the bridge was gravely delayed.

If the work of erecting span 10–11 had been proceeding at anything like a reasonable rate, the whole of the longitudinal splice in the top flange would have been completed except for a short length adjacent to the buckle in box 4–5 prior to 15th October, 1970. In bad weather, the men could have worked in box 8 assembling the necessary strengthening required inside it. Men would also have been available to fit the guards required on the access ladders to the two gantries beneath the span. This would have taken only a day or two and should have been given high priority. The whole length of the longitudinal splice in the bottom flange would then have been completed, thus considerably increasing the strength of the span and its resistance to failure under any circumstances.

For their behaviour on the contract, which inevitably led to the quite unnecessary weakness of the span at the relevant time, the unions and men must bear their share of responsibility for the tragedy that ensued.

10.3.1. Final Conclusions Related to the Terms of Reference.

To conclude the matter of responsibility we make some observations designed to relate the findings which are set out in the foregoing sections to our Terms of Reference.

The Terms of Reference required us to inquire and report upon the circumstances surrounding as well as the causes direct or indirect of the failure of span 10–11 on 15th October.

The choice of these wide general terms instead of specific questions necessitated a correspondingly wide range of inquiry, in the course of which we deemed it our duty to seek first the cause of the failure but also to observe and report upon any matters which we believed to provide an explanation of how and why these causes came into being, even if such matters were not part of the direct chain of causation.

It was our duty to find facts, and where the evidence enabled us to do so, to report upon the responsibility of individuals or parties, but we were not concerned to establish legal liability. It follows that in examining matters which were causes in fact of the disaster, we were not limited to matters which would be regarded as causes in law in the sense of determining legal liability. In the circumstances, any distinction between direct or indirect cause is somewhat artificial. In the following summary we confine ourselves to what we believe to be those matters which are in the direct chain of causation which led to the collapse of span 10–11.

The immediate precipitating cause was the removal of about 30 bolts from a transverse splice in the upper flange of span 10–11 at boxes 4 and 5 near mid-span. The responsibility of Hindshaw, Ward and Tracy for this action is discussed in Sections 2.1.6. and 10.1.5.

The bolts were removed in an attempt to eliminate a buckle which had occurred in the upper flange in the vicinity of the 4–5 splice. The cause of this buckle was placing seven 8-ton blocks of concrete on the upper flange of the span. The use of kentledge was suggested by the JHC site staff and the approval of Hindshaw had been obtained. (Section 2.1.5.).

This kentledge was placed on the flange as an expeditious means of correcting a difference in camber found to exist between the two halves of span 10–11 after they had been jacked up to the top of the piers. (Section 2.1.4.).

The difference in camber occurred because of the failure of the JHC engineers to assemble the boxes of each half span with sufficient care and skill so as to ensure that the camber of each half span was the same as the other or differed only in acceptable degree.

The basic factor from which arose even the possibility of a difference in camber was the proposal made in the tender of WSC for the erection of spans 10–11 and 14–15 in half spans divided longitudinally. This proposal had been accepted by FF & P when advising the Authority as to the acceptance of WSC's tender.

Finally, as an over-riding circumstance within which the above sequence was carried out, the factor of safety for many of the approved erection conditions was already too low, so that the abnormal actions of adding kentledge and undoing bolts reduced the narrow margin even further, and in the latter case left no margin at all.

10.4.1. The Additional Term of Reference.

The evidence forthcoming in the early stages of this inquiry, together with the observations we had then made, caused us to suspect that we might well find features of the design of the steel spans which were inadequate, but which were not part of the cause of the failure of span 10–11 and therefore not strictly within our original Terms of Reference, unless the term " circumstances surrounding " be given a somewhat strained interpretation.

Accordingly, we decided that the chairman exercise the power conferred by the *West Gate Bridge Royal Commission Act* 1970, Section 1 (3) to request an extension of the Terms of Reference. As a result of this request our terms were extended on the 23rd day of February, 1971 to include the following :—

> " to inquire into and report upon whether any aspect of the design of the steel span between piers 10 and 11 is inadequate or undesirable."

Our suspicions proved well founded ; the evidence including our observations upon examination of the standing span 14–15, and of the calculations and drawings produced as exhibits, and in particular the Maunsell, London report, compel us to conclude that there are a number of features of the design which appear inadequate and which require further careful checking and reassessment.

The features which have come to our attention are set out in full in Section 5.1.1. where the above report is cited. From the list it will be seen that there are a great many features which require further investigation. It is therefore our considered view that the whole design of the steel spans should be thoroughly re-examined by an independent consultant in the need to assure the future safety of the structure.

We are satisfied that this check should be undertaken as a matter of urgency.

We desire to record our thanks to some of those who have greatly helped us during the course of this inquiry.

We were fortunate to have the services of the Solicitor General, Mr. B. L. Murray, Q.C. as senior counsel assisting the Commission. He and his juniors, Mr. J. W. Mornane and, until he was appointed to the Bench, His Honour Judge Gorman, worked most assiduously, and their skill in presenting evidence, cross-examination and advice to the Commission from time to time was of very great assistance.

The Country Roads Board, while involved in part of the West Gate Bridge project, was entirely independent of any issue which we were required to investigate. We were therefore able to call upon the Board for assistance which was given very readily and generously in a great number of ways. The Board made available to us its facilities which very greatly assisted us in our task. In particular we had need of an experienced engineer who could make observations, measurements and tests on our behalf. We were fortunate that Mr. Norman Haylock, an officer of the Country Roads Board, was seconded to us for the duration of the Commission and his services proved invaluable.

The Victoria Police, whose work immediately following the disaster has been mentioned in the Report, provided very great assistance to the Commission by arranging the attendance of witnesses and in many other ways essential to the orderly conduct of our investigation. For this most useful co-operation we express our gratitude.

Finally, we record our great appreciation of the services of Mr. Peter Hosking, the Secretary to the Commission. We find it difficult to express adequately our indebtedness to Mr. Hosking, a highly efficient officer who displayed a great capacity for organization. His services, always given cheerfully, lightened the task of the Commission and we are indeed grateful to him.

E. H. E. BARBER, Chairman.
F. B. BULL, Member.
HUBERT SHIRLEY-SMITH, Member.

Melbourne,
14 July, 1971.

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(*This report is not readily available—it is however summarized in Ref.* 2 *below*).

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APPENDIX A.

ALPHABETICAL LIST OF WITNESSES.

Name of Witness.	Pages of Transcript.
Adams, Maxwell Hughes	919–954
Atkinson, Alfred George	620–629
Barmby, Bryan Desmond	965–981
Bartlett, Dennis Charles	912–916
Birkett, Edward Miles	629–656A 3372–3621 4120–4146
Bounds, Ernest Francis	1138–1156
Brown, Dr. William Christopher	2247–2718 2900–2953
Burbury, Timothy Vaughan	388–446
Cerin, Marijan	916–918A
Coates, Sgt. Francis Kevin	1200–1209
Dixon, Trevor John	696–698
Enness, Ernest	447–490 1398–1426
Fernie, Geoffrey Norman	3102–3239
Freeman, Sir Ralph	40–121
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256.	Calculations by Grassl in November, 1970, relating to strength of splice	3308
257.	Correspondence file relating to changeover from WSC to JHC prepared by the Authority	3348/3376/3562/3564/3567/3860A/3890
258.	Volume 7 of calculations made by Grassl and members of his staff relating to the strength of the diagonal brace, based on cross-girder of $\frac{5}{16}$-in. thickness	3361
259.	Letter dated August 24, 1970, JHC to the Authority containing details of industrial stoppages	3368
260.	Letter dated October 23, 1970—Police Department to Commission concerning " Search and Rescue Operations "	3369
261.	Letter dated April 18, 1969—Baxter to Birkett	3387
262.	Notes of conversations Hindshaw and Kerensky. September 14, 19, 22, 24, 1970	3413A–3763
263.	Letter dated June 8, 1970—Birkett to Kerensky	3447
264.	Letter dated September 15, 1970 Hindshaw to Birkett	3458
265.	Letter dated July 6, 1970 Birkett to Vian	3473
266.	Notes of conversations Birkett with Roberts, February 1970, Kerensky and Baxter, July, 1970 and notes of meeting with Kerensky, August, 1970	3513
267.	Documents relating to negotiations Clause 19 Contract E	3562
268.	Letter dated September 17, 1968 Birkett to Roberts	3565
269.	Report dated September 15, 1970 by Hindshaw to Birkett	3580
270.	Letter dated January 30, 1970 FF & P to Lee	3586
271.	Letter dated April 20, 1970 JHC to resident engineer	3595
272.	Two letters dated July 15, 1970 and July 17, 1970, Birkett to Kerensky	3605
273.	Report dated September 21, 1970 Hindshaw to Birkett plus handwritten original ..	3615
274.	Hindshaw's notes for his suggestion of an address to workers	3618
275.	File of correspondence between Werkspoor and FF & P handed to Birkett by Hardenberg	3622
276.	Letter dated June 19, 1970, Birkett to Kerensky	3627
277.	(1) Telex dated February 24, 1970, Brown to Freeman and Kerensky	3631
	(2) Telex dated March 2, 1970 Brown to Freeman and Kerensky	3631
	(3) Telex dated March 2, 1970 Brown to Freeman and Kerensky	3631
278.	Four references for Hindshaw	3636

APPENDIX B—*continued.*

Exhibit No.	Description.	Transcript Page.
279.	Curriculum Vitae of Hindshaw and Crossley	3640–3647
280.	Letter dated May 15, 1970 Hindshaw to Kerensky	3641
281.	Letter dated June 10, 1970 Meyer to Kerensky	3706
282.	Letter dated October 9, 1970 Birkett to Freeman enclosing letter from the Authority dated October 5, 1970	3710
283.	Five drawings relating to increased stiffening prepared in London in July and August, 1970	3910
284.	Five drawings relating to increased stiffening prepared at site in August, 1970	3910
285.	Letter dated October 27, 1970 Birkett to Wilson, with enclosures, telex dated October 5, 1970 Freeman to Birkett	3913
286.	Computer outputs alleged to be forwarded to Crossley on site prior to July, 1970 ..	3918
287.	Three folders of miscellaneous calculations prepared by Crossley at site	3920A
288.	Three stress diagrams prepared in April, 1971	3933
289.	Statistical evaluations of material yield strengths	3935
290.	Personnel referred to in the evidence originally employed by WSC and hired to or involved with JHC	3998
291.	Preliminary information to contractors February, 1967	4018
292.	Letter dated August 28, 1970 McIntosh to project manager, WSC	4080
293.	Five sets of minutes of meetings between Maunsell, JHC, WSC, and sometimes with others, dated from February 3, 1970 to April 4, 1970	4081
	Minutes of meeting on March 17, 1970 between Nixon and Union representatives ..	4268
	Minutes of meeting on March 26, 1970 between WSC and JHC	4292
294.	Letter dated May 26, 1969 from Vandermolen to WSC	4110
295.	Correspondence March 17, 1970 to August 19, 1970 between WSC, JHC and the resident engineer	4120
	JHC staff organization chart	4275
296.	Chart showing time losses due to weather and labour disputes	4141
297.	Letter dated April 10, 1968 Maunsell to Roberts	4142
298.	Charts A, B, C and D which were enclosures of letter dated January 20, 1970 Birkett to Freeman	4146
299.	Appendices 1 to 8 produced by Nixon with his statement	4150
300.	Plan of site on the west side in the vicinity of span 10–11	4150
301.	Box dimension record for half boxes 1 to 8 west, north and south	4181
302.	Specimen inspection record relating to Panel P 271 which is part of box 5 west	4181
303.	Organization Chart JHC Contract E	4204
304.	Measurement of K-plate thickness and measurement of top flange thickness	4205
305.	File of documents relating to stiffening measures	4206
306.	Letter dated April 13, 1970 Nixon to the resident engineer	4227
307.	Diary of Nixon	4227
308.	Roderick's second interim report dated May, 1971	4229
309.	Industrial diary of Nixon	4229
310.	Procedures submitted by WSC to FF & P and approved, July to October, 1969, together with covering letters	4242
311.	Model illustrating diagonal brace	4291
312.	Letter dated February 13, 1967 Lee to Roberts	4302
313.	Letter dated April 11, 1967 consulting engineers to the Authority	4302
314.	Letter dated March 12, 1969 consulting engineers to WSC	4302
315.	Telex dated August 14, 1970 Birkett to Fernie	4302
316.	Memorandum dated October 1, 1970 Hindshaw to Birkett	4302
317.	Letter dated October 5, 1970 Hindshaw to JHC	4302
318.	Extracts relating to the West Gate Bridge from Nixon's monthly reports to the Board of JHC from May 3, 1970 to November 4, 1970	4303
319.	Hardenberg calculations of the critical stress in the bulb flat cover plate	4303

APPENDIX C.

Clause 19 of Contract E.

Clause 19 (*a*) Notwithstanding the provisions of this Agreement or the Conditions of Contract Holland shall not be liable to the Authority in contract tort or otherwise howsoever for—

(i) any failure on its part to construct and complete the works or any part thereof ;

(ii) any defect which shall appear in the part of the Lower Yarra Bridge the erection of which is covered by this Agreement except and unless the same be caused by gross negligence on the part of Holland. To constitute gross negligence there must be a substantial departure from the duty of care required to avoid liability in negligence.

(*b*) Nothing in sub-clause (*a*) shall impair or reduce the right of the Authority to terminate this Agreement for breach of Contract by Holland.

APPENDIX .D.

Ward's Instruction to Release Bolts (Exhibit 99)

28

To: W.F. Tracy
J. Holland & Co.   14th October 1970

Buckle in top flange at 4-5 splice.

Will you please bolt up the top of the diaphragm adjacent to the 4-5 splice before starting jacking down at pier 11.

In order to do this, it will be necessary to release the bolts in the transverse inner top deck panel at the 4-5 N splice. This must be done under my supervision. After removing the cover plates at this point, the plates should be "blacksmithed" back to line, the cover plates replaced and the bolts initial tightened. After box 11 has been erected the bolts on one side will be released and then ~~again~~ initial & final tightened.

D. Ward
F.F. & P.

APPENDIX E.

FURTHER NOTES ON THE DIAGONAL BRACE.

In Section 2.2.3. the reason for fitting special diagonal braces at the diaphragm positions was set out. These special diagonal braces, with palm end plates attached, were used only on the half span 10–11 north. No brace was needed at all at these locations on the south half girders because the diaphragm itself was bolted into the south half box during lifting, and so served to stabilize the transverse rib on that half. Braces should have been provided for 14–15 north, but by the time this was realized it was already too late and instability had taken place. Because of this, no special diagonals were made for the east side.

The details of the diagonal braces both normal and special, were fully set out in the Procedure Manual which also contained procedures for fitting them. The sketch of the end palm plate given there was not drawn to scale and the dimensioning was such as to lead to the possibility of misinterpretation. This was unfortunate as it appears the braces were made directly from the sketch in the Procedure Manual. On Fig. 12 is included an exact reproduction of this sketch, whereon it can be seen that the dimension " $1\frac{3}{4}$ inches " below the bottom right-hand corner of the 8 in. × 6 in. end plate should really be the distance from the back of the angle to the centre of the hole. If the sketch had been to scale this would have been obvious. The fabricator, however, took the dimension to refer to the distance of the hole from the right-hand edge of the plate and in consequence he drilled the hole 2 inches away from its intended position. The sketch from the Procedure Manual incorporated another error in that the detail of the end plate as shown is " handed " for the east side and should have been made " opposite hand " for the west side. This was not realized by the fabricator and the members were made as drawn.

Each of the above errors in fabrication, by themselves, would have made the palm plate project beyond the vertical face of the bent plate transverse rib and thereby made it not possible to bolt up the diaphragm. Fortunately, however, the two errors cancelled each other and the diagonal as actually made did not have its end plate projecting beyond the face of the transverse rib.

It must be admitted that part of the reason for this absence of overhang was that the horizontal flange of the bent plate transverse rib was $3\frac{9}{16}$ inches as measured from the wreck and not 3 inches as on the tender drawings. The backmark to the hole in the flange of the bent plate was $1\frac{13}{16}$ inches and not $1\frac{1}{2}$ inches as on the working drawing. Careful measurements made on the wreck show that the end plate of the brace could not have projected beyond the vertical face of the transverse rib, although it must have come very nearly up to the line of that face.

It is interesting to note that had the end plates been made as intended and had the backmark to the hole in the bent plate been $1\frac{1}{2}$ inches as on the drawing, there would have been a $\frac{3}{4}$-in. overhang of the plate edge beyond the vertical face of the rib. Even with the bent plate rib as in fact made the overhang would have been $\frac{7}{16}$ inch.

