REPORT OF THE
BRITISH COLUMBIA
ROYAL COMMISSION
SECOND NARROWS BRIDGE INQUIRY
1958

VOLUME I

THE HONOURABLE SHERWOOD LETT
Chief Justice of the Supreme Court
of
British Columbia
COMMISSIONER

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TO HIS HONOUR THE LIEUTENANT-GOVERNOR
OF THE PROVINCE OF BRITISH COLUMBIA

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PART I

I have the honour to present herewith my report pursuant to a Commission issued under the "Public Inquiries Act" on the 18th day of June, 1958, whereby I was directed to inquire into any and all of the circumstances surrounding, leading to or having causal connection with the collapse on the 17th day of June, 1958 of a portion of the bridge known as the Second Narrows Bridge in the course of construction across Burrard Inlet under contract with the British Columbia Toll Highways and Bridges Authority, and to determine certain other matters as therein set forth, referred to in Part II of this report. A copy of the Commission is annexed hereto as Schedule 1.

COMMISSION PERSONNEL

Immediately upon my appointment, I appointed Dr. Philip L. Pratley, D.ENG., M.INS.T.C.E., Consulting Engineer of Montreal, as Engineering Consultant and Adviser to the Commission. On the same day he appointed as his field representative Mr. A. B. Sanderson, P.ENG., of Victoria, B.C., who forthwith commenced his investigation at the site. I appointed as additional Engineering Consultants and Advisers to the Commission Dr. F. M. Masters, Consulting Engineer of the firm of Modjeski and Masters of Harrisburg, Pennsylvania, Mr. J. R. H. Otter of Rendel, Palmer & Tritton, Consulting Engineers of London, England, and Mr. Ralph Freeman, of Freeman, Fox and Partners, Consulting Engineers of London, England. Mr. J. R. Giese, a partner in the firm of Modjeski and Masters, acted with Dr. Masters as Engineering Consultant and Adviser, and in conjunction with Messrs. Otter, Freeman and Sanderson conducted a field investigation on behalf of Dr. Masters.

The eminent British Consultants were obtained through the good offices of Sir Arthur Whitaker of London, England, President of the Institution of Civil Engineers. The Commission is indebted to him and the Institution for this assistance, which was rendered gratuitously.

During the preliminary stages of the investigation, I regret to record that Dr. Philip L. Pratley was stricken with an illness which proved fatal. The services of Mr. Sanderson were continued as an additional Consultant and Adviser. The five Engineering Consultants and Advisers are hereinafter sometimes referred to as the Advisers.

I appointed Mr. John L. Farris, Q.c. and Mr. W. J. Wallace, B.A.Sc. as Commission Counsel, and Mr. Robert Wilson as Secretary. Two experienced Investigators were engaged to obtain statements from workmen and eye-witnesses. Particulars of all Commission Personnel are shown in Schedule 4.

INVESTIGATION, PUBLIC HEARINGS

A preliminary public hearing was held at the Court House in Vancouver after due public notice, on 9th July, 1958, at which appearances were entered by Counsel for the Consulting Engineers, Swan, Wooster and Partners (hereinafter sometimes referred to as the Engineers), the Dominion Bridge Company Limited (hereinafter sometimes referred to as the Bridge Company), Superstructure Contractors, and Kiewit-Raymond, Sub-structure Contractors, and the Workmen's Compensation Board; representatives appeared for Local 97 International Association of Bridge, Structural and Ornamental Ironworkers, the Vancouver-New Westminster Building Trades Council and Painters and Decorators Brotherhood, Local 138. An appearance was entered subsequently for the British Columbia Toll Highways and Bridges Authority (hereinafter sometimes referred to as the Authority). Particulars of counsel and representatives are shown in Schedule 5.

At this hearing, by consent of counsel and representatives, Exhibits 1 to 299 (Schedule 10) were filed and were ordered available for examination to all counsel and representatives. The hearing adjourned to 21st July, 1958.

Between June 20th and July 21st, the Commission Investigators interviewed and obtained statements from 85 eye-witnesses and workmen engaged on the bridge at the time of its collapse.

Hearings were continued on 21st, 22nd and 23rd July, 1958, when testimony was given by 40 witnesses. It was necessary to adjourn to 30th September, 1958 to enable salvage operations to be undertaken and further studies, calculations, examinations and tests to be made. Further hearings of witnesses, representatives of interested persons and technical experts were held September 30th to October 6th, and on October 8th, 1958. Exhibits 300 to 338 were filed by consent of all counsel and representatives (see Schedule 10). Submissions of counsel and representatives were made at a final hearing on October 16th, 1958. A list of witnesses...
totalling 67 is contained in Schedule 6, and of hearings in Schedule 7.

The Advisers made investigations at the bridge site, an examination of the exhibits, and conferred informally with representatives of the Authority, its Consulting Engineers, representatives of the Bridge Company, and other interested parties.

Permission was given to the Contractors to proceed with salvage operations subject to certain recovery plans agreed upon between the Contractors and the Commission's Advisers.

During the period 23rd July to 30th September, 1958, the Commission's Advisers continued their studies and examinations in preparation for the further hearings. They were provided with photographs and particulars of the salvage operations as they proceeded.

**BRIEF HISTORY AND DESCRIPTION**

The bridge was being constructed in the Greater Vancouver Area between the north and south shores of Burrard Inlet under contracts let by the Authority. The main Contractors for the sub-structure were Peter Kiewit Sons Company of Canada Limited and Raymond International Co. Ltd., and for the fabrication and erection of steel work, the Bridge Company. The designers of the bridge were Swan, Wooster and Partners, Consulting Engineers of Vancouver, B.C.

When completed the bridge will consist of four northern approach spans leading from a viaduct, a main cantilever section with a central span of 1100 ft. and two anchor spans of 465 ft. The spans are numbered from north to south, 1 to 7 inclusive, span 5 being the north anchor span, span 6 the central span, and span 7 the south anchor span. (See diagram Exhibit 300). The bridge deck, about 80 ft. in width, is designed to carry six lanes of highway traffic and two sidewalks. In design the bridge is considered a long span structure.

The piers of the sub-structure, which had been completed, are numbered consecutively from north to south, 1 to 17. Pier 14 supports the south end of span 4 and the north end of span 5. Pier 15 will support the south end of span 5.

The erection scheme called for two temporary piers known as false bents, numbered N4 and N5, between piers 14 and 15, to provide temporary support for the cantilevering of span 5 from pier 14 toward pier 15.

Bent N4 was located immediately below panel point 4 of span 5, and bent N5 immediately below panel point 8 of span 5.

Prior to the time of collapse, the four approach spans had been erected. Span 5 was in process of erection. This work had been completed to panel point 7 and the erection of panel 7-8 had proceeded to a stage where the two top chords, the east bottom chord, the two diagonal members and the two vertical members had been connected in place. The west bottom chord was being made ready for erection. At this time there were located on the deck of span 5 No. 1 Traveller or Crane (155 tons), a diesel locomotive and two railway trucks (38 tons), the bottom west chord for panel 7-8 (52 tons), and miscellaneous erection equipment (85 tons), making a total superimposed load of 330 tons. (Photographs and diagrams relating to this part of the report are shown in Schedule 8 which for convenience is attached hereto).

Preparations were being made for the crane to lift the west bottom chord from the railway trucks, upward and outward to clear the bridge deck, and then downward to its position on the west side of the structure where it was to have been connected up. At the actual moment of the collapse of span 5 none of the equipment on the span was in active motion and the actual lifting of the chord from the railway trucks had not commenced.

It was at this stage on the afternoon of 17th June, 1958, that the south end of span 5, without warning, collapsed into the water, followed almost instantaneously by the collapse into the water of the south end of span 4, with the resulting unfortunate loss of the lives of 18 of the men working on the structure at the time. (See photograph No. 23, item (c), in Schedule 8 attached).

**SOME CONCLUSIONS FROM THE EVIDENCE**

From the evidence adduced it has been possible to arrive at certain findings of fact and certain inferences which may reasonably be drawn from the facts established.

**Collapses**

1. The collapse occurred about 3.40 p.m. on June 17, 1958;
2. Span 5 collapsed before span 4;
3. The collapse of span 5 took place in three stages:
   a. a short initial dip or drop, variously estimated by eye-witnesses as from 1 to 6 ft;
   b. a pause, or "hesitation";
   c. the collapse into the water of its south end, the north end remaining supported on pier 14;
4. The top of pier 14 was deflected towards the south;
5. The south end of span 4 collapsed into the water from the north side of pier 14.

**Noises**

6. Associated with the first stage of the collapse of span 5 there was a noise or noises variously described by witnesses as an explosion, a rifle crack, thunder, a thud, a loud bang, like iron cracking, like steel breaking, a metallic bang, a big roar, a loud rumble like thunder, a rumbling sound, a terrific explosion.

It is not possible to state definitely from the evidence adduced the precise origin of the noise or noises heard. But it may reasonably be inferred that some witnesses heard noises originating at the top of pier 14, or above it, when bolts in the tie plates were broken, and that
others, depending upon their location on the bridge, heard the rumbling noise caused by the fracture of metal and movement of erection equipment occurring almost simultaneously with the collapse of span 5.

**Tie Plates**

7. Four of the eight tie plates which had previously connected the south end of span 4 and the north end of span 5 had been re-connected, with ten bolts each.

**Pier 14**

8. Photographs of pier 14, taken following the collapse, disclosed that in the jacking chambers the steelwork was undamaged, and in the east chamber everything was in its original position except that the jack had fallen off the northern pre-stressing beam (report of A. B. Sanderson, Schedule 9, p. 23) (photographs Nos. 96 to 99, Exhibit 296).

In the west jacking chamber the tie-down beams were shown to be displaced (A. B. Sanderson's Report, Schedule 9, Sketch IV).

The jack on the south pre-stress beam was in place, and the jack from the north pre-stress beam had fallen to the floor; on the south-west corner the wedges had fallen out, but the blocking was in place, though displaced to the west; on the south-west corner both wedges and blocking had fallen; on the north-east corner the blocking and wedges were displaced south (photographs Nos. 100-103, Exhibit 296).