It is clear that not enough thought was given to the detail by JHC when drawing up the procedure, and FF & P failed to pick up the errors when checking, prior to giving their approval.

The consequence of the two errors of fabrication, described above, was firstly that the base plate was less stiff than it should have been if the hole had been correctly drilled, and this would be particularly so in the post-yield state ; secondly, the leg of the 5-in. angle brace which was parallel to the plane of the diaphragm was in a plane about $5\frac{1}{2}$ inches from that of the vertical part of the transverse rib, instead of being very nearly in the same plane. There were only two possible ways in which the brace could have been fitted and in both of these the leg of the angle which was in the vertical plane was away from the correct position.

JHC were well aware of the double error but there is no evidence that at any time they asked FF & P to approve the use of the incorrectly made braces.

As discussed in Section 2.2.3. the buckle which occured at the 4–5 splice when kentledge was added could only have taken place if the diagonal brace was either missing or badly deformed.

The possibility that the brace was not in place at the time the buckle developed was raised by several witnesses. Shortly after the collapse, Simpson communicated with FF & P in London suggesting that the diagonal brace in box 4 might not have been in position at the vital time and a number of calculations were made by FF & P on this hypothesis. They formed the conclusion that if the diagonal had not been present, buckling would have taken place over the 21-ft. spacing between the adjacent transverse ribs, which were stabilized. This was, of course, what did happen on 14–15 north, when no diagonals were fitted.

On the other hand a number of reliable witnesses gave evidence that the diagonal was indeed in place at all material times and several were able to describe in detail the damaged end connections after the buckle at 4–5 north had taken place. We have also examined the damaged brace itself, as recovered from the wreck, and noted that the plate distortions are consistent with the member having been subjected to an axial overload in tension, see Plate 13. This damage did not take place in the collapse on 15th October, because we are sure that by then the brace really had been removed.

We have also given consideration to the hypothesis that the brace was not in place immediately prior to the buckle of the inner upper panel, but that after the buckle had deen detected on 7th September, some person unknown, attempted to refit it and thereby to force down the buckled plating. In the unsuccessful attempt to do so, according to this hypothesis, the end plates were distorted as the bolts were tightened up. This theory appears to fit the facts but there is not the slightest evidence of witnesses to support it.

APPENDIX E—*continued.*

If then we accept that the diagonal was in place, it is necessary to consider the action whereby it was stretched. In Section 2.2.3. three possible actions are set out and conclusions stated. The supporting evidence for these conclusions is as follows :—

If fouling did take place between the top of the diaphragm and the underside of the transverse rib it is necessary first to see how such fouling was possible. As has already been demonstrated, the end plate on the diagonal did not stick out and so could not by itself be the source of interference. We know, however, that in other parts of the bridge the opposing pairs of transverse ribs were sometimes out of line, by as much as $\frac{3}{4}$ inch. If the misalignment was as little as $\frac{1}{4}$ inch then the top of the diaphragm might have pressed upwards on to the end plate on the diagonal while the kentledge was pushing the north half span down. It could be argued that since there was no projection of the plate, the interference would have occurred anyway, even if there had been no end plate present. The corner of the transverse rib was, however, rounded and unless the overlap of diaphragm and rib was excessive the fouling in the absence of any palm plate might have cleared automatically as the contact point slid across the rounded edge.

The evidence of witnesses as to whether fouling did or did not take place is conflicting.

Ward, Riggall and Miller examined the buckle both inside and outside on Monday, 7th September, 1970, the first day shift after the discovery of the buckle on Sunday night.

Ward is quite definite that when he first examined the situation the diaphragm was fouling the plate of the diagonal brace, the overlap being about $\frac{1}{4}$ inch. He goes on to describe how a packer was subsequently placed between the brace and the diaphragm to clear the obstruction.

Riggall agreed that the inspection was made by himself and Miller, Ward also being there, but claimed that there was no fouling. He had a sketch in his notebook which he claimed was made at the time, to show the relative positions of the diaphragm and the transverse rib. On the other hand, Riggall stated that the form of the distortion of the top palm plate of the diagonal was different from that at the bottom, which is surprising if the only load on the diagonal was that transmitted through the bolts at either end. Miller, the third person present at the inspection, was killed in the collapse of span 10–11.

The evidence of Halsall, a rigger employed by JHC, is relevant, and was given before the question of the behaviour of the diagonal brace had assumed prominence. He stated—

> " On the Monday morning a rigger and myself were sent down to pull the diaphragm clear towards pier 11, and then we had to try to pull the buckle down level agian."

From earlier questioning, it was established that the Monday referred to was Monday, 7th September. Halsall is clearly implying that there was indeed fouling of the diaphragm which he had cleared before attempting unsuccessfully to pull the transverse rib down again.

By the time that James inspected the situation on 19th September, he saw the diaphragm had moved up past the bottom of the transverse rib. This evidence of James is consistent with the situation as stated by Ward and Halsall (taken in conjunction), but is also consistent, directly, with Riggall's statement. It is impossible to determine beyond all reasonable doubt who is correct, Ward or Riggall.

We can see no explanation of Halsall's statement other than that he did exactly what he said, that is to " pull the diaphragm clear ". If Riggall's sketch and his recollection were made after Halsall had done his work they would fit in with other observations, but that is not what Riggall says happened.

Both theoretical analyses and test results suggest that the crippling stress for the 10 ft. 6 in. inner upper panel, with a splice, is in the vicinity of 12–13 tons per sq. in., while the theoretical stress set up when kentledge was added was 12·5 tons per sq. in. Collapse of the panel was thus a likely consequence, irrespective of whether fouling of the diaphragm took place or not. Once the K-plates crippled, the panels would buckle upwards, inducing tension in the diagonal. Brown expressed his opinion that the force induced in the diagonal would be compression, but an elementary examination of the statical equilibrium of the system shows that he was in error, as was unhesitatingly confimed by Roderick. It appears, however, that the force induced in the brace, by this form of panel buckling could have been only marginally big enough to have caused the observed distortion.

As to the remaining possibility that the 21-ft. panel buckled between the transverse ribs on each side of the diaphragm position, analyses were made by both Hardenberg and Grassl which suggested that this form of buckling was not likely to be the first to take place unless there was some other accidental factor operating at the same time. The uncertain " spring stiffness " of the diagonal brace and the other assumptions used, however, make these analyses approximate only. We note that, according to Cullimore's tests, the non-linear " spring stiffness " of the brace would have been reduced almost to zero after $2\frac{1}{2}$ inches of distortion, so that if buckling over 21 feet had been the primary cause of instability, we would have expected the buckle to have been larger than it was, more near to the very large buckles which occurred on the east side where no diagonals were fitted.

In summary, we believe that some diaphragm interference did occur, but that the 10 ft. 6 in. panel, containing the splice and its weak K-plates was in any case on the point of instability.

APPENDIX F.

Statement of Industrial Stoppages on Contract E for Period 13.4.70 *to* 14.8.70.

EXHIBIT 259.

" Mr. O. Meyer,
Australian Carbon Black Pty. Ltd.,
National Mutual Centre,
9th Floor, 447 Collins-street,
Melbourne, Vic. 3000.

Dear Sir,

LOWER YARRA CROSSING—CONTRACT " E ".

Further to our telephone conversation of today's date, the following lists the dates of stoppages on " E " Contract, Westgate Bridge, with a brief reason for the stoppage :—

Date.	*Length of Stoppage.*	*Reason.*
13.4.70	12.30 p.m.—rest of the day	Provision of first-aid man at pier 15.
14.4.70	7.20 a.m.—10.20 a.m.	
16.4.70	9.30 a.m.—rest of the day	Protest over not getting paid for time lost on 13/4 and 14/4.
23.4.70	9.30 a.m.—rest of the day	Dispute over JH using sub-contractors on the site.
24.4.70	7.20 a.m.—11.30 a.m.	,, ,, ,, ,, ,, ,,
30.4.70	All day	Demarcation between AEU fitter ASE hydraulics technician
1.5.70	All day	
4.5.70	7.30 a.m.—rest of the day	Claim for period Saturday when men were sent home because of rain.
5.5.70	All day	
6.5.70	All day	
7.5.70	7.20 a.m.—11.20 a.m.	
8.5.70	12.30 p.m.—rest of the day	Morotorium Protest
25.5.70	All day	Men wanted dismissal of hydraulics technician because he worked 10 minutes later than the rest and it was too dark to work—plus reinstatement of dismissed boilermaker.
26.5.70	All day	
27.5.70	All day	
28.5.70	All day	
29.5.70	All day	
1.6.70	All day	
2.6.70	12.30 p.m.—rest of the day	Protest over our refusal to reinstate dismissed boilermaker.
10.6.70	12.30 p.m.—3.00 p.m.	Meeting over being sent home Saturday because steel was wet and unsafe.
12.6.70	12.30 p.m.—2.00 p.m.	Ironworker meeting to elect delegate
18.6.70	12.30 p.m.—1.45 p.m.	Meeting to discuss inclement weather.
23.6.70	12.30 p.m.—rest of the day	Meeting to discuss company offers in respect of claims for gum boot money—site jackets, &c.
26.6.70	2.40 p.m.—rest of the day	Meeting to discuss provision of boat on Sundays
2.7.70	1.10 p.m.—rest of the day	Claim toilets not clean.
3.7.70	7.20 a.m.—11.20 a.m. 1.50 p.m.—2.30 p.m.	Claim toilets not clean.
4.7.70	2.30 p.m.—3.15 p.m.	Discussion *re* payment for boilermakers east side who worked in the rain.
8.7.70	12.30 p.m.—3.40 p.m.	Meeting over " No new F.I.A. members and men won't have night shift ".
9.7.70	7.40 a.m.—9.15 a.m.	Meeting of F.I.A. to discuss new starters on site.
14.7.70	All day	General meeting to discuss availability for working regular overtime
15.7.70	7.20 a.m.—10.30 a.m. 12.30 p.m.—1.30 p.m.	
16.7.70	7.20 a.m.—12.30 p.m. 3.00 p.m.—3.45 p.m.	Meetings to discuss dismissal of man dismissed for fighting.
3.8.70—14.8.70		Ban on fitting work on the site because of dismissed fitter.

A graph showing time lost and the percentage of available working time lost is also included for your information.

Trusting this is sufficient for your requirement.

Yours faithfully,

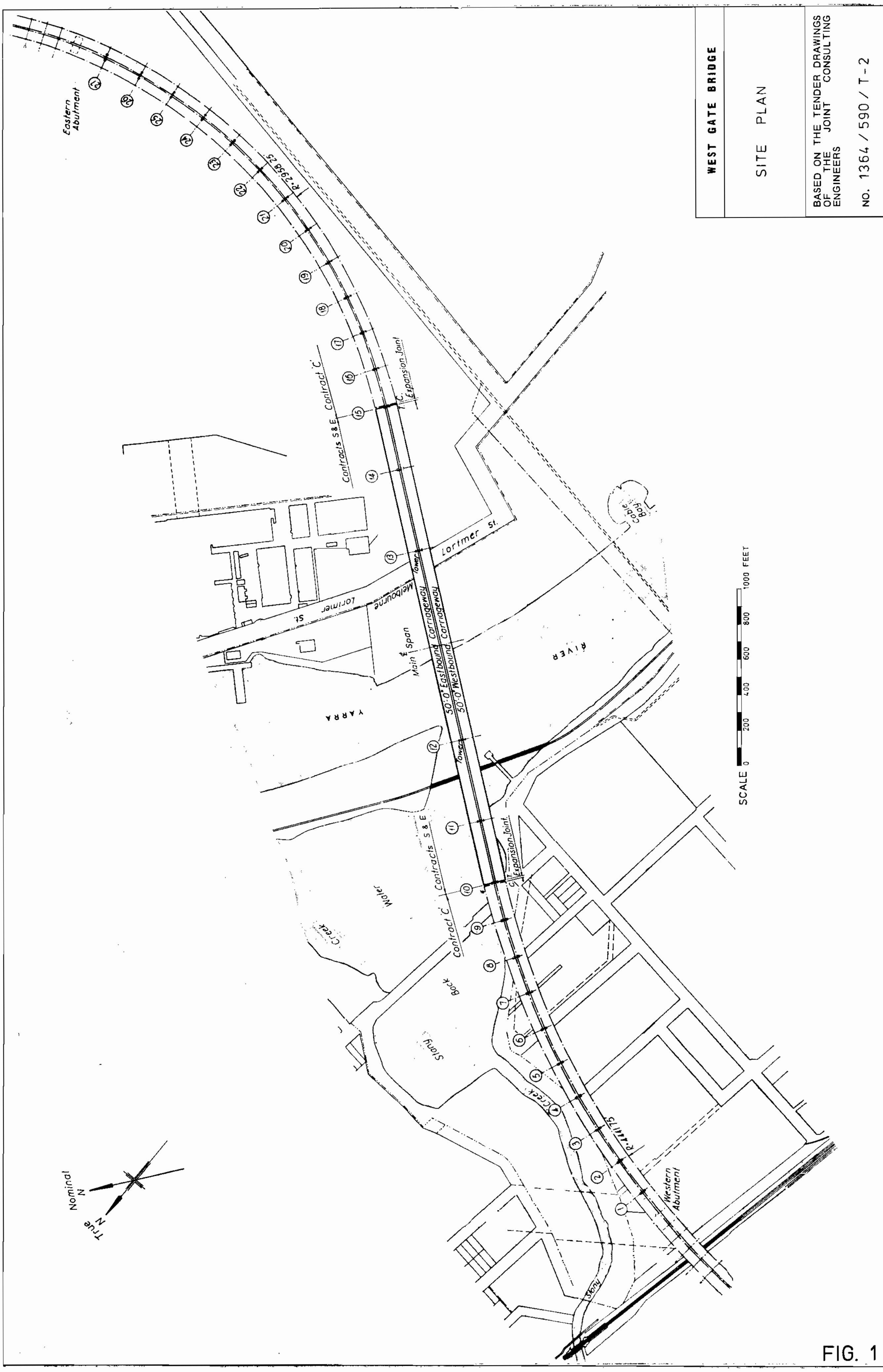
JOHN HOLLAND AND CO. PTY. LIMITED.

L. PLAXTON,
Industrial Officer ".

APPENDIX G.

List of Men Who Lost their Lives in the Diaster.