**Deck Load**

9. For approximately 20 to 30 minutes before the collapse of span 5, the locomotive, trucks and west bottom chord mentioned above were in position and stationary on the deck of that span.

**Safety**

10. All ordinary safety regulations of the Workmen's Compensation Board applicable to this construction project, and a number of special safety measures recommended by the Board, were fully complied with by the Contractors.

In addition to the provision of a qualified First Aid attendant, First Aid room and all First Aid equipment and other requirements of the regulations, the Bridge Company had appointed a Safety Inspector on the project, who made continuous inspections. The Company held weekly safety meetings (rather than the monthly safety meetings required by the Workmen's Compensation Board) which were regularly attended by all erection personnel.

Any complaints relating to safety measures were rectified immediately by the Contractors.

The Bridge Company had provided a safety or rescue boat, powered by two outboard motors, manned by a boatman at all times during working hours, and equipped with life belts and life rings. This boat was on duty at the time of the collapse.

All men employed on the construction and painting over water were equipped with, trained in the use of, and strictly required to wear individual buoyancy equipment.

Reports of the weekly safety meetings and the inspection reports made by the Workmen's Compensation Board Inspectors, filed as exhibits, confirm the testimony of the Director of Accident Prevention of the Workmen's Compensation Board that the Board received the finest of cooperation from the contractors and trades people generally on this project and that the Bridge Company had always been most cooperative in the prevention of accidents. (See Exhibits 225, 226, Schedule 10).

**INVESTIGATION AND ELIMINATION OF POSSIBLE CAUSES**

Following the first hearing of the evidence of eyewitnesses and workmen, and an examination of the design and erection plans and drawings, the Commission's Advisers directed their attention to the elimination of those possible causes of the collapse which in the light of events could not be substantiated by the evidence. Upon the evidence adduced, including the acceptable opinions of the experts who testified, the following possible causes can be eliminated.

**1. Design and Specifications**

General examination of the contract plans and specifications by the Advisers revealed no departure from sound engineering practice, normal for the design of a bridge structure of this magnitude. The specifications and design criteria are quite similar to, and for the most part identical with, those used for similar structures designed in the United States and the United Kingdom. There was no indication of anything weak or faulty about the permanent parts of the bridge itself which could have initiated the collapse.

**2. Unusual External Forces**

The collapse was not caused by sabotage, explosion, extraordinary winds, or earthquake. The velocity and direction of the tide were normal; no ship was in collision with any part of the structure or foundations.

**3. No. 1 Crane**

No defects in the crane or winch mechanism were found to have existed before the collapse that might have contributed to the collapse. All its cables were in good serviceable condition. No evidence was found to show that failure in the crane system caused the collapse, or that any false move was made in the operation of the crane. (See report of R. N. McLellan, pp. 14 & 15, Schedule 12).

**4. Span 4**

There was no structural weakness or instability in span 4 which could have brought down that span, initiated the collapse of span 5, or brought about the fracture or tilting of pier 14.

**5. Pier 14**

There is no evidence to show pre-collapse weakness
or failure at pier 14. It was soundly constructed in accordance with the design. Its state of rupture was the result and not the cause of the collapse.

6. Span 5

No condition of over-stress existed in any structural member of span 5 due to the erection procedure, all critical members having been duly reinforced where needed. (Report of Modjeski & Masters, Sec. II, pp. 2 & 3, Schedule 9).

7. Workmanship

Members of the erection gangs testified that the relations of the crew to each other and with their employer and the foremen were satisfactory, and indicated that the crew members had confidence in and good relations with each other, and there was no disharmony. The collapse was not due to careless or faulty workmanship on the part of the workmen engaged in the process of erecting or painting the bridge structure.

8. Contractor’s Erection Methods, Equipment & Devices (other than Bent N4)

Except for the four beams comprising the upper tier of the grillage at false bent N4, no inherent weakness was found in the erection methods, equipment and devices used by the contractor, the Bridge Company.

9. Possible Disturbances of Tie-downs in Pier 14

A theory was advanced that the steel wedges and packs securing the prestressed tie-downs in the west reaction chamber of pier 14 became displaced during the last passage of the locomotive and train over this pier shortly before the collapse, and that the resulting disturbance of the equilibrium of span 5 initiated the failure of the upper grillage beams of false bent N4, those beams being already in a critical condition. Consideration was given to this theory. No evidence was found to indicate that the wedges and packs were displaced prior to the collapse; and the observed mode of collapse is not consistent with such a prior displacement.

It was also suggested that an initiating disturbance might have been caused by the resistance offered by, or by the sudden crushing of, the temporary soft blocking inserted to restrict longitudinal movement of the tie-downs. No evidence was found that such crushing occurred prior to the collapse; and the relevant meteorological records do not indicate temperature variations wide enough to account for such crushing, or to account for longitudinal resistance being developed in the soft blocking, sufficient to cause any displacement of the wedges and packs securing the bottom ends of the tie-downs.

10. Falsework Bent N4 (except Upper Grillage Beams or Stringers)

This temporary structure is described in detail in the report contained in Schedule 9 and is shown in the erection drawings of the Contractor (Exhibit 116, DB drawing S3703/E13 and DB drawing 3703/D4). It consisted of two steel columns and their inter-connecting bracing, an upper tier and a lower tier of grillage beams and two pile groups. The beams comprising the upper tier are referred to as the “stringers.”

The type of pile provided was suitable. The piles themselves were carefully and accurately driven. The condition of the pile groups, after being subjected to high dynamic loadings during the collapse, showed that they were adequate to withstand the erection loads to be placed upon them. The groups of 24 individual piles forming the east and west foundations of the false bent were adequately placed to provide effective distribution of loads. The beams comprising the lower tier of the grillage were heavy welded I sections, specially designed and fabricated for their purpose, and were adequate to transmit the loads with a satisfactory margin for eccentric over-load.

No indication of defective or substandard material was found in the upper grillage beams. On the contrary, the manner in which they folded over each other without cracking (except in a very few instances of very severe edge loading) showed the steel to be of excellent quality. Measurements of the undamaged portions indicated reasonably accurate rolling and fabrication. (Advisers’ Joint Report, Schedule 9, p. 18, para 4.22).

11. Failure other than Stringers

No evidence was found from which it could be concluded that failure of any item other than the stringers comprising the upper tier of grillage in bent N4 initiated or contributed to the collapse of the bridge.

CAUSE OF COLLAPSE

From the evidence adduced I find the cause of the collapse was the failure of the upper tier of grillage beams in false bent N4.

Prior to the hearings, which resumed on September 30th, the Commission’s Advisers completed their studies and arrived at tentative conclusions as to the cause of the collapse. These conclusions were communicated to the Bridge Company, Swan, Wooster and Partners and Mr. Minshall. This was done upon the understanding that the Bridge Company and Swan, Wooster and Partners would communicate their views or objections to the Commission’s Advisers prior to the hearing on September 30th. It was considered that such an interchange of information and tentative conclusions would permit time for a study of all expert opinion prior to the hearing, and save considerable time and expense in arriving at final conclusions to be given in evidence.

In the result, the unanimous conclusions of the Commission’s Advisers, arrived at independently, as to the reasons for the collapse were concurred in by Swan, Wooster and Partners on 23rd September, 1958, and by representatives of the Bridge Company on 26th September, 1958, and the Commission was frankly informed by both of the above, prior to the hearing on
September 30th, that they proposed to take this position at the hearing.

At the date of submission of the Advisers' reports, the lower parts of the two columns of false bent N4 had not been salvaged. These members were recovered a few days later and examination of them disclosed nothing which caused the Commission's Advisers, the Bridge Company, or Swan, Wooster and Partners to alter their conclusions.

The conclusion as to the cause contained in the unanimous joint report of the Commission's Advisers (Schedule 9, Exhibit 317) has in general been concurred in by the Bridge Company, by its Vice-President and Manager of its Eastern Division, Mr. R. S. Eadie, and by Swan, Wooster and Partners. The calculations made by the Commission's Advisers relating to the check for web stiffness and the check for shear of the upper grillage beams of false bent N4 were also concurred in by Mr. Eadie.

Accordingly, I feel entitled to rely with confidence, and have based my finding as to the cause, upon their conclusion, namely:

"The primary cause of the accident is elastic instability of the webs of the stringer beams of N4 grillage, accentuated by the plywood packings above and below the beams. The instability was due to the omission of stiffeners and effective diaphragming in the grillage, and this in turn was basically due to an error in the calculations. Such diaphragming as was provided was inadequate."


The reports of Mr. A. B. Sanderson (Exhibit 316), of Modjeski and Masters (Schedule 9, Exhibit 319), and the unanimous joint report of Dr. Masters and Messrs. Otter, Freeman and Giese (Exhibit 317) (copies of which have for convenience been included in Volume 1 of this report as Schedule 9), as well as the various reports referred to on page 2 of the joint report, must be read as an integral part of this report.

COMMENT ON THE USE OF PLYWOOD

By way of amplification of the Advisers' conclusions in respect of the use of plywood, their opinion, as expressed by Mr. Ralph Freeman, was that the plywood which was used as "soft" packing above and below the upper tier of grillage beams of false bent N4 was a contributory cause of the failure of the grillage solely because of the absence of stiffeners and effective diaphragming in the upper tier of the grillage.

If the upper tier had been fitted with stiffeners and effective diaphragms the plywood would not have interfered with the intended performance of the grillage.

However, the arrangement of the grillage was such that the pressure on the plywood was about 1340 lbs. per square inch under the west leg of bent N4. This pressure was beyond the elastic range of the material. If plywood were to be used as soft packing the grillage should have been arranged so as to keep the pressure on the plywood well within its elastic range. This would have avoided the "creep" in the plywood referred to in para. 4.27 of the Advisers' Joint Report and would have mitigated the undesirably high lateral bending stress in the flanges of the upper grillage beams mentioned in para. 4.29 of the Joint Report.