BARBUTO, Royden ..	.. Boilermaker
BIGMORE, Ross Gregor	.. Carpenter
BOSCOLO, Amedeo ..	.. Carpenter
BUTTERS, BERNARD ..	.. Boilermaker
CARMICHAEL, Cyril ..	.. Riggers Asst.
CROSSLEY, Peter ..	.. Deputy Resident Eng.
DAWSON, Peter ..	.. Rigger
EDEN, Abraham ..	.. Rigger
FALZON, Anthony ..	.. Carpenter
FERNANDEZ, Esequiel	.. Riggers Asst.
FITZSIMMONDS, Bernard	.. Riggers Asst.
GERADA, Victor ..	.. Riggers Asst.
GRIST, John Richard	.. Foreman Boiler-maker
HARBURN, William Nicholson ..	Boilermaker
HINDSHAW, Jack ..	.. Resident Eng.
HUNSDALE, Trevor Thomas	.. Fitter
LITTLE, John ..	.. Rigger
LUND, Charles Waite	.. Foreman Boiler-maker
MCGUIRE, Peter ..	.. Leading Hand Rigger
MILLER, Ian ..	.. Asst. Project Manager
MURPHY, Jeremiah Joseph	.. Rigger
O'BRIEN, Dennis Anthony	.. Rigger
OZELIS, Joseph ..	.. First-aid Atten-dant
PIERMARINI, Frank ..	.. Rigger
PRAM, George ..	.. Leading Hand Rigger
SCARLETT, Lesley ..	.. Riggers Asst.
STEWART, Christopher Gordon	Boilermaker
SUAREZ, Alfonso ..	.. Boilermaker
TRACY, William ..	.. Section Eng.
TSIHILIDIS, George ..	.. Boilermaker
UPSDELL, Edgar Frederick	.. Storeman
WEST, Robert Charles	.. Boilermaker
WHELAN, Robert Maxwell	.. Boilermaker
WOODS, Patrick John	.. Rigger
WRIGHT, Barry ..	.. Boilermaker



WEST GATE BRIDGE
SITE PLAN
BASED ON THE TENDER DRAWINGS OF THE JOINT CONSULTING ENGINEERS
NO. 1364 / 590 / T-2
Eastern Abutment
Western Abutment
Contracts S&E
Contract 'C'
Expansion Joint
Lorimer St.
Melbourne
Main Span
50'0" Eastbound Carriageway
50'0" Westbound Carriageway
Tower
YARRA
RIVER
Cable Bay
Stony Creek
Back Water
SCALE 0 200 400 600 800 1000 FEET
Nominal N
True N

FIG. 1

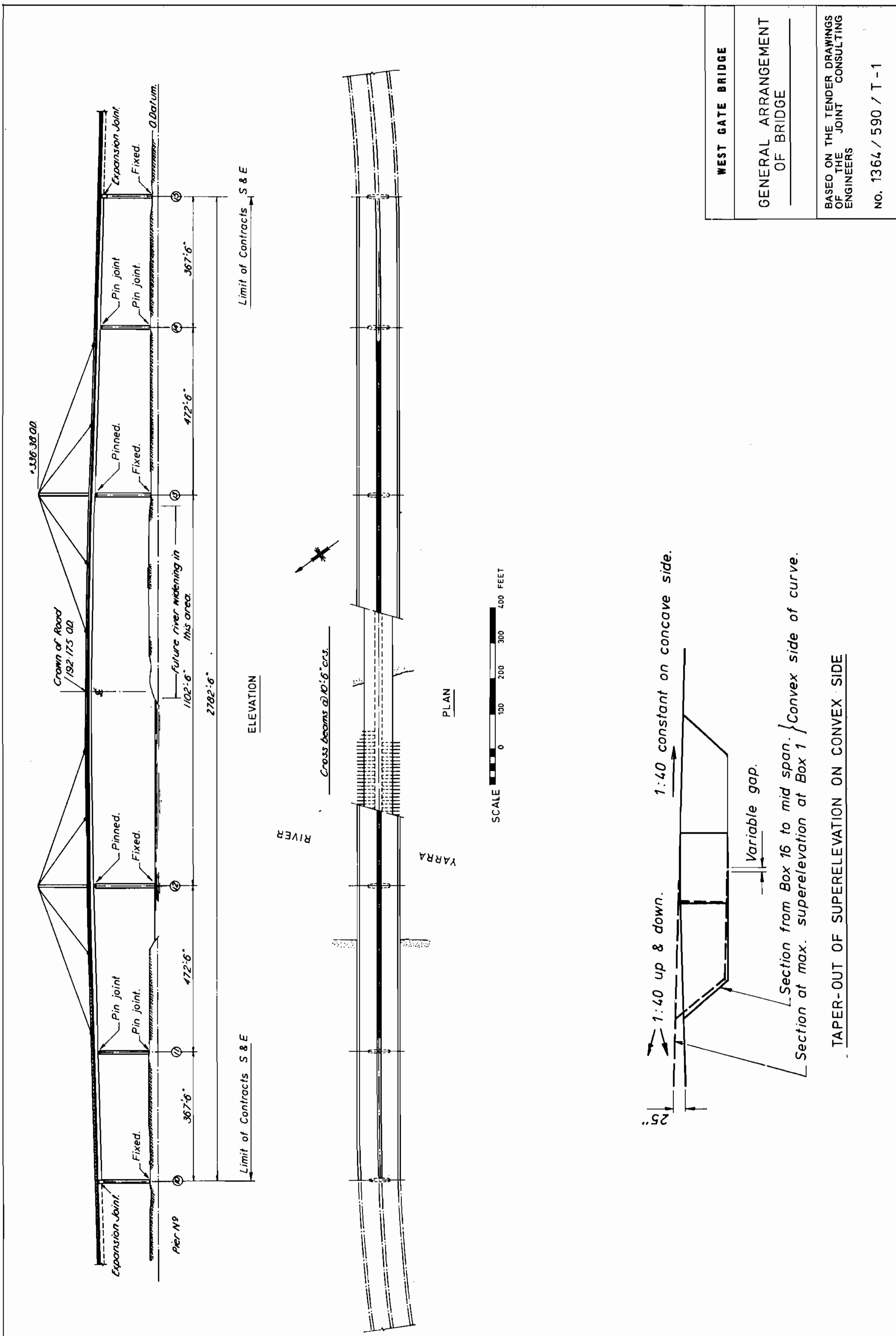
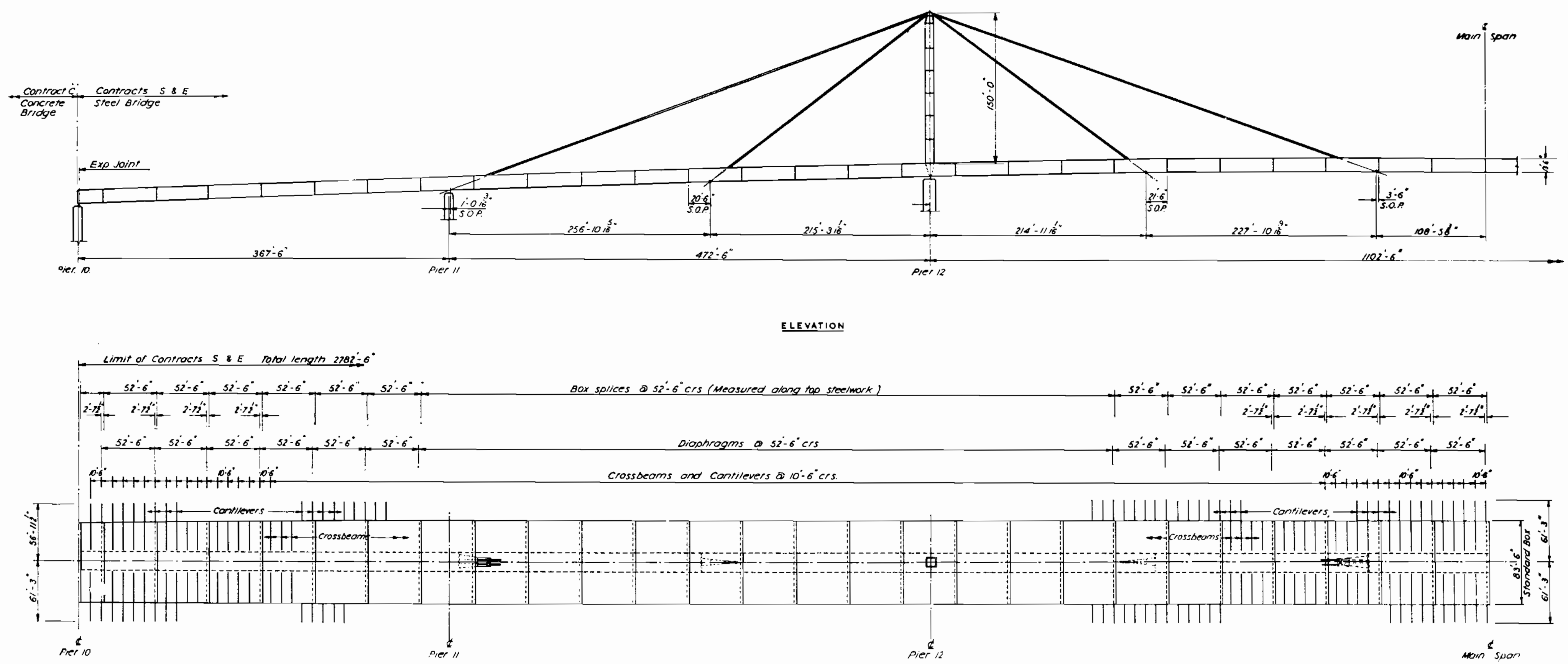

WEST GATE BRIDGE
GENERAL ARRANGEMENT OF BRIDGE
BASED ON THE TENDER DRAWINGS OF THE JOINT CONSULTING ENGINEERS
NO. 1364 / 590 / T-1
ELEVATION
+336·38 O.D.
Crown of Road 192·175 O.D.
Expansion Joint.
Fixed.
Pin joint
Pin joint.
Pinned.
0 Datum.
Pier No 9
367'6"
472'6"
1102'6"
2782'6"
Future river widening in this area.
Limit of Contracts S & E
PLAN
Cross beams @10'6" crs.
RIVER
YARRA
SCALE
0
100
200
300
400 FEET
1:40 constant on concave side.
1:40 up & down.
Variable gap.
Section from Box 16 to mid span.
Section at max. superelevation at Box 1
Convex side of curve.
25"
TAPER-OUT OF SUPERELEVATION ON CONVEX SIDE

FIG. 2



NOTE

The positions of the crossbeams & cantilevers is as shown on the tender drawings. In fact they are incorrectly placed as can be seen from fig. 9. As built all the crossbeams and cantilevers shown were one half pitch (5'-3") from the positions shown.

HALF PLAN OF DECK

SCALE 0 50 100 200 FEET

WEST GATE BRIDGE

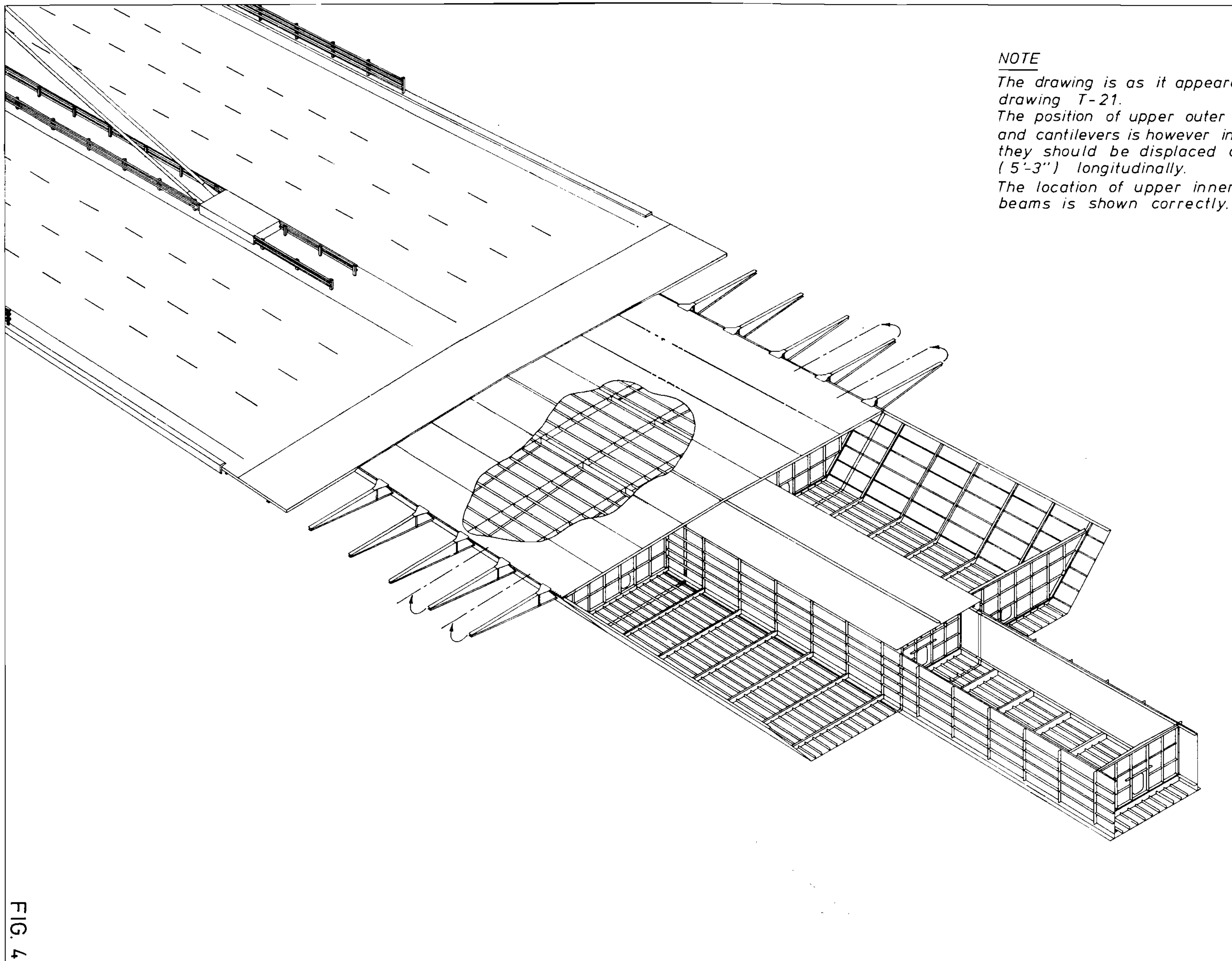
MAIN SPAN

GENERAL ARRANGEMENT OF STEELWORK

BASED ON THE TENDER DRAWINGS OF THE JOINT CONSULTING ENGINEERS

NO. 1364 / 590 / T-4

FIG. 3



NOTE

The drawing is as it appeared on the tender drawing T-21.
The position of upper outer transverse beams and cantilevers is however incorrectly shown, they should be displaced one half pitch (5'-3") longitudinally.
The location of upper inner transverse beams is shown correctly.

WEST GATE BRIDGE

BOX GIRDER
ASSEMBLY DIAGRAM

BASED ON THE TENDER DRAWINGS OF THE JOINT CONSULTING ENGINEERS

NO. 1364 / 590 / T-21

FIG. 4

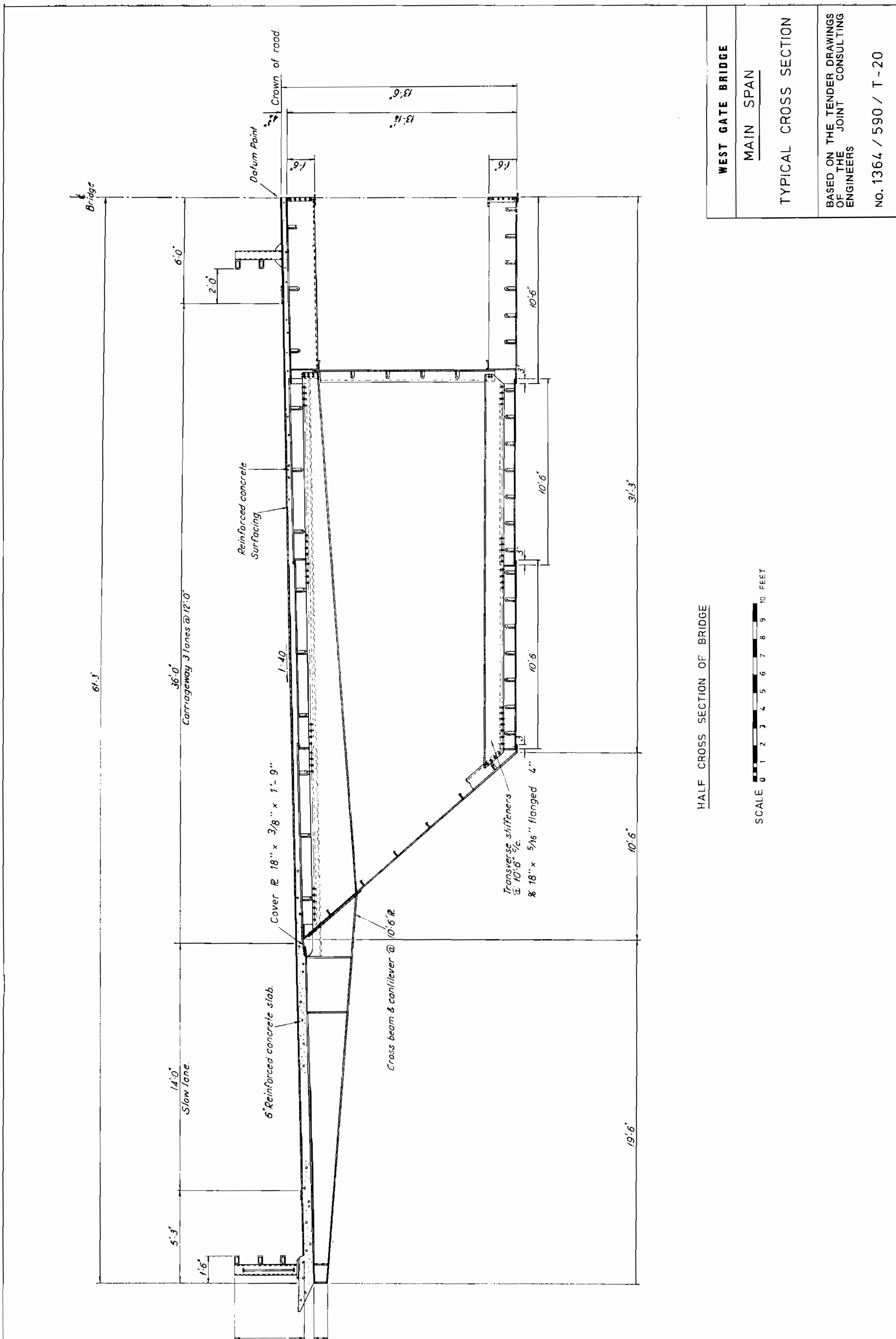

WEST GATE BRIDGE
MAIN SPAN
TYPICAL CROSS SECTION
BASED ON THE TENDER DRAWINGS OF THE JOINT CONSULTING ENGINEERS
NO. 1364 / 590 / T-20
HALF CROSS SECTION OF BRIDGE
SCALE
FEET
Bridge
Crown of road.
Datum Point
13'6"
1'6"
6'0"
2'0"
Reinforced concrete Surfacing
36'0"
Carriageway 3 lanes @ 12'0"
61'3"
1-40
Cover PL 18" x 3/8" x 1'-9"
Transverse stiffeners @ 10'6" c/c.
& 18" x 5/16" flanged
4"
10'6"
31'3"
Cross beam & cantilever @ 10'6" PL
6" Reinforced concrete slab.
14'0"
Slow lane.
19'6"
5'3"
4'0"

FIG. 5

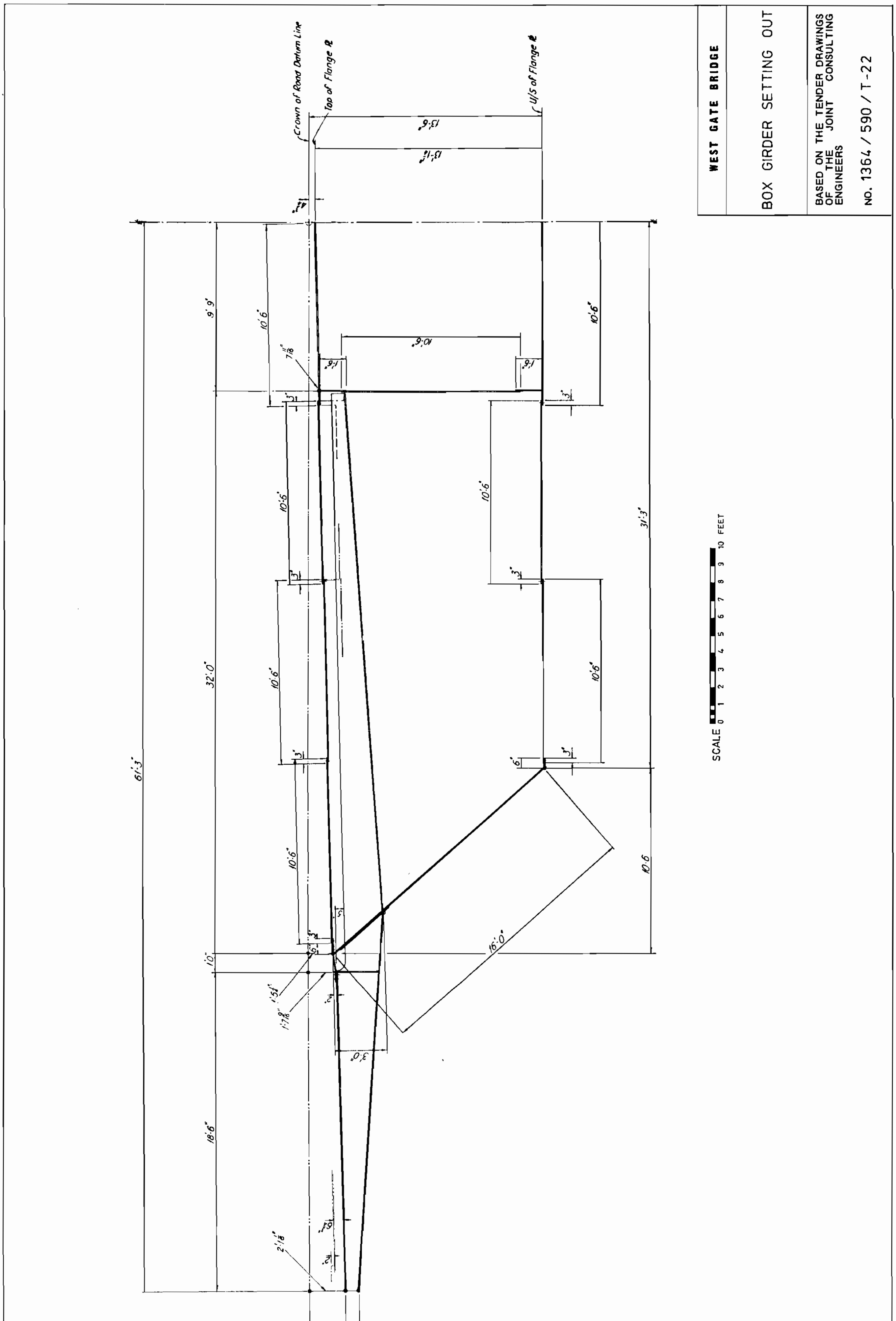
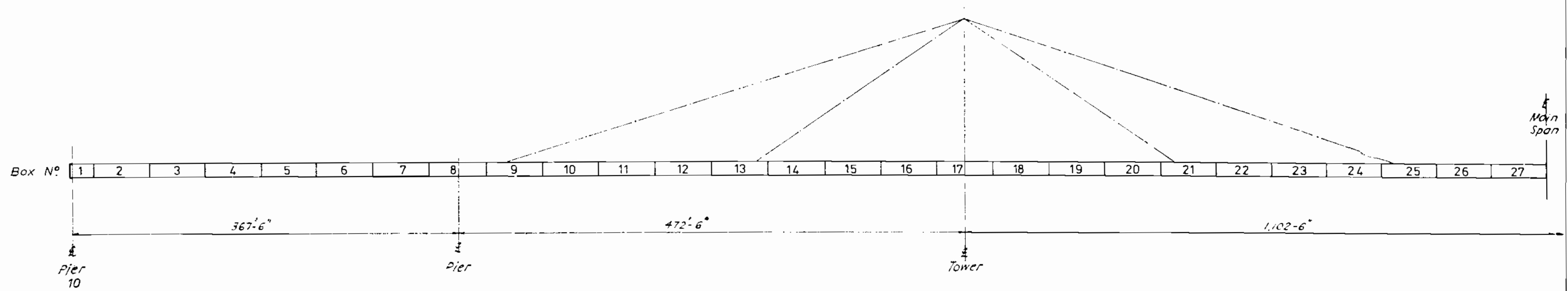

WEST GATE BRIDGE
BOX GIRDER SETTING OUT
BASED ON THE TENDER DRAWINGS OF THE JOINT CONSULTING ENGINEERS
NO. 1364 / 590 / T-22
Crown of Road Datum Line
Top of Flange ℄
U/S of Flange ℄
13'6"
13'1½"
61'3"
32'0"
18'6"
9'9"
31'3"
10'6"
16'0"
3'0"
SCALE 0 1 2 3 4 5 6 7 8 9 10 FEET

FIG. 6



Top Flange Panels	Plate	3/8" MS; 3/8" HYS
	Stiffeners	3/No Bulb Flats 6" @ 7.42# HYS
Outer Web Panels	Plate	3/8" thick
	Stiffeners	Vertical Stiffeners Bulb Flats 8" @ 12.59# MS Longitudinal Stiffeners Bulb Flats 4" @ 5.19# MS
Inner Web Panels	Plate	3/8" MS; 1/2" HYS; 3/8" MS; 1/2" HYS; 3/8" MS; 3/8" HYS; 3/8" MS; 1/2" HYS; 3/8" MS; 1/2" HYS; 3/8" HYS
	Stiffeners	Vertical Stiffeners Bulb Flats 6" @ 7.42# MS Longitudinal Stiffeners Bulb Flats 4" @ 5.19# MS
Bottom Flg Panels	Plate	3/8" MS; 1/2" HYS; 5/8" HYS; 1/2" HYS; 3/8" HYS; 5/8" HYS; 3/8" HYS; 3/4" HYS; 3/8" HYS; 5/8" HYS; 3/8" HYS; 1/2" HYS
	Stiffeners	7/No Bulb Flats 6" @ 7.42# HYS

TENDER DIMENSIONS

Top Flange Panels	Plate	3/8" MS; 3/8" H.Y.S; 1/2" H.Y.S.; 1/2" HYS; 3/8" H.Y.S.; 1/2" HYS; 1/2" HYS; 1/2" HYS
	Stiffeners	5 / Nº Bulbflats 6" × 7.42# H.Y.S.; 3 / Nº Bulbflats 6" × 7.42# H.Y.S.; 3 / Nº Bulbflats 6" × 7.42# H.Y.S.
Outer Web Panels	Plate	3/8" H.Y.S.
	Stiffeners	Vertical stiffeners Bulbflats 8" × 12.59# M.S. Longitudinal stiffeners Bulbflats 4" × 5.19# M.S.
Inner Web Panels	Plate	3/8" M.S.; 1/2" H.Y.S.; 3/8" M.S.; 1/2" H.Y.S; 3/8" M.S.; 3/8" H.Y.S.; 3/8" M.S.; 1/2" H.Y.S.; 3/8" M.S.; 1/2" H.Y.S.; 3/8" H.Y.S.
	Stiffeners	Vertical stiffeners Bulbflats 6" × 7.42# M.S. Longitudinal stiffeners Bulbflats 4" × 5.19# M.S.
Bottom Flg Panels	Plate	3/8" MS; 3/8" H.Y.S.; 5/8" HYS; 1/2" H.Y.S.; 7/8" H.Y.S.; 3/8" H.Y.S.; 3/4" H.Y.S.; 5/8" H.Y.S.; 3/8" H.Y.S.; 3/4" HYS; 3/8" H.Y.S.; 5/8" H.Y.S.; 3/4" H.Y.S.; 3/8" H.Y.S.; 3/4" H.Y.S.; 1/2" H.Y.S.
	Stiffeners	7 / Nº Bulbflats 6" × 7.42# H.Y.S.

SIZES ON WORKING DRAWINGS

Notes on Cover Plates:-

1. All splice plates to be in same material as main plates.
2. Splice plates on both sides of plates on all flanges and webs.
3. Panel lengths shown on drawings may be reduced slightly to one standard length making allowance for variations due to deck curvature and erection camber provided cover plates are adjusted to suit.