FURTHER TERMS OF COMMISSION

Under the terms of the Commission I was further directed specifically to determine what technical or engineering advice the British Columbia Toll Highways and Bridges Authority, and any contractor or contractors in any way involved in the construction of the bridge, received in connection with its design, erection or construction, whether such advice was sound, and whether such advice was followed or was to any extent disregarded by any person or persons in the employ of the British Columbia Toll Highways and Bridges Authority, or by anyone acting on its behalf, or by any contractor or sub-contractor engaged in this undertaking, and to ascertain whether the negligence or faulty judgment of any person, persons, firm or corporation in any way contributed to or caused the said collapse.

THE BRITISH COLUMBIA TOLL HIGHWAYS
AND BRIDGES AUTHORITY

The British Columbia Toll Highways and Bridges Authority is a Crown Corporation established by the Legislature of the Province of British Columbia for the purpose of constructing and operating toll highways and bridges (see Statutes of British Columbia, 1953, Second Session, Chapter 37, and Amendments). It consists of six members under the chairmanship of the Premier of the Province. It makes decisions basically on matters of policy. It does not maintain a technical staff to carry out its purposes. The technical staff of the Department of Highways of the Province of British Columbia (hereinafter referred to as the Department) acts on behalf of the Authority in relation to all technical matters, and the Authority relies upon the engineering advice and recommendations of the engineering staff of that Department. Where bridges are concerned, the Authority receives advice from the Bridge Division of the Department as to the design of the bridge.

For this project, with the approval of the Authority, the firm of Swan, Wooster and Partners was retained as consulting engineers.

This firm enjoyed the reputation in the Province of being "a firm of first class calibre in the matter of building major bridges," and had previously rendered
highly satisfactory services on other bridge projects in the Province. The senior partner, Colonel W. G. Swan, has had fifty years' experience in bridge construction, thirty-three years as a Consulting Engineer, and was consultant for the Pattullo Bridge, the Lion's Gate Bridge (with Messrs. Monsarrat and Pratley), the Kelowna Bridge and the Kitimat Bridge.

The only written contract between the Department and the Engineers was a letter, dated 17th May, 1956, (Exhibit 183) from the office of the Bridge Engineer of the Department, signed by the Bridge Engineer, to which reference is made later in this report. After investigation, the Engineers prepared design drawings and specifications, which were examined generally by the Bridge Division of the Department and found to be satisfactory. Tenders were called and two separate contracts, dated 9th January, 1956 and 7th August, 1957, were entered into in the name of the Authority for construction of the foundations and for the superstructure respectively.

The Authority received the advice and recommendation of the Bridge Division of the Department as to the design of the bridge as prepared by the Engineers. This advice as to design was sound and was followed, and was not disregarded in any respect by the Authority. The Authority did not receive any advice in connection with erection procedure or construction.

There was no negligence or faulty judgment on the part of the Authority which caused or contributed to the collapse.

THE CONTRACTORS

I understand the word “advice” as used in the terms of the Commission “in relation to any contractor involved in construction of the bridge,” to mean advice obtained by a contractor from an external source, and not “advice” obtained from or given by the Contractor's own salaried employees in the course of their employment. I would also exclude from the definition of “advice” in this context, any instruction or information issued to a contractor by the Engineers. Under this definition the question as to what technical or engineering advice any contractor or contractors involved received in connection with the design of the permanent bridge is inapplicable, since neither of the two contractors concerned prepared or was responsible for the design.

Insofar as either contractors or any sub-contractors did receive advice as to the design of the bridge itself, such advice was sound and was followed and was not disregarded by the contractors or any sub-contractor engaged in the undertaking, or by any persons in the employ of the Authority or acting on its behalf.

No evidence was found to indicate that the contractors or sub-contractors for the substructure, disregarded or failed to follow any advice received by them, or that they received any advice which was not sound in relation to construction, or that there was any negligence or faulty judgment on their part which in any way contributed to or caused the collapse.

As regards construction of false bent N4, the Bridge Company obtained advice from Paul M. Cook, P. Eng., a Consulting Engineer, with regard to the stability and load bearing capacity of the pile groups of that false bent (Exhibit 101). Its conclusion was that the pile groups were stable and safe. This advice was sound and was followed and was not disregarded by the Bridge Company.

What “advice” the Bridge Company received from its own employees in connection with the design, erection and construction of false bent N4 is dealt with below.

The terms of the Commission use the term “negligence of any corporation.” There is some question as to whether or not, under such circumstances as existed here, negligence in the legal sense can be found against an incorporated company, as distinct from the negligence of its servants or agents.

In my view, for the purposes of this report, it is unnecessary to elaborate upon such a distinction, since there is no doubt that negligence of its servants and agents is a matter for which a corporation may, in certain circumstances, be found responsible.

Having determined the cause of the collapse, the first question is, was there negligence or faulty judgment in connection with the design, erection or construction of the upper grillage of false bent N4 which caused or contributed to the cause of the collapse?

To this question my answer is that the collapse was caused by and was the result of negligence. It remains then to determine what was the negligence and to fix responsibility for it.

I do not propose to set out the voluminous evidence in detail, but, dealing first with the position of the Bridge Company, certain material evidence must be referred to.

The Bridge Company's contract specification (Exhibit 7) included this clause:

“2.2.3. Falsework. The Contractor shall furnish, construct and subsequently remove all falsework required for the erection of the steelwork. Falsework shall be properly designed and substantially constructed and maintained for the loads which will come upon it. The Contractor shall submit to the Engineer, plans showing the falsework he proposes to use to enable the Engineer to satisfy himself that the falsework proposed to be used complies with the requirements of this Specification. Approval of the Contractor's plans shall not be considered as relieving the Contractor of any responsibility . . .”

It will be noted that this clause contains two contractual obligations of the Bridge Company to the Authority, namely, to design properly and substantially construct the falsework, and to submit plans to the Engineers.

Mr. R. S. Eadie, a highly qualified Engineer of approximately 35 years' experience in bridge design and construction, and now Vice-President and Manager of the Eastern Division of the Bridge Company, and Vice-President of the Canadian Standards Association,
was the principal witness for the Bridge Company. He testified that about the 18th or 19th of June, immediately following the collapse, the Company's Chief Engineer, with the Vice-President of Engineering, came to Vancouver and conducted a personal investigation. Their preliminary observations led them to suspect that the cause of the failure was the failure of the upper grillage beams and that at that time found that a mathematical error had been made in the brief for the grillage beams. They returned to Montreal, where their material and preliminary observations were considered by Mr. Eadie and the staff of the Bridge Company.

Mr. Eadie came to Vancouver on July 9th and over a period personally conducted an investigation which included an examination of the fabrication procedure, which he found to be excellent, and of the erection procedure followed, which in general he found satisfactory, and a complete examination of false bent N4 and of Pier 14.

On his return to Montreal, he and his staff considered and investigated all possibilities which might have had a bearing on the cause of the accident and finally came to the conclusion that the failure occurred in the upper grillage beams "due to the mathematical error, human error, made in the design of these beams." At a meeting in Vancouver on September 26th, prior to the resumed hearings on September 30th, Mr. Eadie and other representatives of the Bridge Company informed the Commission's Advisers that in general the Bridge Company concurred in their conclusions as to the cause.

I found Mr. Eadie a competent, frank and helpful witness. I felt I could accept his evidence as reliable and trustworthy, and that he as a responsible representative of the Company was endeavouring to assist the Commission in every way possible in ascertaining the cause of the collapse.

Mr. Eadie testified that:

"on a job of this magnitude we usually take it out from under the ordinary organization of the division and assign a Senior Engineer to do nothing else but look after this job, look after the job."

Again, he stated:

"After the contract was awarded, the erection being such a major job was taken out of the ordinary organization and put in under the special organization for this project."

and later he referred to the arrangement as

"set up as a special process organization for this job as we do all over the country on any job of this magnitude."

In accordance with this practice, a Senior Engineer (shown in the Company's Management Guide as "Field Engineer") was assigned to the project, together with an Assistant Engineer.

The Field Engineer was considered a competent and experienced Engineer who had served with the Bridge Company for a total of 21 years. He had 12 years' service in the Erection Department, 4 years' service in the Design and Estimating Departments, and 5 years in the Contracting Department. He was considered by the Company as probably their second most experienced Erection Engineer in the whole Company across Canada, and most competent both as a Design Engineer and Erection Engineer. Mr. Eadie stated he was noted particularly for his thoroughness and safety-mindedness.

His assistant was a young engineer who had graduated in 1954, had one year's general experience; two years with another bridge company as Assistant Resident Engineer on supervision of construction. He joined the Bridge Company early in 1957 and in the few months prior to June, 1957, had carried out various design work for the Bridge Company, and in the words of Mr. Eadie "had been found to be a very capable designer." When this project was started he was transferred to it as Assistant Field Engineer.

There was produced in evidence a design sheet (Exhibit 119) which Mr. Eadie testified was made by the Assistant Engineer and checked by the Field Engineer. This sheet contains the calculations for the design of false bent N4 and as appears thereon two errors were made, described in the Joint Report (Exhibit 317, p. 23) as follows:

"The sheet is unnumbered but bears the title "design of caps and distributing beams using 36WF160 beams between pairs of bents" and is dated 29 June, 1957. Under the heading "CHECK SHEAR," to determine the shear stress the area has been taken as the gross area of the whole beam, including flanges and webs (47.09 sq. in.) instead of the gross area of the webs (23.5 sq. in.), as is required by accepted elastic theory and by the AASHO and all other design specifications. The shear stress has therefore, been wrongly calculated to be 6 ksi instead of 12 ksi. If 6 ksi had actually been the shear stress, it might have justified the use of the adopted beams without stiffening, as is the conclusion recorded on the calculation sheet; but this would not have been the case if the correct shear stress had been calculated. Stiffeners and diaphragms would then have been provided, and the accident would not have occurred. There is a second error on the same sheet, under the heading "CHECK FOR WEB STIFFENERS," in which the flange thickness (1") has been used instead of the web thickness (0.653") the higher stresses properly calculated would have called attention to the high general stressing of these beams even though they would still have been permissible in this case."