WEST GATE BRIDGE

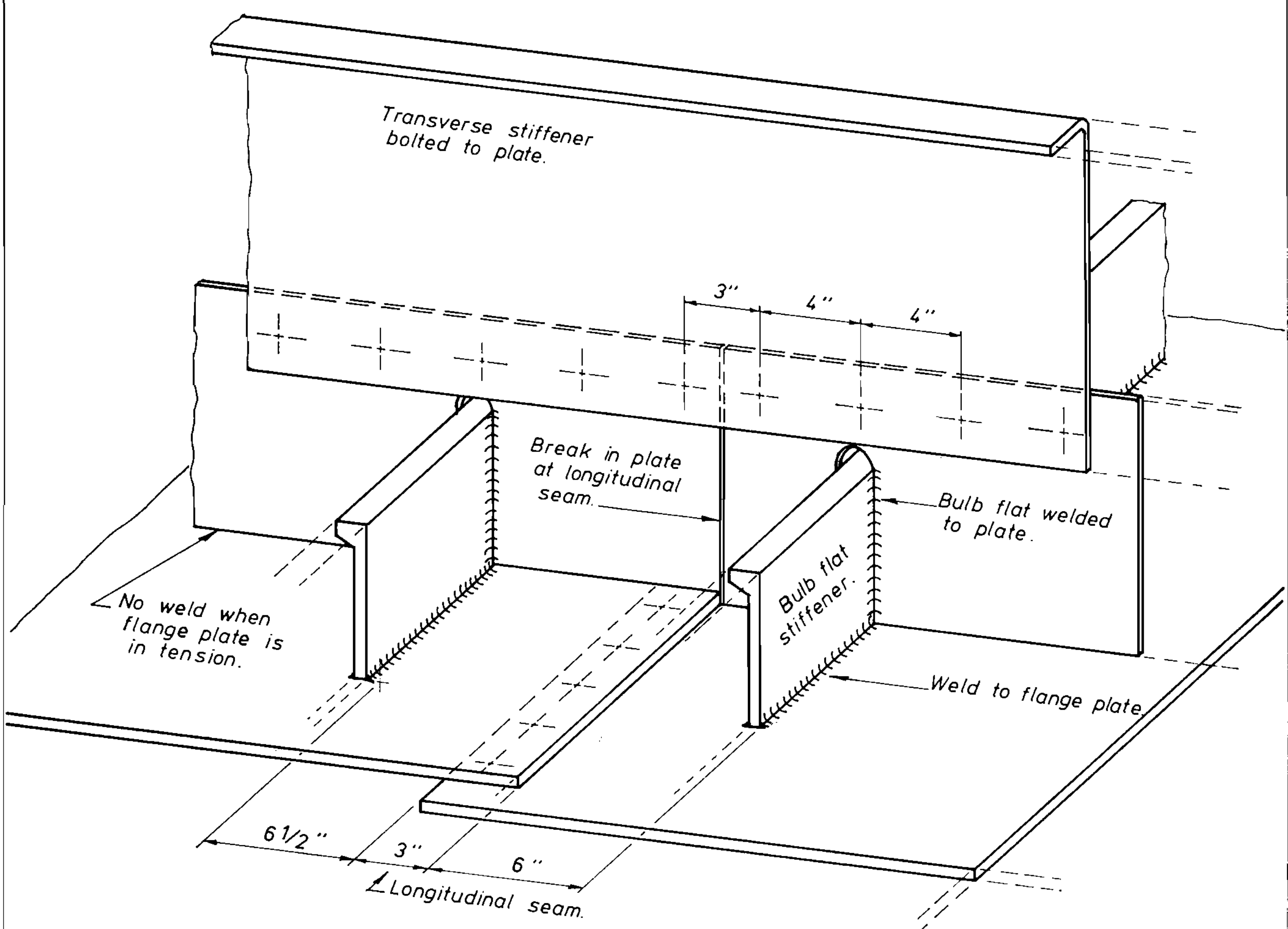
BOX GIRDER

MAKE UP SHEET

BASED ON THE TENDER DRAWINGS OF THE JOINT CONSULTING ENGINEERS

NO. 1364 / 590 / T-24

FIG. 7



DETAIL OF STIFFENERS TO PLATING

WEST GATE BRIDGE
STIFFENERS TO PLATING
SPAN 10-11 & 14-15
FIG. 8

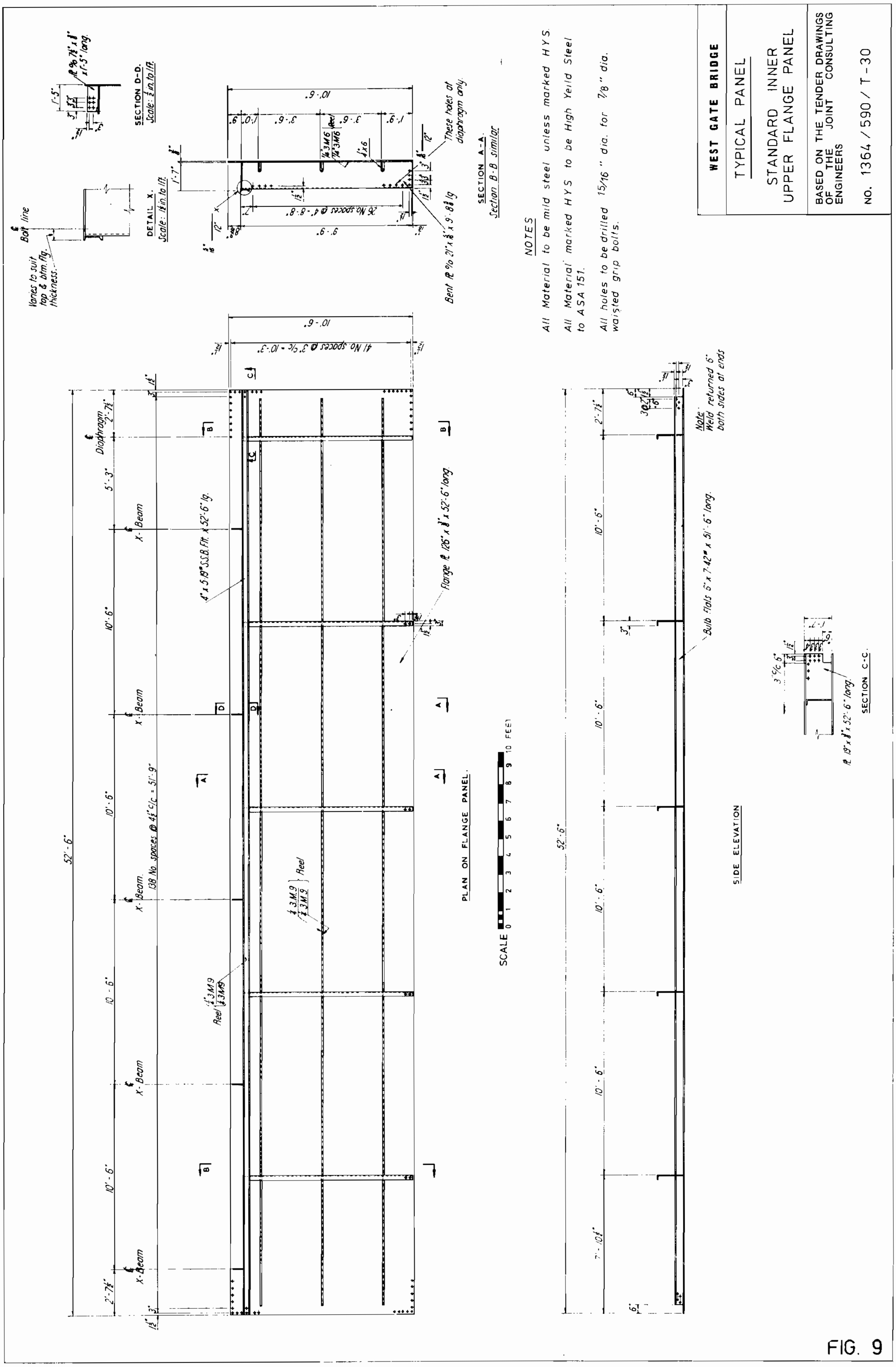

WEST GATE BRIDGE
TYPICAL PANEL
STANDARD INNER UPPER FLANGE PANEL
BASED ON THE TENDER DRAWINGS OF THE JOINT CONSULTING ENGINEERS
NO. 1364 / 590 / T-30
NOTES
All Material to be mild steel unless marked HYS.
All Material marked HYS to be High Yeild Steel to ASA 151.
All holes to be drilled 15/16" dia. for 7/8" dia. waisted grip bolts.
PLAN ON FLANGE PANEL
SIDE ELEVATION
SECTION A-A
Section B-B similar
SECTION C-C
SECTION D-D
DETAIL X
SCALE 0 1 2 3 4 5 6 7 8 9 10 FEET

FIG. 9

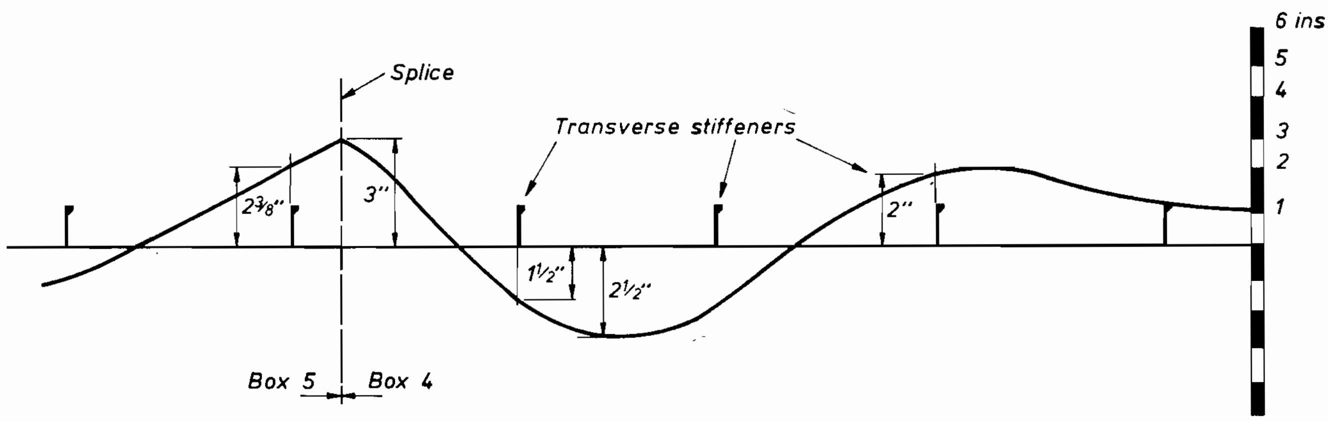


FIG. 10. BUCKLED SHAPE AT OUTER LONGITUDINAL STIFFENER SPAN 14-15N.

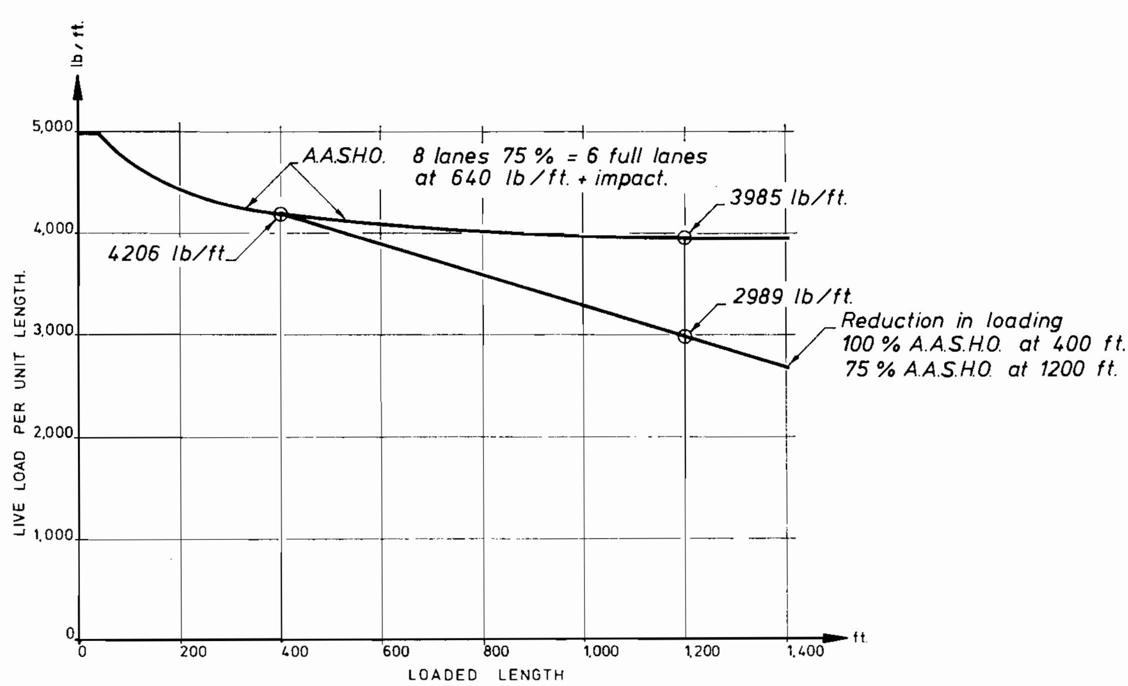
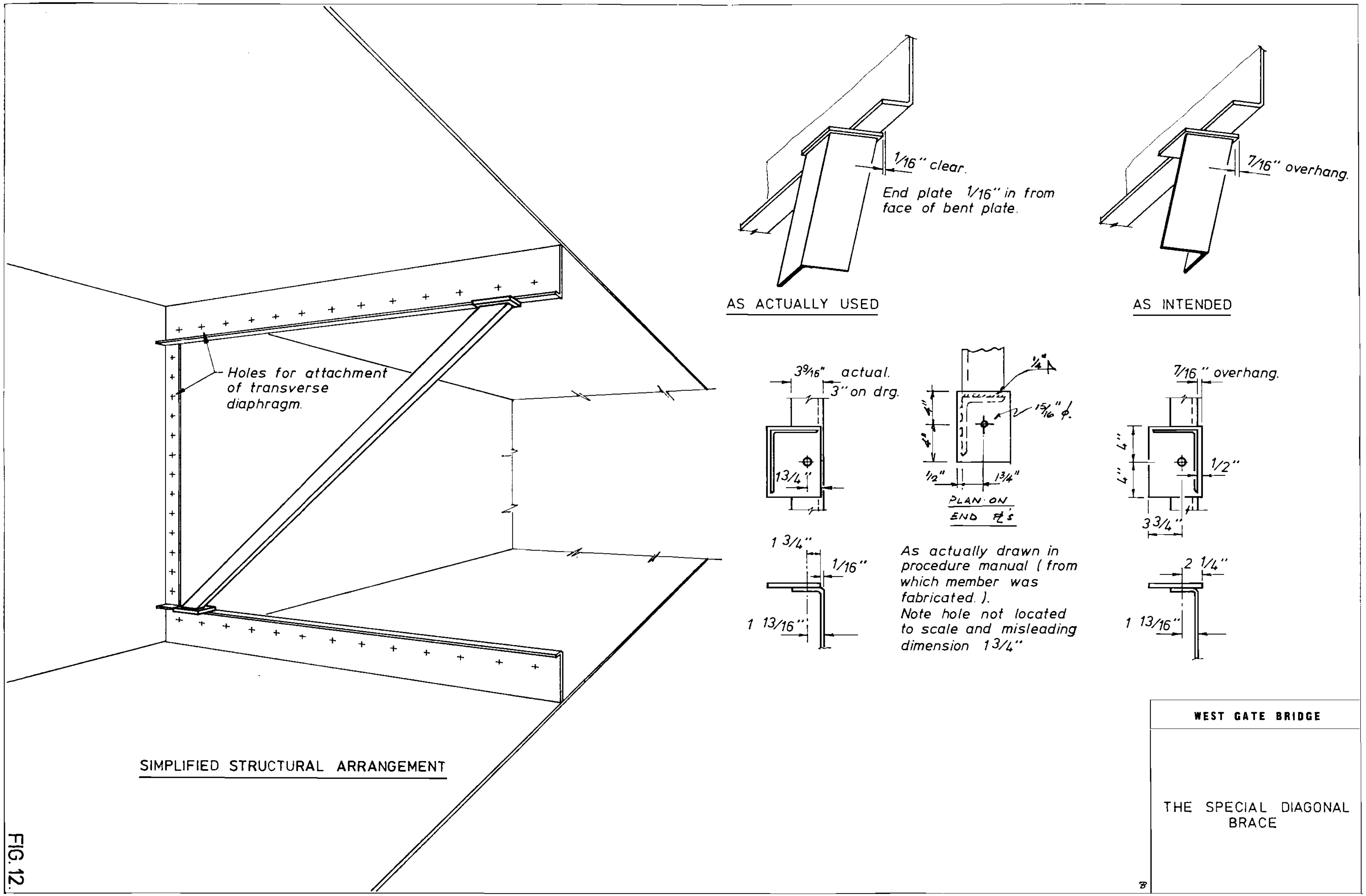


FIG. 11. LIVE LOAD CURVE FOR 8 LANES.

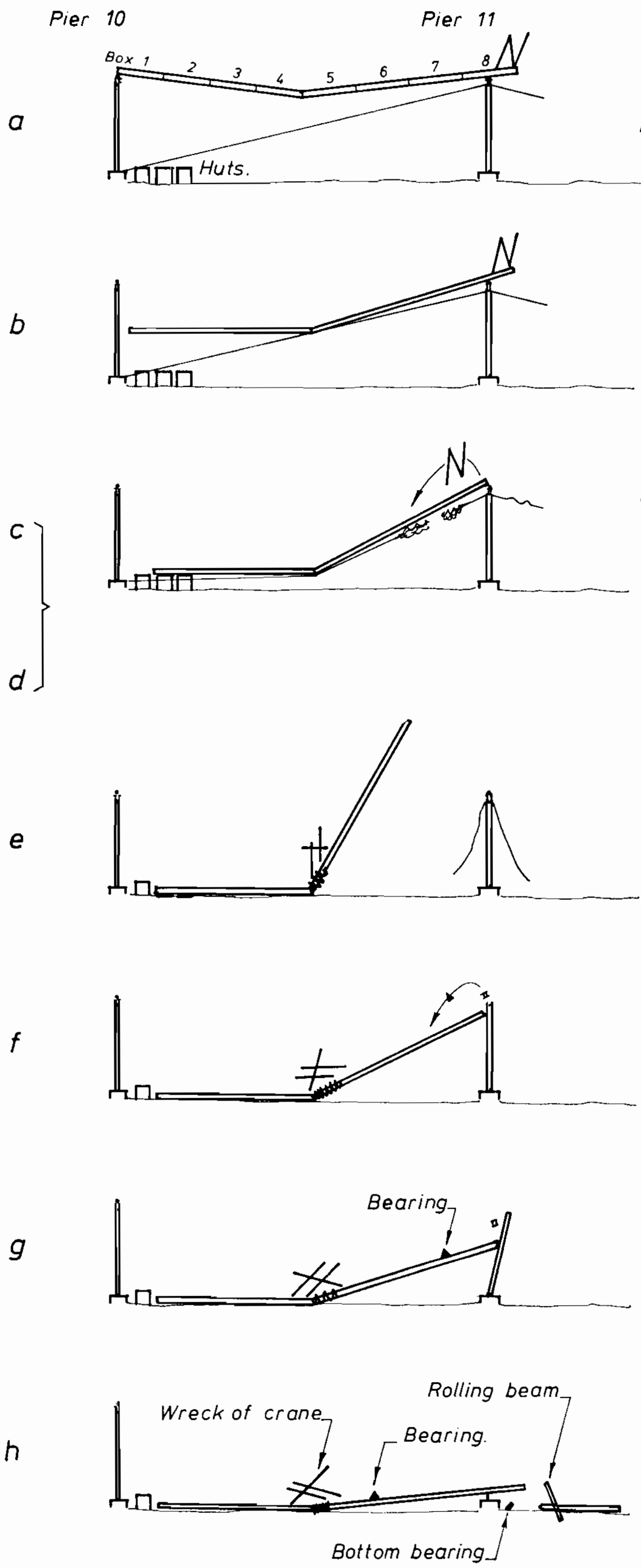
WEST GATE BRIDGE

FIG'S. 10 & 11.



1/16'' clear.
End plate 1/16'' in from face of bent plate.
7/16'' overhang.
AS ACTUALLY USED
AS INTENDED
3 9/16'' actual.
3'' on drg.
13/4''
1/4''
15/16'' ϕ.
4''
4''
1/2''
13/4''
PLAN ON END PL'S
7/16 '' overhang.
4''
4''
1/2''
3 3/4''
1 3/4''
1/16''
1 13/16''
As actually drawn in procedure manual (from which member was fabricated.).
Note hole not located to scale and misleading dimension 1 3/4''
2 1/4''
1 13/16''
Holes for attachment of transverse diaphragm.
SIMPLIFIED STRUCTURAL ARRANGEMENT
WEST GATE BRIDGE
THE SPECIAL DIAGONAL BRACE

FIG. 12.



a — Buckle starts at 4/5 splice.

b — Box 1 pulls off Pier 10.
Boxes 1-4 almost horizontal.

c, d — Boxes 1-4 hit ground - still horizontal.
Instantaneous centre of units 5-8 changes from top of 11 to 4/5 splice.
Crane jerked off.

e — Rotary momentum carries boxes 5 - 8 clear of Pier 11.

f — Boxes 5-8 reverse rotation and crash into Pier 11 - jerking bearings etc. on to deck of bridge.

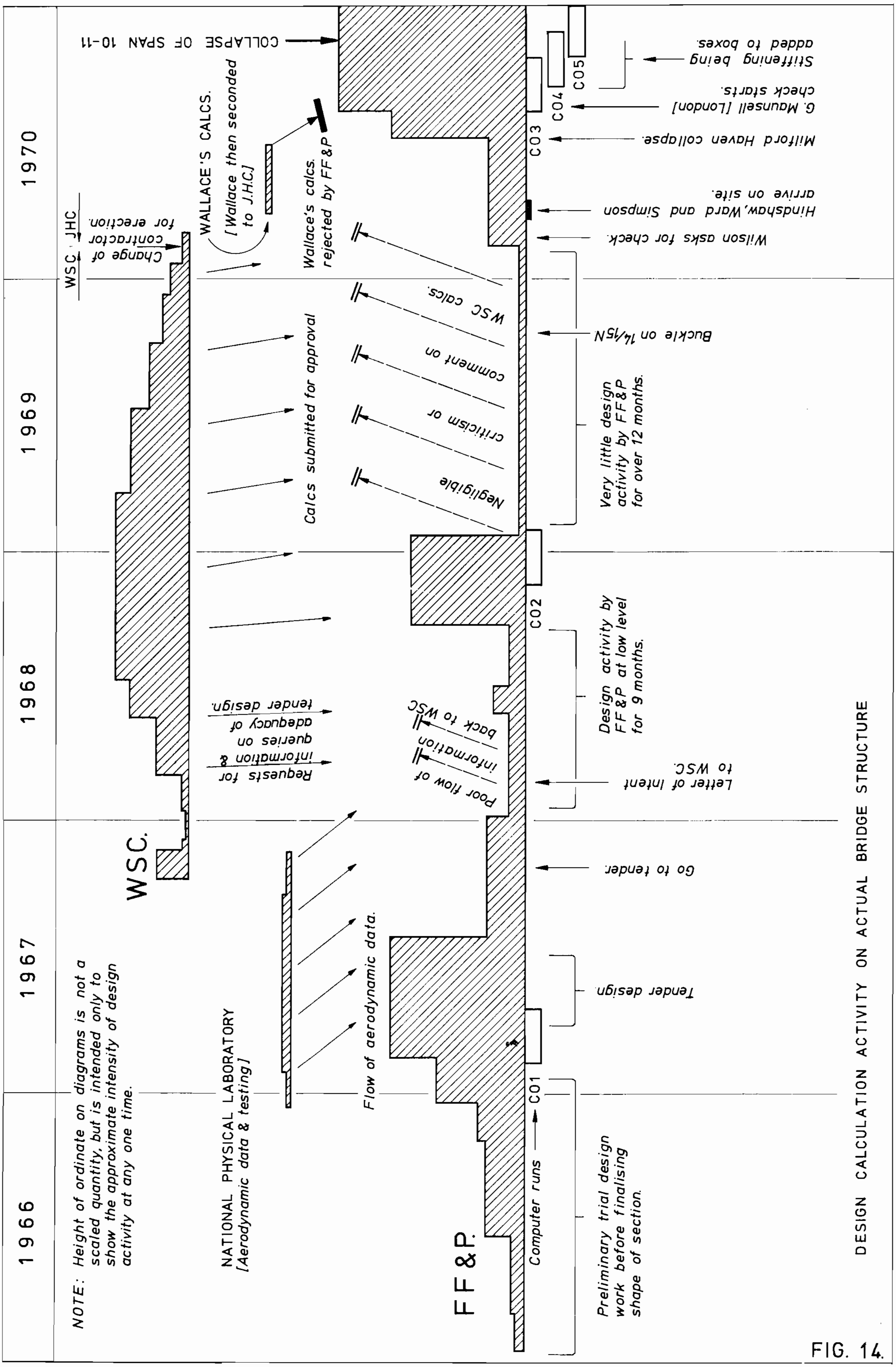
g — Pier 11 pushed over.

h — Pier 11 hits mud and slides sideways.

WEST GATE BRIDGE

DYNAMICS OF FAILURE

FIG. 13



DESIGN CALCULATION ACTIVITY ON ACTUAL BRIDGE STRUCTURE

FIG. 14.

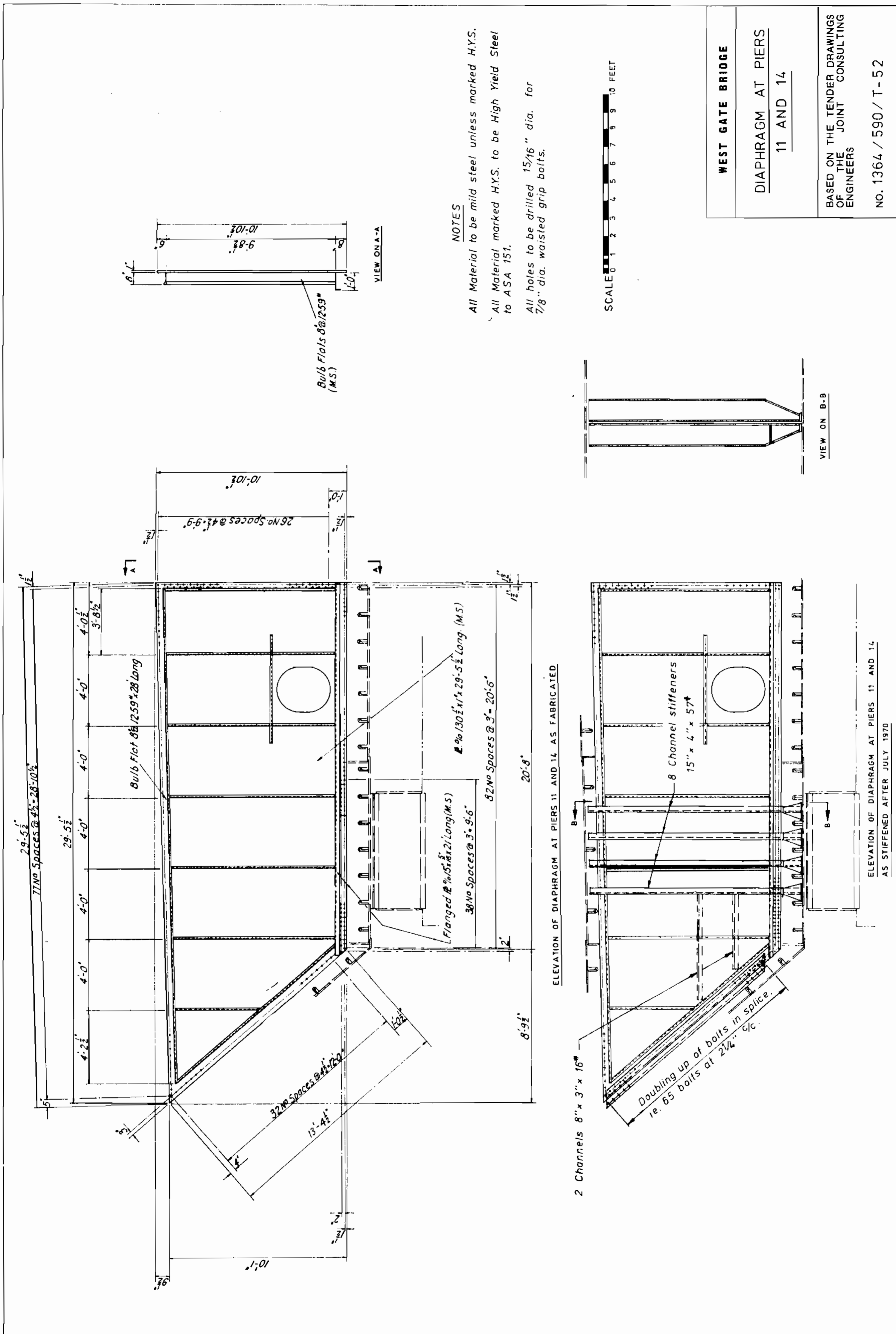

WEST GATE BRIDGE
DIAPHRAGM AT PIERS 11 AND 14
BASED ON THE TENDER DRAWINGS OF THE JOINT CONSULTING ENGINEERS
NO. 1364 / 590 / T-52
NOTES
All Material to be mild steel unless marked H.Y.S.
All Material marked H.Y.S. to be High Yield Steel to ASA 151.
All holes to be drilled 15/16" dia. for 7/8" dia. waisted grip bolts.
SCALE
FEET
VIEW ON A-A
VIEW ON B-B
ELEVATION OF DIAPHRAGM AT PIERS 11 AND 14 AS FABRICATED
ELEVATION OF DIAPHRAGM AT PIERS 11 AND 14 AS STIFFENED AFTER JULY 1970
8 Channel stiffeners 15" x 4" x 57#
2 Channels 8" x 3" x 16#
Doubling up of bolts in splice. ie. 65 bolts at 2¼" c/c.

FIG. 15

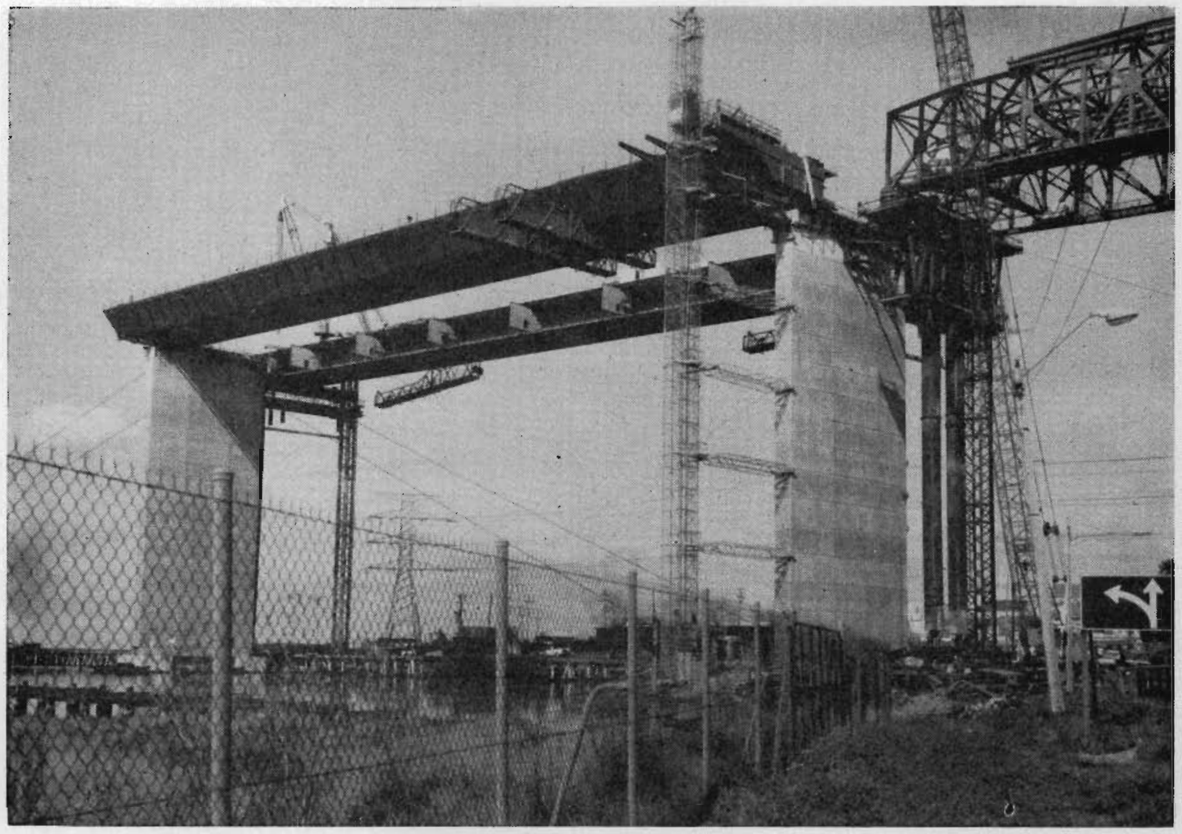


PLATE 2. SPAN 10-11, SHOWING SOUTH HALF SPAN BEING JACKED UP.
Note : Transverse diaphragms are projecting out of south half span.

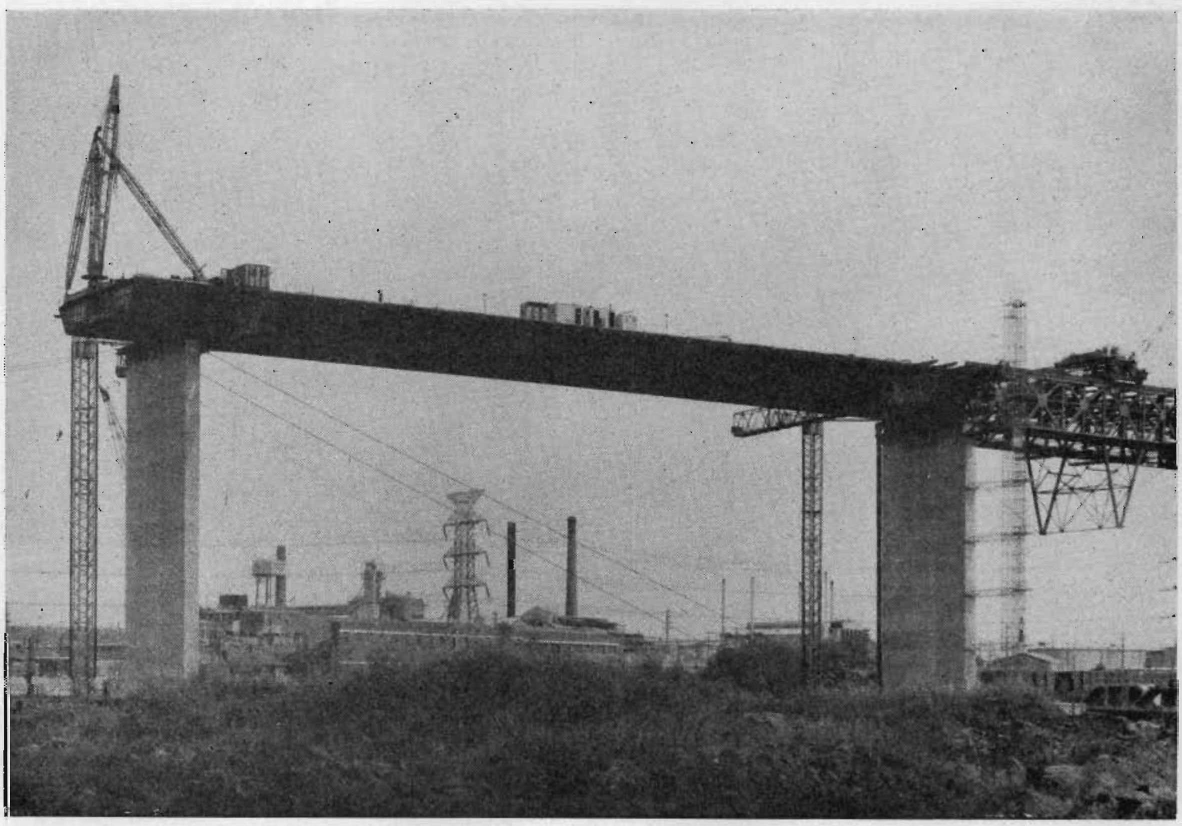


PLATE 3. SPAN 10-11 SHORTLY BEFORE THE COLLAPSE.

PLATE 1. VIEW OF COLLAPSED SPAN FROM TOP OF PIER 10.

Note : Longitudinal splice is open on boxes 1-3, in foreground. The stiff-legged derrick was attached to box 8 (top right-hand corner) before collapse.

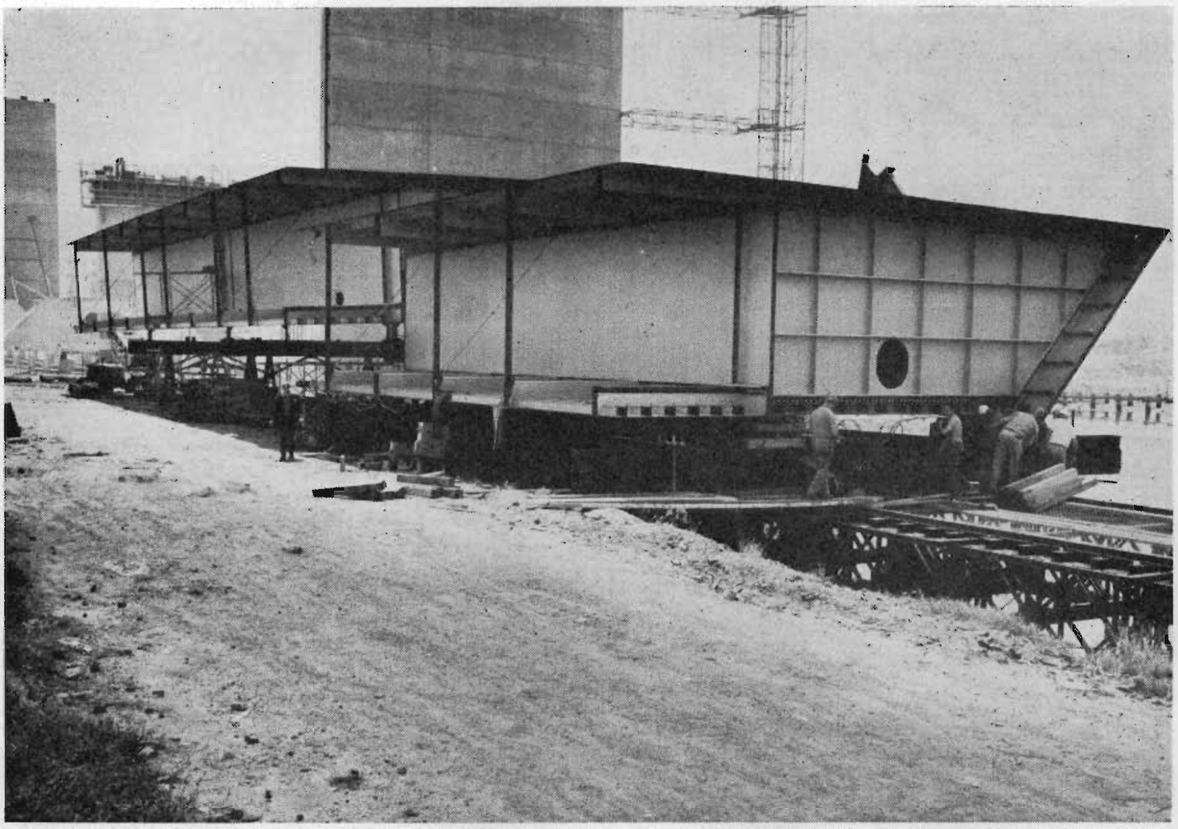


PLATE 4. TYPICAL BOX SHOWN READY FOR BOLTING TO HALF SPAN 10-11 NORTH.
Note : Temporary wire braces and vertical posts were fitted during handling of boxes.