Of these two errors, apparently one was not detected by either the Field Engineer who checked it or his Assistant who prepared it. The other, in that portion of the sheet relating to "check for web stiffness," was detected and a pencilled notation was made crossing out the figure "1" and substituting the figure "0.653".

Mr. Eadie testified that nobody knows definitely who made that note in pencil. But whoever inserted the correct figure did not make a recalculation on the basis of the corrected figure. Notwithstanding this incorrect calculation, the letters "OK" appear in the margin. Mr. Eadie testified that it was not the function of the draftsman of the plans of bent N4 to question the absence of stiffeners or diaphragms. He was not an engineer but a draftsman.

According to the evidence adduced, based upon this sheet or brief with its uncorrected calculation, the upper grillage was subsequently placed in bent N4 without stiffening.
There was produced in evidence (Exhibit 136) what is known as the Bridge Company’s “Management Guide, Pacific Division.” This shows a “Chief Engineer” whose duties and responsibilities include ensuring that all designs and details are safe and effective within the Company’s field of responsibility and are in accordance with the applicable codes, standards and specifications, but he is authorized to delegate some of his responsibility, and does so. He did not check the brief, nor could it be expected that he would do so personally.

The guide shows also an “Erection Engineer” whose responsibility of “designing and preparing drawings and specifications for erection schemes” was apparently combined with that of the “Field Engineer” on this particular project. Mr. Eadie, referring to the Field Engineer under the special organization set up for this project, stated that this Engineer “was performing the duties of Field Engineer which is a parallel function when you look at the Chart V6 to the Erection Engineer” and had no other duties except on this job.

Also shown is an “Erection Manager” responsible for ensuring that all erection work conforms to the applicable codes, standards and specifications. He too has power to delegate his responsibility, and as Mr. Eadie said, could not possibly be expected personally to check all the hundreds of brief sheets involved in the erection process.

A “Structural Engineer” is shown in the guide responsible for “supervising or carrying out the preparation of structural designs for customers or for the Company’s use.” But Mr. Eadie testified he would not in this case have responsibility to consider the design of the grillage, once the special organization for this project was set up, although he would have had a supervisory responsibility in respect to design if it had been an ordinary bridge. The “Design Engineer” shown in the guide did not exercise supervision over the design of the upper grillage for the same reasons.

Accordingly, for the reasons given by Mr. Eadie, it appears that between June 29th, 1957 and the date of the collapse, the decision to use the four WF160 stringers in the upper grillage without stiffening, as shown on the sheet, was not reviewed or checked by anyone except the Field Engineer.

Both the Field Engineer and his Assistant were unfortunately carried to their deaths in the collapse and consequently are not here to give any explanation of their actions or of Exhibit 119. One can only speculate as to why the recalculation was not made; why, when an error was found, the whole matter of the design was not referred to someone else in the organization for further checking. It may be that the Field Engineer felt it unnecessary to refer to someone else for checking such a comparatively ordinary matter (as the evidence indicated it was) as the calculations and design of an upper tier of grillage beams in a false bent. There may have been other explanations which either of those Engineers might have been able to give had they been here to testify.

In the absence of explanations from the Engineers who perished in the collapse, it would appear that the erection of the grillage without stiffeners and effective diaphragming, originated in the error in the calculations shown by the calculation sheet. But, in my view, it would be unfair to those who had to do with those calculations, and an over-simplification, to say that it was only a mathematical error which caused or contributed to the collapse.

If the project had not been taken out of the “ordinary organization” with its several provisions for checking and supervising falsework designs, and if, as one is entitled to assume, each one of those officials responsible would have discharged his responsibilities when called upon, then the error would in all probability have been detected and corrected at an early stage before bent N4 was erected. From the evidence it would appear that by putting the project under the “special organization,” the responsibility of designing the falsework was given to an able, young, but comparatively inexperienced engineer, and checked by only one other person.

By so doing, the Bridge Company’s Engineers, other than the Field Engineer and his Assistant, were apparently relieved of the responsibilities under the Management Guide as to the falsework, and the ordinary provisions for checking falsework provided in the Management Guide, were not in force. Even if the setting up of a special organization for a major project such as this, following the usual practice of this experienced bridge building company, has worked successfully in other major projects, it is evident from the results here, that on this occasion it did not provide an adequate or effective checking of the design and calculations made in connection with the design of the upper grillage of bent N4.

The second contractual obligation of the Bridge Company, namely, to submit to the Engineer plans showing the falsework he proposes to use in bent N4, was not performed. When asked why not, Mr. Eadie frankly replied that as far as he could find out it was “purely an oversight.”

The evidence of the Consulting Engineers and the Commission’s Advisers was that if detailed plans of the falsework had been submitted to them they would have expected that the errors and the inadequacy of the design would have been discovered.

Any suggestion that the Bridge Company was endeavouring to save expense by using inadequate stringers in the upper grillage because material was used which was ultimately to form a part of the completed structure, is not borne out by the evidence of the Commission’s Advisers, or by any of the evidence given at the Inquiry.

The evidence disclosed that the Bridge Company has been carrying on business throughout Canada for over seventy-five years. Its high reputation and competence
employs highly qualified engineers and has efficiently
throughout Canada. The evidence showed that the
steel fabrication and the erection generally on this
particular bridge were excellent.

Mr. Jestley, counsel for the Bridge Company, sub-
mitted that the organization procedure followed in this
particular project was not only normal but was the
safest way to supervise the erection and construction
of this bridge, and that "the appointment of two highly
qualified and experienced engineers to devote their full
time to this project, rather than have a number of
engineers doing different phases of the work, followed
the usual practice of the Bridge Company."

He further submitted that in order to be certain
everything was done in the most efficient, safe manner,
one of its most highly experienced Field Engineers was
appointed to supervise the erection, and that in view
of the fabrication methods, the erection procedure, the
competent engineers, and the precautions taken by the
Company, nothing more could or should have been
reasonably expected of it. With this submission I do
not agree. While it is, as counsel for the Bridge
Company stated, easy to be wise after the event, it
must be recognized that human errors may and do
occur and where safety of the public and of erection
crews is concerned, reasonable safeguards in the form
of adequate and effective checking must be provided.

Accordingly, upon the evidence, my answer to this
further question is that the collapse was caused by
and was the result of negligence which consisted in:

(a) failing properly to design and substantially to
construct false bent N4 for the loads which would
come upon it as required by Clause 2-2-3 of the con-
tact specification (Exhibit 7); and

(b) failing to submit to the Engineers plans showing
the falsework the Contractor proposed to use in the
erection of span 5 as required by Clause 2-2-3; and

(c) leaving the design of the upper grillage of false
bent N4 to a comparatively inexperienced engineer,
and failing to provide for adequate or effective check-
ing of the design and the calculations made in con-
nection with the design.

For this negligence, I find the Dominion Bridge
Company Limited responsible.

THE CONSULTING ENGINEERS

I have given consideration to the position of the
Consulting Engineers. Mr. Steer, their counsel, sub-
mitted that having in mind the terms of their contract
with the Authority, and the specifications, the Engineers
fully performed their duty to the Authority, and that
there was no legal or other responsibility upon the
Engineers for the unfortunate occurrence.

The letter of May 17, 1956 (Exhibit 183) from the
Bridge Engineer of the Department of Highways,
referred to above, constituted the only written contract
between the Authority and the Engineers and reads
in part as follows:

"Following the recent discussion regarding your engineering fees
for this project the Minister has approved the following:

(a) For plans, specifications, tenders and all matters short of
supervision of the work in progress, 2.7% of the cost of
construction covered by the tenders.

(b) For supervision of construction 2.46% of the amount ap-
proved for payment to the Contractor on contract progress
estimates."

And in the final paragraph states:

"Item (b) includes the provision of a full-time resident en-
inging staff on the work and all expenses in connection therewith,
but does not include the fees of inspection and testing engineers
for specialized inspection and testing of materials entering into
the work, which will be paid by the Authority."

Clause 2-2-3 of the contract specifications, quoted
above, provides that the contractor shall construct
all falsework required for the erection of the steel-
work, and that

"falsework shall be properly designed and substantially con-
structed and maintained for the loads which will come upon it.'

and that the contractor

"shall submit to the Engineer plans showing the falsework he
proposes to use to enable the Engineer to satisfy himself that the
falsework proposed to be used complies with the requirements
of this Specification."

It further provides that approval of the Contractor's
plans shall not be considered as relieving the Contractor
of any responsibility."

The contract in which Clause 2-2-3 appears was
dated August 7, 1957, almost fifteen months after the
date of the letter to the Engineers (Exhibit 183).

The specifications were drawn up by the Engineers' firm and the Engineers accordingly must be presumed
to have had full knowledge of Clause 2-2-3.

Mr. Saunders, Bridge Engineer of the Department,
a professional engineer of 29 years' standing, em-
ployed by the Department for 39 years, and who signed
the letter of May 17, 1956 (Exhibit 183), when asked
what was meant by "supervision of construction,
"testified that it meant "all the work necessary to ensure
that the contract is carried out according to the draw-
ings and contract documents," and "that the finished
work should be in conformity with the original design."

He also testified that the Department received no
advice from the Consultants on erection procedure and
that the Consultants were not asked for advice regard-
ing erection procedure; that the Department did not
get advice from or exercise any supervision over the
Contractor at all; that was left entirely to the 'Con-
sultant'; nor did it make any inspections of any sort;
the Department did not exercise any jurisdiction over
the erection of falsework or falsework design, and the
question of approval or disapproval of falsework or
falsework design did not come before the Department
at any time; that from the time the design of the Bridge
was approved by the Authority, the question of super-
vision and all responsibility for the structure was left
in the hands of the Consulting Engineers.

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Mr. Wooster testified that his firm's supervision
"would include checking to see that the final structure, the permanent structure, was as we designed it, that the bolts were properly put in, tensioned, that members fitted properly, that painting was done properly, and would not include any of the falsework. It would include approval of the procedure of erection but not any of the details or equipment. It also includes the checking of all shop drawings, and more or less the supervision of the shop fabrication through another firm who would report to us."

Colonel Swan, whose bridge experience extends over a period in excess of fifty years, and who has dealt with the Department for other bridges in the Province, when asked if on any previous project upon which he had acted as Consultant he had "actually gone into the work of a contractor such as Dominion Bridge. . . to the extent of examining in detail plans for falsework grillage, such as failed in this instance," testified "no, heretofore we have never done anything except to approve the procedure drawing and to take an interest so far as we were capable of doing so, in the general plans and the carrying out of the falsework construction."

He testified that he was aware that bents N4 and N5 had to be used, or something of a similar nature, in the erection of span 5, and that he definitely knew about it when the work was started in the field because he had "followed the construction of the basic part of N4 and N5 while being built." He said he had not received any design of N4 and N5 and when asked what was his reaction to that—that he did not receive it—he replied:

"Well, I think probably I didn't give it too much thought . . . primarily because we had never done it before and secondly that the specifications indicated that no matter what action we took in the matter the responsibility was still that of the contractor."

Colonel Swan testified that in respect of the pile driving for false bents N4 and N5 he did not consider his firm had responsibility for them, but that he had given the Bridge Company all the information his firm had on borings and their experience in the pile driving on the encompassing structures and that, while his firm watched it, they did not inspect it or did not have the usual inspection responsibility for the driving of the piles. He went on to say:

"We simply observed it, obtained the record verbally from the Dominion Bridge Company's engineering staff and in all cases those piles were driven practically to refusal."

He further stated:

"I think I must say that after the pile support had been driven I gave very little thought to the question of the distribution and the erection of the support to carry the load under N4. N5 hadn't been reached. Dominion Bridge Company has been working for me for 25 years, they have never had any failure in falsework during that period and it was—I wouldn't call it an elementary but it was pretty much a routine matter of designing the distribution system to carry the required load at N4 down to the pile foundation."

Other engineers agreed with Colonel Swan that the designing of such a grillage was a routine and not an uncommon matter or unusual problem.

The evidence does, I think, establish that the inadequacy of the design of the upper grillage would probably not have been apparent on a visual examination.

In cross-examination, Colonel Swan was asked the following questions and gave the following answers:

Q "... If detailed plans of this falsework had been submitted to your organization would you expect that the errors and the inadequacy of the design would have been discovered?
A I think they would have been discovered.
Q And the accident avoided?
A I think so."

Colonel Swan was asked, if he had seen the plans only and not the calculations, would he not have approved of the plans, and his answer was:

"I would have said the plans looked all right, but I think it would be quite impossible to adopt the position that if the plans had been submitted to us we would want to know something about the conditions under which it was operated."

He went on to say that if the plans had been submitted to him he "definitely would assume from my experience with Dominion Bridge Company that the design of it would have been adequate.

The evidence shows that its Senior Partner, Colonel Swan, having had the supervision of bridges built by the Dominion Bridge Company for more than 25 years and never before having received from that Company the design of falsework, and never having had a failure in the falsework used by that Company in erection, the firm of Swan and Wooster relied upon the Bridge Company to design and construct the falsework properly.

Mr. Eadie testified that in his experience he had found only a few cases where in very special structures the specifications required the contractor to submit detailed plans of the falsework to the consulting engineers. Mr. Saunders confirmed the testimony of Colonel Swan that for the Pattullo Bridge, built about 1926-1938, there was no requirement of checking falsework plans, but he stated that in more recent years for the larger bridges such as the Agassiz-Rosedale (1954) and Nine Mile Canyon Bridge, the contractor was required by the specifications to submit designs of the falsework and did so, and the plans were examined by the Department.

From the evidence, it is apparent that the Engineers were acting in accordance with a practice which existed in British Columbia, insofar as the Dominion Bridge Company was concerned, for a long period of years, and apparently down to 1954, when a clause similar to Clause 2-2-3 was for the first time included in the specifications of the Agassiz-Rosedale Bridge.

It is clear from the evidence before me that had the Engineers called for plans showing the falsework which the Contractor proposed to use, in sufficient detail to enable the Engineers to satisfy themselves as to compliance with the requirements of the specification, the inadequacy of the upper grillage probably would have been discovered and the collapse avoided.
The question is, was there in the circumstances here existing, an obligation on the Engineers to require the Contractor to submit plans of the falsework?

There are three circumstances here which must be taken into consideration, the wording of the letter of May 17th, Clause 2-2-3 of the specification, and the ordinary professional relationship and duty of a consulting engineer to his client.

In these circumstances, there was here, in my view, an obligation on the Engineers to satisfy themselves that the falsework proposed to be used complied with the requirements of the specifications, namely, that the falsework should be properly designed and substantially constructed for the loads which would come upon it. The Toll Authority's contract specifications had provided that the Contractor must submit plans of the falsework to the Engineers to enable them to satisfy themselves that the falsework complied with the requirements of the specification. The Engineers knew this. They must be taken to have known that the Toll Authority was relying on the Engineers to check the plans of the falsework. When the Contractor failed to submit these plans, it was the duty of the Engineers to their client, to require from the Contractor their production. This duty the Engineers failed to perform. In view of their long and successful association with the Contractor, this failure is understandable, but it cannot be overlooked. Accordingly, I must find there was an obligation on the Engineers to require the Contractor to submit plans of the falsework.

The evidence disclosed that it is the practice of those outstanding firms in the United Kingdom and the United States represented by the Commission's Advisers, Dr. Masters and Messrs. Giese, Otter and Freeman, to examine all falsework plans of any contractor, and to require submission of such plans, including the adequacy of the falsework. These witnesses made it clear that they were not testifying that this was the generally accepted practice of consulting engineers in their respective jurisdictions, but limited their evidence to the practice of their own particular firms. They indicated that by their practice they did not intend to relieve the contractor of his responsibility for the adequacy of the falsework.

RECOMMENDATIONS

Beyond saying that a Consulting Engineer must exercise all reasonable skill, care and diligence in the discharge of his duties, it is neither desirable nor practicable to lay down the precise boundaries within which such qualities should be exercised.

However, evidence adduced in this Inquiry does seem to indicate that, in bridge construction work, and especially in the construction of long span or exceptionally large bridges, some Consulting Engineers recommend (either by reference to the appropriate national standard specification or in detail) the stresses to be used in the design of temporary works; and that the erection contractor shall submit to the engineer full particulars of the erection procedures and details of design of the temporary works which the contractor proposes to adopt, to enable the engineer to examine those procedures and details. The engineer performs that examination to the extent which he judges advisable in the circumstances. The contractor should not use the proposed temporary works in erection until the engineer has signified that he has completed his examination and that he has no objection to the contractor's proposals.

Appropriate reservations could be made to ensure that the engineer does not in any sense assume or relieve the contractor of the contractual and financial responsibility for the adequacy of the contractor's proposed procedures and temporary works, whilst the contractor retains full freedom of choice of erection methods and procedure. The examination by the engineer nevertheless constitutes an additional means of discovering any latent engineering design or computation errors which may have escaped the notice of the contractor's own checking arrangements.

Insofar as the prevailing practice in British Columbia does not conform to the practice outlined above, (and I make no finding upon the evidence adduced that it does not) it would seem to me that the adoption of such a practice in the construction of large bridges in the Province would safeguard the interests of the public and of erection crews to the greatest practicable extent against the inherent and ever-present dangers of human error, and I recommend that if Your Honour sees fit, this matter might be brought to the attention of the Professional Engineers in the Province for consideration.

I further recommend that this practice be adopted in relation to the remaining construction of the Second Narrows Bridge. The examination by the Advisers to the Commission of the detailed plans of the Contractor and Consulting Engineers, of necessity, was confined to the construction up to the point of collapse. They were not asked, and had no opportunity to check into erection methods to be used for completing the central span. There is then no evidence before me as to what checks have been or are being made in respect to the remaining construction.

In the light of what has happened, I recommend to the Authority that it require the Consulting Engineers to demand of the Contractors all their proposed plans (including computations and stress analyses showing the strengthening of the structure itself in order to provide for safe cantilever erection without falsework, and any devices used for effecting the closure of the cantilever span) for the erection of the remainder of the work to enable the Engineers to examine those procedures and details. The Engineers' examination should include, by way of precaution, an examination...
of the functioning of the tie-down devices in the well of pier 14. The Contractor should not use any temporary works in erection until the Engineers have signified that they have completed their examination and have no objection to the Contractor’s proposals.

SUBMISSION OF THE VANCOUVER, NEW WESTMINSTER AND DISTRICT BUILDING TRADES COUNCIL

Mr. Herbert Macaulay filed a submission (Exhibit 338) on behalf of the Vancouver, New Westminster and District Building Trades Council which contained a recommendation relating to the employment of Consulting Engineers by the Workmen’s Compensation Board. This submission was referred to counsel for the Board. Under date of October 20, 1958, the Acting Chairman of the Workmen’s Compensation Board wrote to the Commission stating its position in respect of the recommendation. A copy of that letter is forwarded herewith as Appendix “A” for consideration of the proper authority. I do not consider it part of my function to deal with the matter beyond respectfully directing Your Honour’s attention to the submission and reply.

RESCUE

This report would, I feel, be incomplete if it failed to direct the attention of Your Honour to the rescue operations which were undertaken following the collapse. The prompt action of many persons ashore and afloat who, without regard to their own safety or the possibility of further collapse, assisted in the rescue of people from the water and the search for survivors, displayed courage of a high order. Their unselfish efforts are deserving of public appreciation and commendation. (See Exhibit 301).

CONCLUSION

My task was greatly simplified by the cooperation and assistance at all stages of the investigation by the Dominion Bridge Company Limited, Messrs. Swan, Wooster and Partners, the British Columbia Toll Highways and Bridges Authority, the contractors for the sub-structure, Peter Kiewit Sons Company of Canada Limited, Raymond International Co. Ltd., the members of the Unions employed on the project, and the eye-witnesses who came forward to testify.

The precise cause of a tragedy of this nature must be determined, not by conjecture or surmise, but by the accurate, scientific investigation of skilled and experienced experts. If their conclusions command professional acceptance, then the findings and recommendations of the Commission may be of assistance in avoiding a recurrence of such a tragedy in future.