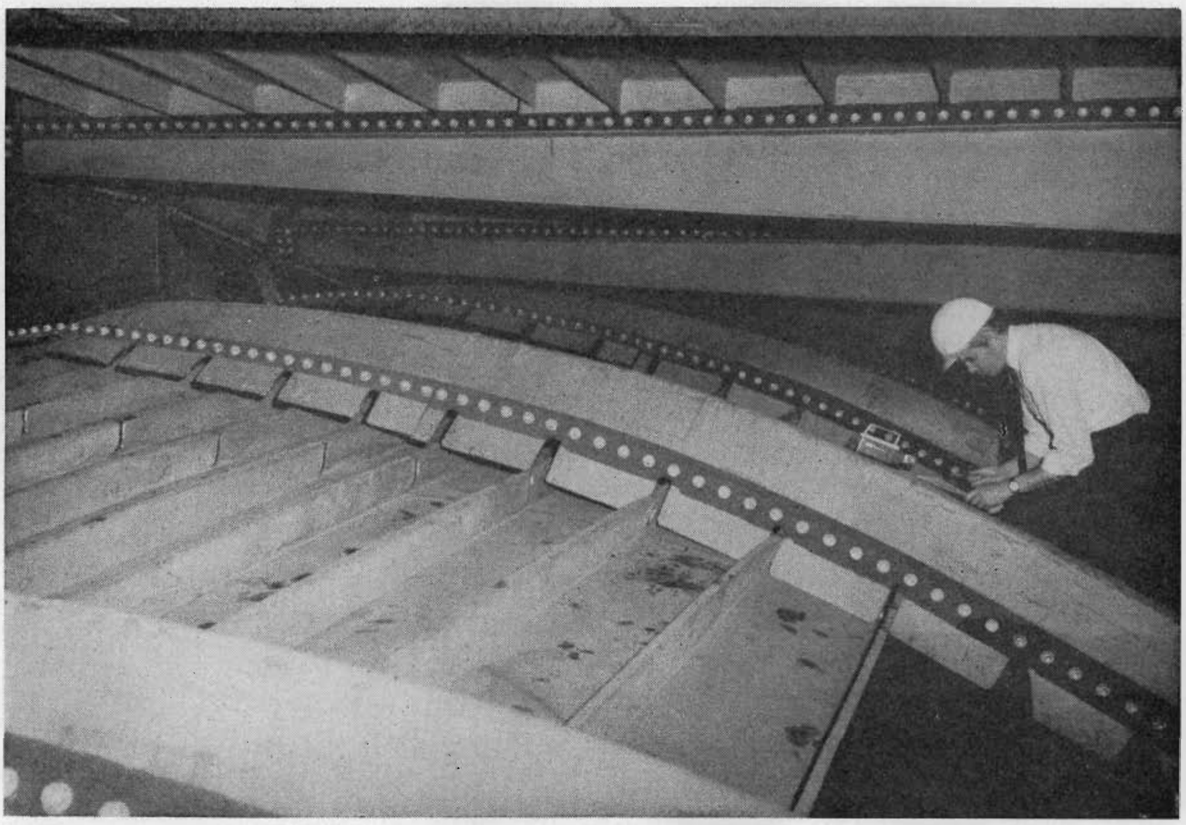


PLATE 5. INSIDE BOX 4, OUTER CELL AFTER COLLAPSE.
Note : Bottom flange has been forced up about 8 feet. Transverse beams were not welded to flange plate but have torn off the welds connecting them to the bulb flat stiffeners.

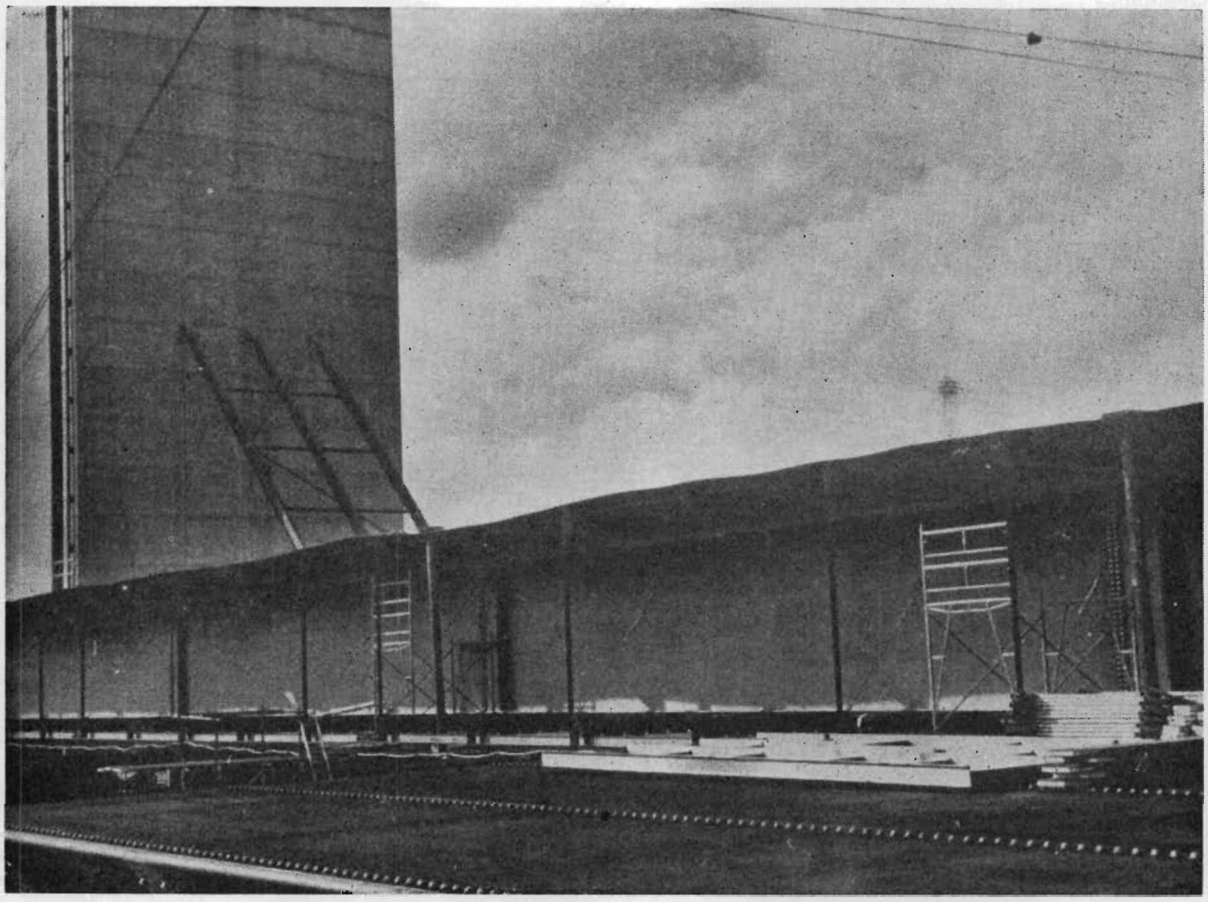


PLATE 6. BUCKLE ON INNER UPPER PANEL 14-15 NORTH.
Note : Effective diagonal bracing was not fitted.

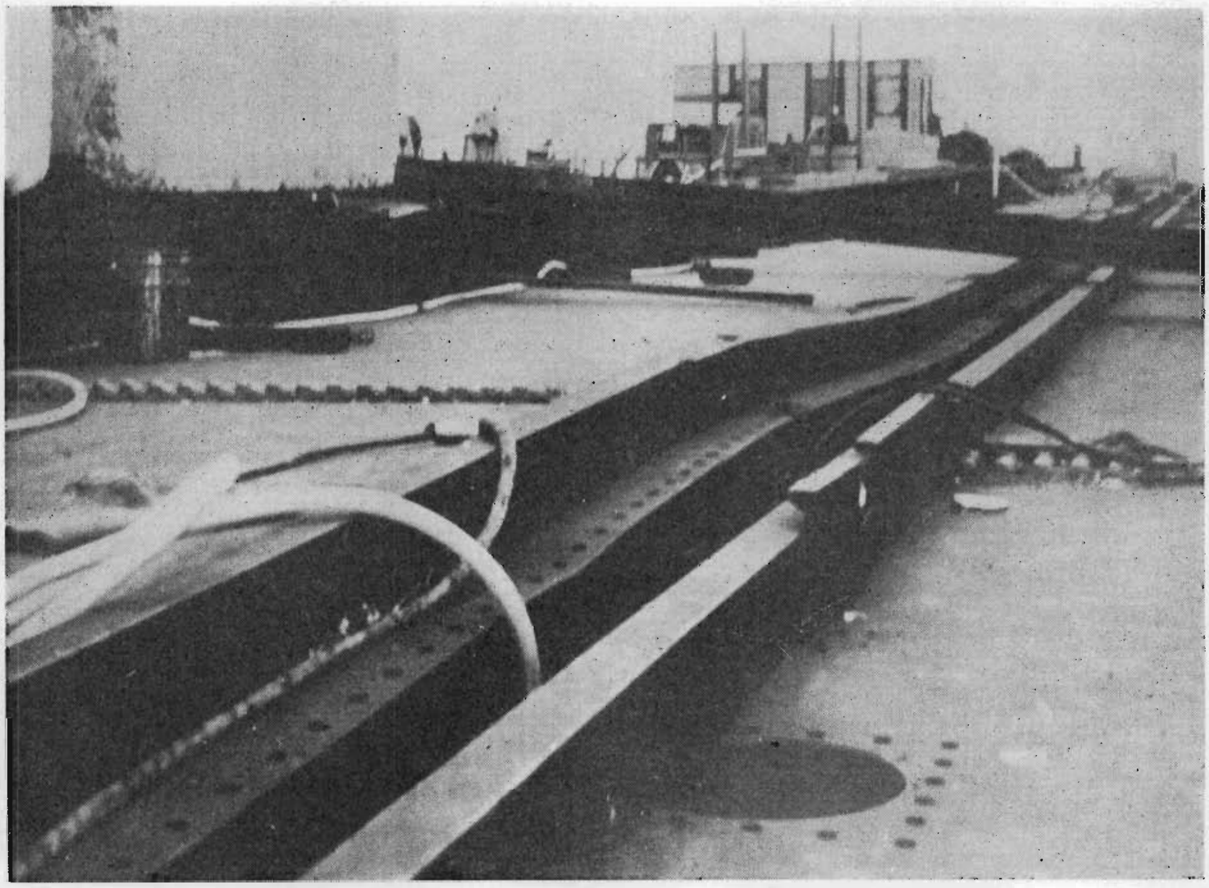


PLATE 7. BUCKLE ON INNER UPPER PANEL 10-11 NORTH.
Note : Two of the blocks of kentledge can be seen, top left. The transverse seam 4-5 which was undone appears at mid-height.

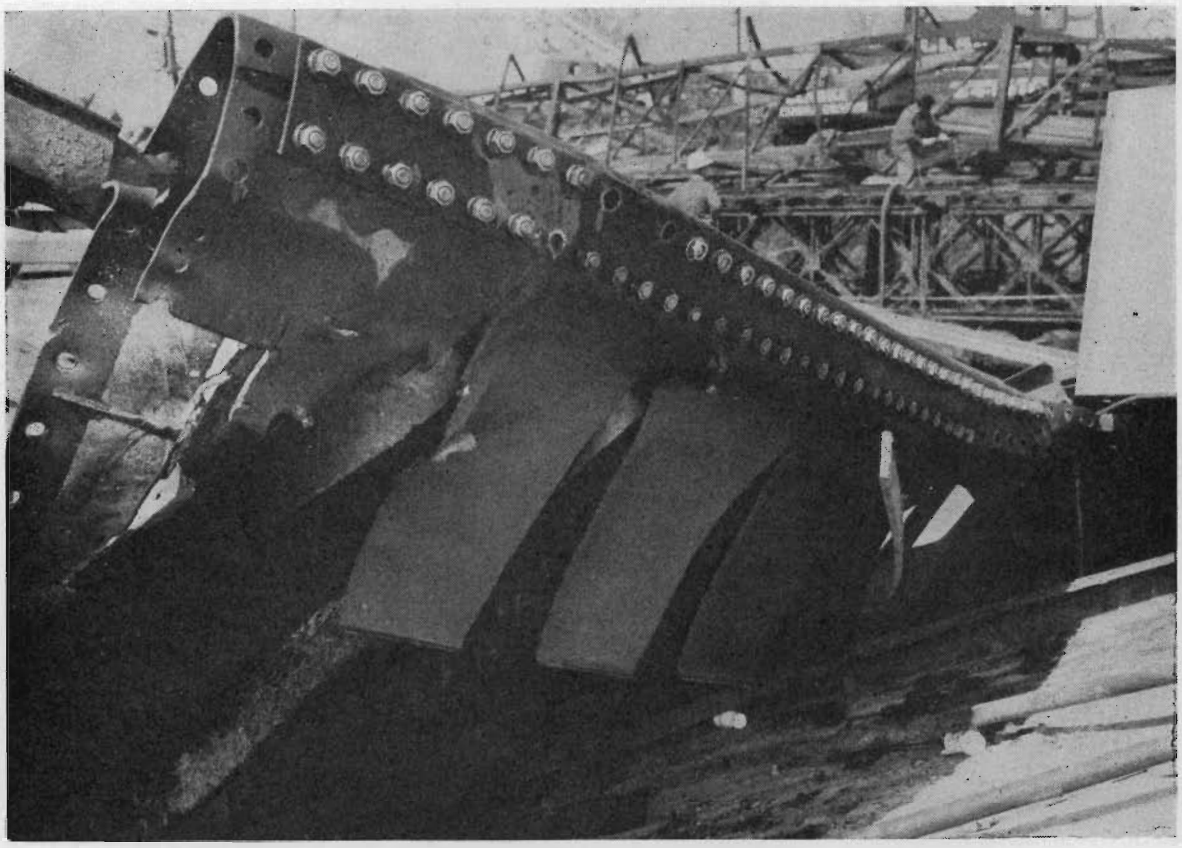


PLATE 8. TRANSVERSE SPLICE 4-5, INNER UPPER PANEL, AFTER COLLAPSE.

Note : These are the bolts which were undone on 15th October. The bridge centre line is on the right of the photograph. The bulb flat stiffeners have torn triangular pieces out of the deck plate.