I am very grateful to each one of the Engineering Consultants and Advisers to the Commission for their valuable services and for the highly skilled professional qualities which they placed at the disposal of the Commission, including their impartiality and fairness.

I am also grateful to Mr. John L. Farris, Q.C. and Mr. W. J. Wallace for their assistance and conspicuous ability in dealing with difficult technical matters, and the manner in which they presented to the Commission the evidence relating to the various matters considered during the Inquiry.

I also acknowledge with appreciation the faithful and unremitting services of Mr. Robert Wilson as Secretary to the Commission, and the services of other members of the staff.

DATED at Vancouver, British Columbia, this 27th day of November, A.D. 1958.

SHERWOOD LETT, Commissioner.
SEAL OF THE PROVINCE OF BRITISH COLUMBIA

COAT OF ARMS

PROVINCE OF BRITISH COLUMBIA

ELIZABETH THE SECOND, by the Grace of God, of the United Kingdom, Canada, and Her Other Realms and Territories, Queen, Head of the Commonwealth, Defender of the Faith.

In the matter of the “Public Inquiries Act”

A COMMISSION

To The Honourable Chief Justice Sherwood Lett

WHEREAS section 3 of the “Public Inquiries Act” provides that whenever the Lieutenant-Governor in Council deems it expedient to cause inquiry to be made into and concerning any matter connected with the good government of the Province or the conduct of any part of the public business thereof the Lieutenant-Governor in Council may by Commission intituled in the matter of that Act and issued under the Great Seal appoint Commissioners or a sole Commissioner to inquire into such matters:

AND WHEREAS a bridge known as The Second Narrows Bridge has been in the course of construction across Burrard Inlet under contract with the British Columbia Toll Highways and Bridges Authority:

AND WHEREAS on Tuesday the 17th day of June, 1958, through cause or causes as yet unknown a certain portion of the said bridge as constructed to this date collapsed and as a result grievous loss of life and severe damage to the bridge have occurred:

AND WHEREAS His Honour the Lieutenant-Governor by and with the advice of his Executive Council hath deemed advisable in the public interest to appoint a sole Commissioner to inquire into any and all of the circumstances surrounding, leading to or having any causal connection with the aforesaid collapse and specifically to determine what technical or engineering advice the British Columbia Toll Highways and Bridges Authority and any contractor or contractors in any way involved in the construction of the bridge received in connection with its design, erection or construction, whether such advice was sound, and whether such advice was followed or to any extent disregarded by any person or persons in the employ of the British Columbia Toll Highways and Bridges Authority or by anyone acting on its behalf or by any contractor or subcontractor engaged in this undertaking, and to ascertain whether the negligence or faulty judgment of any person, persons, firm or corporation in any way contributed to or caused the said collapse.

NOW KNOW YE THEREFORE, that reposing every trust and confidence in your loyalty, integrity and ability, We do by these presents, under and by virtue of the powers contained in the “Public Inquiries Act” and in accordance with an Order-in-Council, dated the eighteenth day of June, A.D. 1958, appoint you, the Honourable Chief Justice Sherwood Lett, a Sole Commissioner to inquire into the matters aforesaid:

AND WE direct you, the said Commissioner, to report thereon to the Lieutenant-Governor in Council with the utmost dispatch consistent with the holding of a thorough inquiry into the matters aforesaid:

IN TESTIMONY WHEREOF We have caused these Our Letters to be made Patent, and the Great Seal of the Province to be hereunto affixed.

WITNESS, His Honour FRANK MACKENZIE ROSS, Lieutenant-Governor of Our said Province of British Columbia, in Our City of Victoria, in Our Said Province, this eighteenth day of June, in the year of our Lord one thousand nine hundred and fifty-eight, and in the seventh year of Our Reign.

BY COMMAND

W. D. BLACK, Provincial Secretary
SCHEDULE 2

ORDER IN COUNCIL No. 1466

I HEREBY CERTIFY THAT the following is a true copy of a Minute of the Honourable the Executive Council of the Province of British Columbia, approved by His Honour the Lieutenant-Governor on the 18th day of June, A.D. 1958.

R. A. PENNINGTON,
Deputy Provincial Secretary

TO HIS HONOUR
The Lieutenant-Governor in Council:
The undersigned has the honour to

REPORT:

THAT section 3 of the “Public Inquiries Act” provides that whenever the Lieutenant-Governor in Council deems it expedient to cause inquiry to be made into and concerning any matter connected with the good government of the Province or the conduct of any part of the public business thereof the Lieutenant-Governor in Council may by Commission intituled in the matter of that Act and issued under the Great Seal appoint Commissioners or a sole Commissioner to inquire into such matters:

AND THAT a bridge known as The Second Narrows Bridge has been in the course of construction across Burrard Inlet under contract with the British Columbia Toll Highways and Bridges Authority:

AND THAT on Tuesday the 17th day of June, 1958, through cause or causes as yet unknown a certain portion of the said bridge as constructed to this date collapsed and as a result grievous loss of life and severe damage to the bridge have occurred:

AND THAT it is deemed advisable in the public interest to appoint a sole Commissioner to inquire into any and all of the circumstances surrounding, leading to or having any causal connection with the aforesaid collapse and specifically to determine what technical or engineering advice the British Columbia Toll Highways and Bridges Authority and any contractor or contractors in any way involved in the construction of the bridge received in connection with its design, erection or construction, whether such advice was sound, and whether such advice was followed or to any extent disregarded by any person or persons in the employ of the British Columbia Toll Highways and Bridges Authority or by anyone acting on its behalf or by any contractor or subcontractor engaged in this undertaking, and to ascertain whether the negligence or faulty judgment of any person, persons, firm or corporation in any way contributed to or caused the said collapse.

AND TO RECOMMEND THAT pursuant to the authority aforesaid the Honourable Chief Justice Sherwood Lett of the Supreme Court of British Columbia, be appointed a sole Commissioner to inquire into the matters aforesaid and to report thereon in due course to the Lieutenant-Governor in Council:

AND THAT the remuneration for witness fees and allowances to witnesses in respect of mileage and maintenance be on the same scale as provided in the Supreme Court of British Columbia:

AND THAT the Commissioner be authorized to employ counsel and such engineers or other professional advisers and such clerks and stenographers as are considered necessary for the purpose of conducting the inquiry at the usual rates for such service:

AND THAT the Commissioner be requested to report his findings to the Lieutenant-Governor in Council with the utmost dispatch consistent with the holding of a thorough inquiry into the matters aforesaid.

DATED this 18th day of June, A.D. 1958.

R. W. BONNER, Attorney-General

APPROVED this 18th day of June, A.D. 1958.

W. A. C. BENNETT, Presiding Member of the Executive Council
SCHEDULE 3

COMMISSIONER'S OATH OF OFFICE

I, SHERWOOD LETT, Chief Justice of the Supreme Court of the Province of British Columbia, do swear that I will truly and faithfully execute the powers and trusts vested in me by his Honour the Lieutenant-Governor, under and pursuant to the “Public Inquiries Act,” according to the best of my knowledge and judgment. So help me God.

SHERWOOD LETT, (Signed)

The above oath was taken and subscribed before me at the City of Vancouver, in the Province of British Columbia, this 20th day of June, A.D. 1958.

N. W. WHITTAKER, (Signed)
Judge of the Supreme Court of British Columbia

PUBLIC NOTICE PURSUANT TO SECTION 6 OF “PUBLIC INQUIRIES ACT,”
PUBLISHED IN THE BRITISH COLUMBIA GAZETTE, JUNE 26TH, 1958

“PUBLIC INQUIRIES ACT”

June 18th, 1958.

Pursuant to the provisions of the “Public Inquiries Act” chapter 162 of the “Revised Statutes of British Columbia 1948” His Honour the Lieutenant-Governor in Council has appointed the Honourable Chief Justice Sherwood Lett of the Supreme Court of British Columbia a sole Commissioner to inquire into the matters as set forth in Order in Council numbered 1466, approved by His Honour the Lieutenant-Governor in Council on the 18th day of June, 1958, in the following terms:—

“That a bridge known as The Second Narrows Bridge has been in the course of construction across Burrard Inlet under contract with the British Columbia Toll Highways and Bridges Authority:

“And that on Tuesday, the 17th day of June, 1958 through cause or causes as yet unknown a certain portion of the said bridge as constructed to this date collapsed and as a result grievous loss of life and severe damage to the bridge have occurred:

“And that it is deemed advisable in the public interest to appoint a sole Commissioner to inquire into any and all of the circumstances surrounding, leading to, or having any causal connection with the aforesaid collapse, and specifically to determine what technical or engineering advice the British Columbia Toll Highways and Bridges Authority and any contractor or contractors in any way involved in the construction of the bridge received in connection with its design, erection or construction, whether such advice was sound, and whether such advice was followed or to any extent disregarded by any person or persons in the employ of the British Columbia Toll Highways and Bridges Authority or by anyone acting on its behalf or by any contractor or subcontractor engaged in this undertaking and to ascertain whether the negligence or faulty judgment of any person, persons, firm or corporation in any way contributed to or caused the said collapse.”

The first hearing of the Commission will be held at the Court-house, 800 West Georgia Street, in the City of Vancouver, B.C., on Wednesday, the 9th day of July, 1958, at 10 o'clock in the forenoon.
PERSONNEL OF THE COMMISSION

COMMISSIONER

Sherwood Lett, Chief Justice of the Supreme Court of British Columbia.

COUNSEL

John L. Farris, Q.C., Vancouver, B.C., Senior Commission Counsel.

W. J. Wallace, B.A.SC., Vancouver, B.C., Assistant Commission Counsel.

ENGINEERING CONSULTANTS AND ADVISERS TO COMMISSION


Dr. Philip L. Pratley, D.Eng., M.Inst.C.E., M.E.I.C., Montreal, Quebec (deceased after appointment).

A. B. Sanderson, B.A.SC., M.E.I.C., P.Eng., President, A. B. Sanderson & Co. Ltd., Victoria, B.C., (Representative of Dr. Pratley and appointed additional Engineering Consultant and Adviser to Commission on Dr. Pratley's death).

INVESTIGATION OF SPECIAL FEATURES

William M. Armstrong, B.A.SC., Professor of Metallurgy, University of British Columbia. Selection and testing of steel samples from upper grillage beams 2 and 3. Examination of connecting plates 302A and single shear tests on bolts.

Major Ross P. Dunbar, Royal Canadian Ordnance Corps, Vancouver, B.C., Report re possibility of explosion.


William Pryde, b.sc., District Sales Manager, Explosives Division, Canadian Industries Ltd., Vancouver, B.C., Report re possibility of explosion.

INVESTIGATORS AND INTERROGATORS OF WORKMEN AND EYE WITNESSES

V. D. Fast (Former R.C.M.P. Officer) Victoria, B.C.

H. Starek (Former R.C.M.P. Officer) Vancouver, B.C.

SECRETARY

Robert Wilson, Vancouver, B.C.

STENOGRAPHIC STAFF

Mrs. E. Farquharson, Vancouver, B.C.

Mrs. J. L. Wharton, Vancouver, B.C.

REPORT OF HEARINGS

Official Court Reporters, Vancouver Judicial District, Court House, Vancouver, B.C.

SECURITY STAFF

The British Columbia Corps of Commissionaires, Vancouver, B.C. Furnished 24 hour per day security service at the Bridge site pending examination by Commission's Engineering Consultants.
# Schedule 5

## List of Counsel and Representatives Entering Appearances at Commission Hearings

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<tr>
<td>Vernon R. Hill, Esq.,</td>
<td>Senior Counsel for Dominion Bridge Company Limited.</td>
</tr>
<tr>
<td>(at Hearings September 30th, 1958 to October 16th, 1958)</td>
<td></td>
</tr>
<tr>
<td>H. Lyle Jestley, Esq.,</td>
<td>Assistant Counsel for Dominion Bridge Company Limited.</td>
</tr>
<tr>
<td>W. S. Henson, Esq.,</td>
<td>Counsel for Peter Kiewit Sons' Company of Canada Limited, and Raymond International Co. Ltd., (withdrew on 22nd July, 1958)</td>
</tr>
<tr>
<td>J. A. Clark, Esq., q.c.</td>
<td>Engineering Consultant for Local Union No. 97 of the International Association of Bridge, Structural and Ornamental Iron Workers.</td>
</tr>
<tr>
<td>J. S. Maguire, Esq.,</td>
<td>Counsel for Workmen's Compensation Board.</td>
</tr>
<tr>
<td>R. C. Bray, Esq.,</td>
<td>Counsel for H. H. Minshall. (both withdrew October 6th, 1958)</td>
</tr>
<tr>
<td>Ralph Sullivan, Esq.,</td>
<td>Elected Representative for Vancouver, New Westminster and District Building Trades Council.</td>
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<tr>
<td>H. H. Minshall, P.Eng., M.Eng.,</td>
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<tr>
<td>J. E. Boughton, Esq.,</td>
<td>Elected Representative for Vancouver, New Westminster and District Building Trades Council and also Representative for Local Union No. 138 Brotherhood of Painters, Decorators and Paperhangers of America.</td>
</tr>
<tr>
<td>Norman Eddison, Esq.,</td>
<td></td>
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<tr>
<td>Herbert Macaulay, Esq.,</td>
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<tr>
<td>M. M. McFarlane, Esq., q.c.</td>
<td>Counsel for British Columbia Toll Highways and Bridges Authority.</td>
</tr>
<tr>
<td>(appearance entered at Hearing September 30th, 1958)</td>
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<tr>
<td>E. J. C. Stewart, Esq.,</td>
<td>Assistant Counsel for British Columbia Toll Highways and Bridges Authority.</td>
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<tr>
<td>No.</td>
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<tr>
<td>1</td>
<td>Adrian B. Sanderson</td>
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<td>2</td>
<td>Erling Anderson</td>
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<td>James Walter Welsh</td>
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<td>James Wesley English</td>
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<td>Denis Gladstone</td>
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<td>Robert Norman McLellan</td>
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<td>46</td>
<td>Louis Osopov</td>
</tr>
</tbody>
</table>
47. Alan Alexander Kay................................. Civil Engineer, (G. S. Eldridge & Co. Ltd.,) Vancouver, B.C.
48. William M. Armstrong......................... Professor of Metallurgy at the University of British Columbia, Vancouver, B.C.
49. Alexander Hrennikoff.......................... Professor of Civil Engineering at the University of British Columbia, Vancouver, B.C.

Adrian B. Sanderson, recalled

50. Ralph Freeman................................. Professional Engineer—Commission Engineering Consultant and Adviser
52. John R. Giese................................. Professional Engineer—Commission Engineering Consultant and Adviser
53. Dr. Frank M. Masters........................ Professional Engineer—Commission Engineering Consultant and Adviser
54. Harry H. Minshall............................ Professional Engineer (Minshall & Smith Engineering Ltd.) Vancouver, B.C.
55. Robert Scott Eadie............................ Vice-President and Manager of the Eastern Division, Dominion Bridge Co. Ltd., Montreal, Quebec
56. Col. William G. Swan......................... Consulting Engineer, Senior Partner, Swan, Wooster & Partners, Vancouver, B.C.
57. Hiram F. Wooster............................. Professional Engineer, Partner, Swan, Wooster & Partners, Vancouver, B.C.
58. Frederick E. Dembiske....................... Professional Engineer and Special Projects Engineer for the Department of Highways, Province of British Columbia, Victoria, B.C.

Dr. Frank M. Masters, recalled

59. John S. Prescott.............................. Manager of the Erection Department, Pacific Division of Dominion Bridge Co. Ltd.

John R. Giese, recalled
Ralph Freeman, recalled
Joseph R. H. Otter, recalled

60. Dr. John V. Fisher............................ Secretary, British Columbia Toll Highways and Bridges Authority, Victoria, B.C.
61. Cedric K. Saunders.............................. Professional Engineer and Bridge Engineer for the Department of Highways of British Columbia, Victoria, B.C.
62. John W. Williamson........................... Structural Iron Worker
63. Herbert J. Barratt............................. Consulting Engineer, Phillips Barratt & Partners, Vancouver, B.C.

A. B. Sanderson, recalled
Cedric K. Saunders, recalled

64. William C. McKenzie.......................... Professional Engineer, (Choukalos Woodburn Hooley & McKenzie Ltd.,) Vancouver, B.C.
65. Charles Edward Andrew....................... Civil Engineer and Chief Consultant Engineer for Washington State Toll Bridge Authority, Seattle, Washington, U.S.A.

Harry H. Minshall, recalled
John S. Prescott, recalled
Professor William M. Armstrong, recalled

66. Major Ross P. Dunbar.......................... Royal Canadian Ordnance Corps, Vancouver, B.C.
67. William Pryde................................. District Sales Manager, Explosives Division, Canadian Industries Ltd., Vancouver, B.C.

Ralph Freeman, recalled
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PHOTOGRAPHS AND DRAWINGS RELATING TO PART I AND PART II OF THE REPORT

(a) Aerial photograph taken June 15th, 1958, two days before the bridge collapsed, camera pointing easterly. Exhibit 296—19.

(b) Special picture prepared by Dominion Bridge Co. Ltd., showing spans 4 and 5 with inserts added to show where steel was advanced at time of collapse. Exhibit 296—19B.

(c) Aerial photograph taken on July 1st, 1958, of spans 4 and 5 after bridge collapsed, camera facing easterly. Exhibit 296—23.


(e) Drawing No. 084-12, showing grillage assembly of Bent N4 prepared by A. B. Sanderson & Co. Ltd., dated September 29th, 1958. Exhibit 308.

SCHEDULE 8

(a) Aerial photograph taken June 15, 1958, two days before bridge collapsed, camera pointing easterly. Exhibit 296 - 19.
SCHEDULE 8

(b) Special picture prepared by Dominion Bridge Co., Ltd., showing spans 4 and 5 with inserts added to show where steel was advanced at time of collapse. Exhibit 296 - 19B.
SCHEDULE 8

(c) Aerial photograph taken on July 1, 1958, of spans 4 and 5, camera facing easterly.
Exhibit 296 - 23,
(e) Drawing No. 084-12 showing grillage assembly of Bent N4 prepared by A. B. Sanderson & Co., Ltd., dated September 29, 1958. Copy of Exhibit 308.
(Reduced scale)
Schedule 8
(d) Drawing No. 1, General Arrangement prepared by A. B. Sanderson & Co., Ltd., dated July 14, 1958. Copy of Exhibit 300. (Reduced scale)
DOMINION BRIDGE CO., LIMITED
VANCOUVER, B.C.

STRUCTURE: 2ND NORTHERN BRIDGE
LOCATION: FW Anchor Span

PROPRIETOR: DOMINION BRIDGE CO., LTD.
ARCHITECT: VANCOUVER, B.C.

DATE: JUN. 29, 1957

DESIGN OF CAPS AND DISTRIBUTING BEAMS USING 36WF160 BEAMS BETWEEN
Points of Bents

Distributing Beam

Try reducing length of cap and using 4 stringers

Assuming even distribution, \( R = \frac{2700}{4} \) Kips

\[ M_0 = \frac{2700}{4} \left[ 5 + 1.67 \right] = \frac{2700 \times 1}{2} \]

\[ = 4500 - 1350 = 3150 \text{ Kip-ft.} \]

\[ Z \text{ Required} = \frac{3150 \times 12}{20} = 189 \times 10^3 \text{ in}^3 \]

Using \( 4 \)

\( Z = 1890 : 472.5 \text{ in}^3 \)

36WF160 has \( Z = 541.0 \text{ in}^3 \) - O.K.