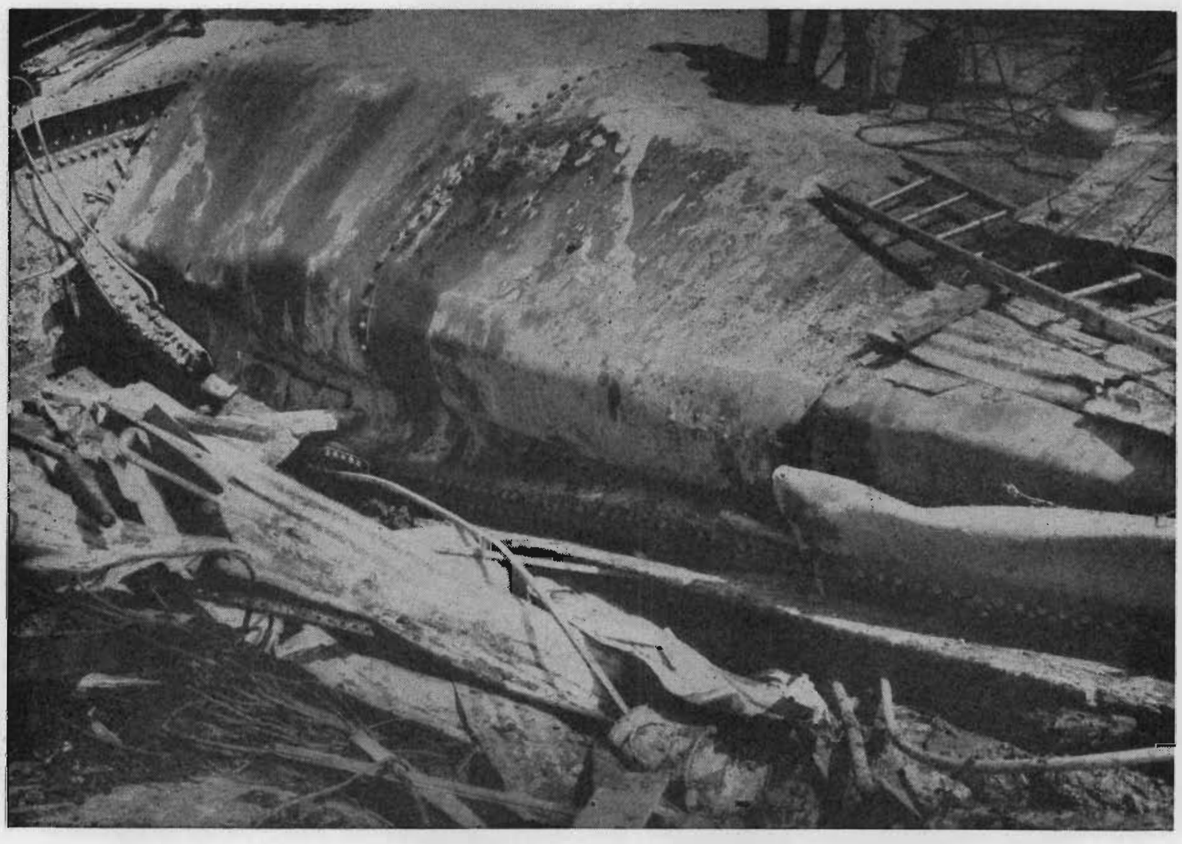


PLATE 9. TRANSVERSE SPLICE 4-5 NORTH, OUTER PANELS, AFTER COLLAPSE.

Note : The excessive folding of flange plate can be seen. The inner web appears on the left of the photograph. This is the area affected by fire.

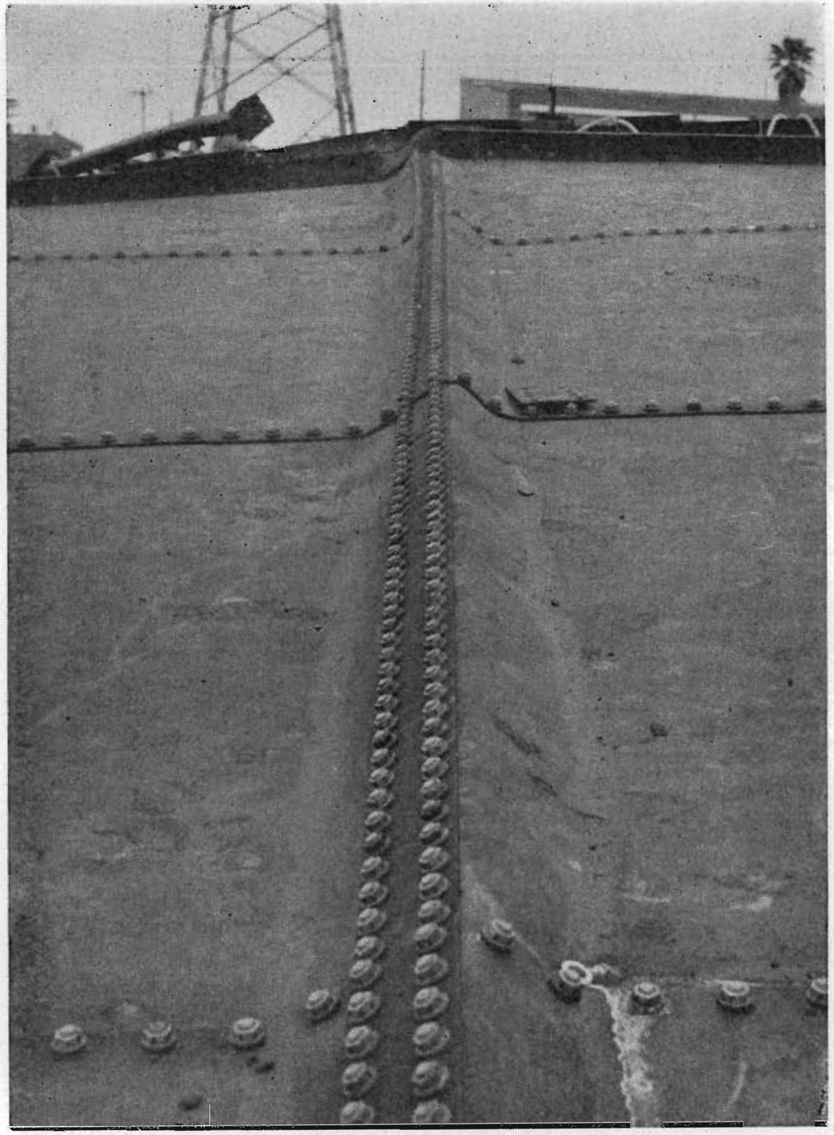


PLATE 10. TRANSVERSE SPLICE 2-3 AFTER COLLAPSE.

Note : This damage was almost certainly due to impact of the falling span. The photograph shows the form of buckling that can accompany the collapse of the K-plates.

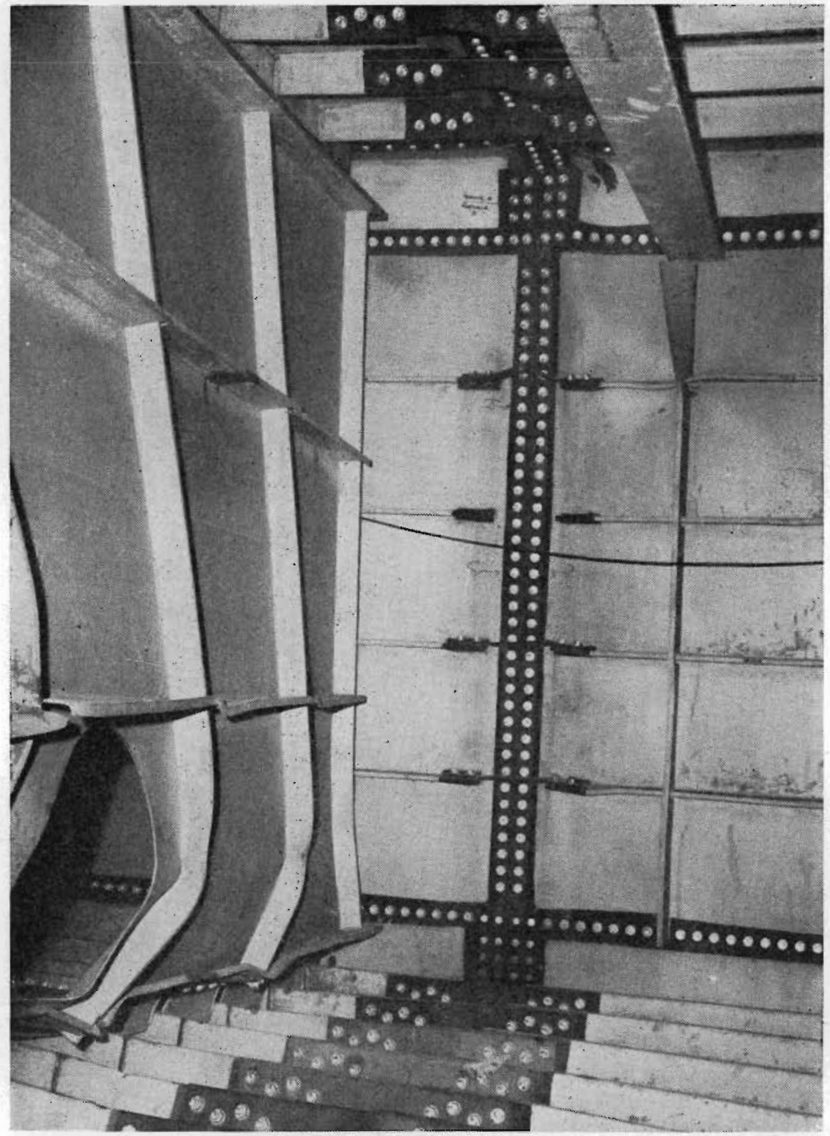


PLATE 11. TRANSVERSE SPLICE 2-3 AFTER COLLAPSE—FROM INSIDE OUTER CELL OF BOX 3.

Note : Buckled K-plates can be seen on inner web and upper flange. The third K-plate up on web was never fitted. The crippling of the M.S. web at the top can be seen.

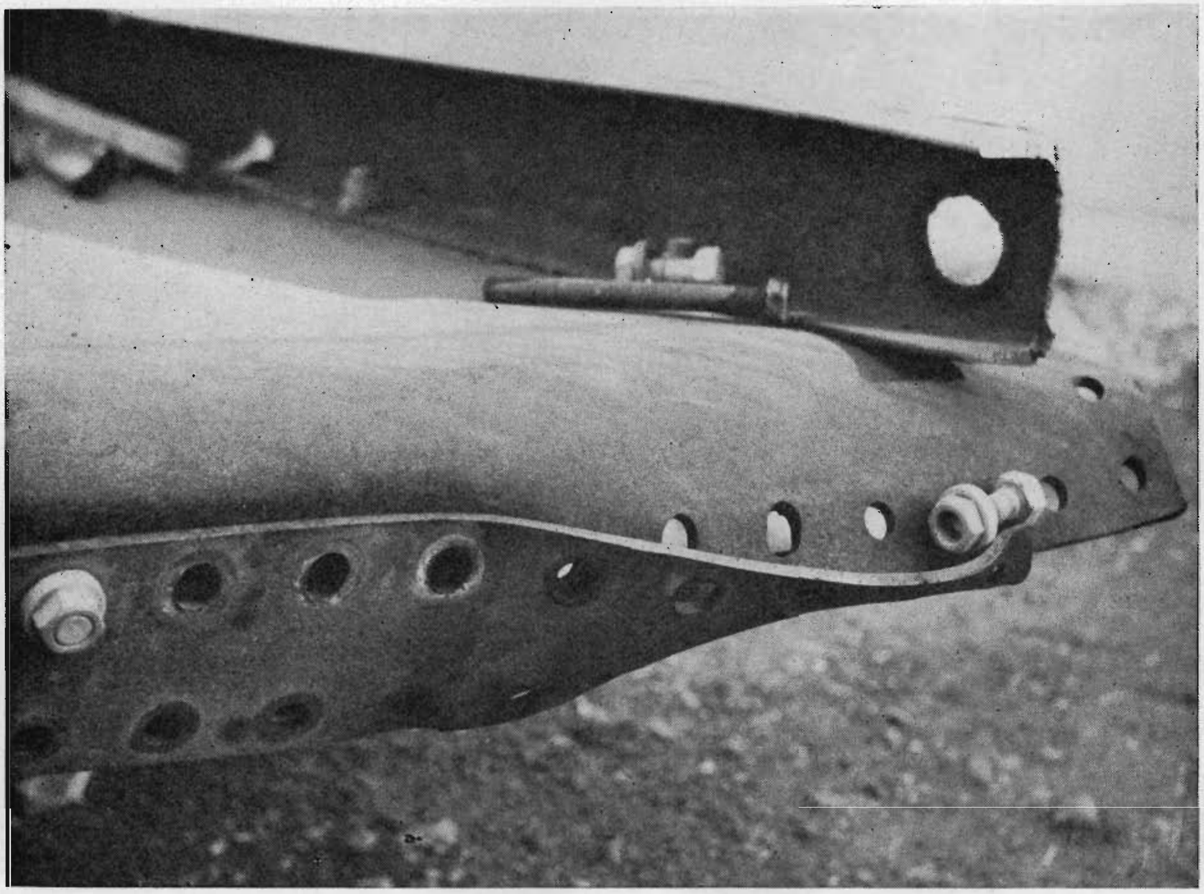


PLATE 12. DETAIL OF 4-5 NORTH SPLICE CUT OUT FROM WRECK.

Note : The extent of overdrilling can be seen on some holes. It appears that the last few bolts were never replaced.

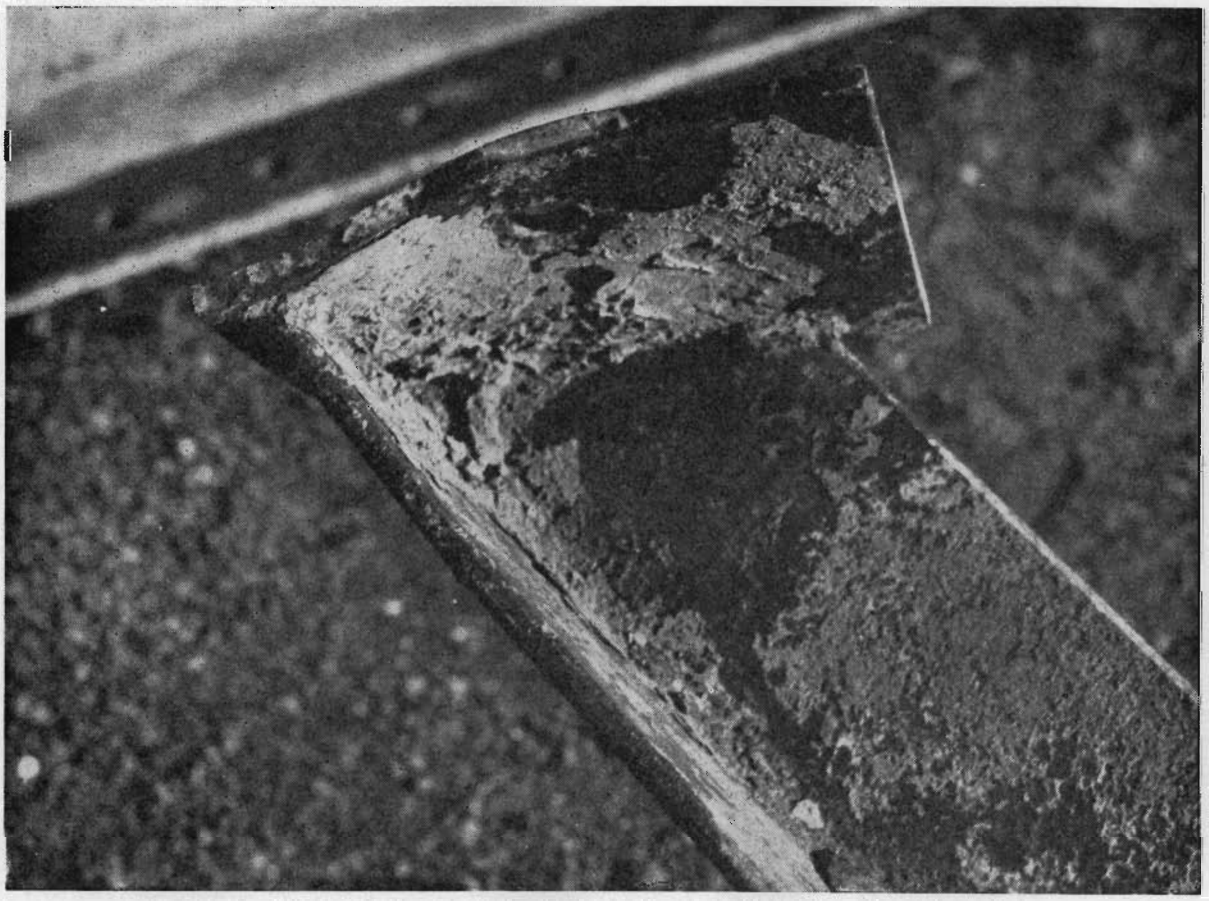


PLATE 13. END OF DIAGONAL BRACE FROM BOX 4 NORTH SPAN 10-11.

Note : The distortion of the end plate can be clearly seen.

By Authority: C. H. RIXON, Government Printer, Melbourne.