Check for web stiffness

Top

\[ \text{Area} = 4 \times 11 \times 4 \times 0.633 \]

\[ f = \frac{2700}{192} = 14 \text{ Kips/lin. ft.} - O.K. \]

Bottom

\[ \text{Area} = 16 \times 14 \times 1 = 224 \text{ in}^2 \]

\[ f = \frac{2700}{224} = 12 \text{ Kips/lin. ft.} - O.K. \]

Check shear

\[ S = \frac{2700 \times 1.33}{4} = 811 \text{ Kips} \]

\[ \text{Area of 36WF160} = 470.9 \text{ in}^2 \]

\[ f_s = 6 \text{ Kips/in.} \]

Use 4 WF160 without stiffening.
SCHEDULE 9

REPORTS SUBMITTED IN EVIDENCE BY COMMISSION'S ENGINEERING CONSULTANTS AND ADVISERS

(a) A. B. Sanderson's Report dated September 1958. Exhibit 316.
(c) Modjeski and Masters Report dated September 1958. Exhibit 319.
SCHEDULE 9

(a) A. B. Sanderson's Report dated September, 1958. Copy of Exhibit 316.

REPORT ON

INVESTIGATION

SECOND NARROWS BRIDGE INQUIRY

A. B. Sanderson, P.ENG.
SCHEDULE 9

(a) A. B. Sanderson's Report dated September, 1958. Copy of Exhibit 316.

SECOND NARROWS BRIDGE INQUIRY INVESTIGATION

A. B. Sanderson, P.ENG.
September 22, 1958.

J.L. Farris Esq., Q.C.,
Commission Counsel,
Royal Commission,
Second Narrows Bridge Inquiry,
510 W. Hastings St.,
Vancouver, B.C.

Dear Sir:

Submitted herewith is my report on the investigation carried out to determine the technical facts relative to the collapse of the Second Narrows Bridge.

The report includes an outline of conditions at the time of the collapse; a description of the collapsed structure; the results of calculations made to determine loads, reactions and stresses at the time of collapse; an outline of investigations and tests by special consultants; a description of events accompanying the collapse as deduced from the observed configuration and damage; and an opinion as to the immediate cause of the collapse.

It is my opinion, based upon the data now available, that the collapse was due to failure of the stringers in the grillage of bent N4. This failure was caused by critical stress conditions resulting in elastic instability of the web of these stringers.

Respectfully submitted,

A. B. Sanderson, P. Eng.

ABS/jp
REPORT ON
INVESTIGATION
SECOND NARROWS BRIDGE INQUIRY

September 1958

A. B. Sanderson, P.ENG.
A. B. Sanderson and Company Ltd.
1200 W. Pender Street
Vancouver, B.C.
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SECOND NARROWS BRIDGE INQUIRY

Abbreviations:  
S/W  Swan Wooster and Partners  
D.B. Co.  Dominion Bridge Company Ltd.  
ASTM  American Society for Testing Materials  
AASHO  American Association of State Highway Officials

LIST OF SKETCHES

I  Service Train  
II  Elevations on Pedestal—Piers 13 and 14  
III  Position of Legs, Bent N4  
IV  Position of Tie Down Beams West Chamber  
V  Plan of Approx. Location of Service Train & Load After Fall

LIST OF DRAWINGS

084-3  Position of Fallen Spans  
084-4  Bearings—Pier 14  
084-7  Location and Weights of Erected Steel at time of Collapse  
084-8  Location and Weights of all Loads at time of Collapse  
084-9  Erection Stresses at time of Collapse  
084-10  Span 4—Geometry of Fall  
084-11  Span 5—Geometry of Fall
1. INTRODUCTION

The steel superstructure of the Second Narrows Bridge collapsed during erection at about 3.40 pm June 17, 1958. The North Anchor Arm which was being erected by cantilevering over a false bent collapsed and almost simultaneously the adjacent approach span, Span No. 4, also collapsed.

Fifty-nine men were working on the spans at the time of collapse. Eighteen of these were killed. Some days later a diver searching the wreckage for bodies of some of the missing was drowned, bringing the total number of dead to 19.

Consulting Engineers for the bridge were Swan, Wooster and Partners acting for the owner, the B.C. Toll Highways and Bridges Authority. The superstructure was being fabricated and erected by Dominion Bridge Company Ltd. The substructure had been built by Peter Kiewit Sons Co. of Canada Ltd. and Raymond International Co. Ltd. as a joint venture. The joint venture was managed by Raymond International.

The bridge will have a main cantilever section of 1100 foot central span and 465 foot anchor spans. There are no approach spans at the South end. At the North end there are 4—280 foot deck truss spans and about 1000 feet of approach viaduct. The approach viaduct comprises 9—120 foot prestressed concrete stringer spans. The bridge carries 6 lanes of traffic and 2 sidewalks. The general layout is shown on S/W Dwg. 1A, Job 133.

Photo No. 10 is a progress picture taken March 11th, 1958. At this time Span 4 was being erected by cantilevering off Span 3 and steel was approaching Pier 14. The picture is taken looking North-West.

Photo No. 19 shows the situation on June 15th, two days before the collapse. At the time of collapse the traveller had moved ahead one panel and placing of an additional panel of steel was almost complete.

Photos No. 16, 20 and 21 were taken immediately after the collapse. They show the collapsed spans 4 and 5: the timber construction trestle alongside the bridge on the West side extending out to Pier 15; the old Second Narrows Bridge paralleling the new bridge on the East side; and the general area in the vicinity of the bridge.

Spans 1 and 2 were not involved in the collapse. Span 3 was slightly damaged at its South end but was not otherwise involved. Spans 4 and 5 collapsed and were essentially destroyed.

Piers 10, 11 and 12 were not affected. Piers 13 and 14 were extensively damaged.

2. SUBSTRUCTURE

(a) Water Conditions

The tide in Burrard Inlet has a normal range of about 10.5 feet and an extreme range given as 14 to 16 feet.

Strong tidal currents exist in the neighbourhood of Second Narrows. Maximum velocities in the Narrows are given in the Tide Tables as $6\frac{1}{2}$ knots for the flood and $5\frac{1}{2}$ knots for the ebb.

For June 17th the tides and currents are given in Table 1.

The maximum currents during flood occur 2 cables East of the Bridge. In the vicinity of Pier 14 and Bent N4 the current has been observed to run strongly, but the velocities here would probably be less than the maximum velocities in the Narrows.

(b) Foundation Conditions

The bridge site is underlain by glacial deposits and by outwash materials from the adjacent mountains. The site was originally investigated by Rocanen Engineering Co. Ltd. whose report indicates that the site is underlain by gravels and sands. Some of the granular materials contain silt but not in large amounts. Also some peat and organic material was found at depths of about 100 feet. Some of the sands are fine and not consolidated.

Additional drilling by Boyle Bros. confirmed the results of Rocanen Engineering's holes. At Pier 14 a hole was put down to 143 feet below water surface. Materials encountered were similar to those reported by Rocanen Engineering Co. Ltd.

Test Piles, both timber and steel H pile, were driven. The results of these tests are given on S/W Dwg. U-4887. Logs of drill holes are also given on the same drawing.

(c) Pier 13

The design of Pier 13 is shown on S/W Dwg. U-4909. The Pier is founded upon 470 timber piles driven to practical refusal within a sheet pile cofferdam, 48' x 105'. The cofferdam was excavated to Elev. -27, the piles then driven, and the piling cut off 3 feet above the bottom of the excavation. At Pier 13 the original bottom was at about Elev. -18.

The cofferdam from the bottom of the excavation at Elev. -27 to Elev. -11 was filled with a seal course of tremie concrete. Above Elev. -11 to Elev. +14 the pedestal 16' x 74' was poured in the dry.

The pier shaft consists of two columns 10' x 10' x 79', high, extending to Elev. 93. A cross beam 6' x 12' connects the tops of the columns.

The columns are reinforced with No. 11 bars at about 6' centres in each face. Horizontal bars are No. 6 at 12' centres. Column bars extend down through the pedestal to the top of the seal course. Reinforcing bar splices are called for as not less than 2'-6" lap. Cover is 2'.
(d) **Pier 14**

The design of Pier 14 is shown on S/W Dwg. U-4911, U-4912 and U-4913.

The Pier is founded on 808 timber piles driven on a 3' x 3' spacing. The piles are driven within a sheet pile cofferdam excavated to Elev. -29. This is approximately 5-6 feet below the mud line. The pile cut-off is 3 feet above the bottom of the excavation, in the seal course. The seal course is 18 feet deep and of tremie concrete. It is approximately 62' x 141' in plan.

Above the seal course, the top of which is Elev. -11, the Pier was built in the dry. The pier base, or pedestal, is 24' x 106' and extends to Elev. 14, 6 feet above ordinary high water.

The pier shafts consist of two columns 18' x 18' x 91' high. The tops of the shafts form the bridge seats at Elev. 105'. A cross beam 10' wide x 12' deep connects the tops of the columns.

The columns contain a well and chamber for the cantilever anchor arm tie down arrangement.

The columns are reinforced with No. 11 bars at about 6' centres vertical and No. 6 bars at 12' centres horizontal. Cover is 2 inches. Vertical bars are lapped 2'-6" at splices. All column steel extends down through the pedestal to the top of the seal course.

The sheet pile cofferdam was cut-off at the top of the seal course and left in place.

Table II gives the results of 28 day compression tests in the concrete of the pedestal and columns.

3. **SUPERSTRUCTURE**

Span No. 4 is a 280 foot deck truss with Warren web system. It is detailed on D.B.Co. Dwg. S3701/1 to 40 (inclusive). The span is fixed on Pier 13 and expands on a roller nest at Pier 14.

The fixed shoe on Pier 13 had 2" x 3" slotted holes for 1¾" diameter anchor bolts. The roller nest on Pier 14 had stainless steel rollers.

Span 5 is the anchor arm for the main cantilever section of the bridge. It is 465 foot span. Steelwork is detailed on D.B.Co. Dwg. S3702/101-245 (inclusive). In the completed bridge the reaction of Span 4 at Pier 14 reverses under live load. A combination roller bearing and tie down is provided for this condition. The general arrangement is shown on S/W Dwg. 36A, Job 133.


The bearing and expansion shoe is detailed on D.B.Co. Dwg. S3702/114.

The principal steelwork was specified as Low Alloy Steel ASTM 242, or equal. Steel to B.S.968 was used to conform to this requirement.

4. **ERECTION SCHEME**

(a) **General**

From Pier 13 the Approach Span No. 4 was erected by cantilevering off Span No. 3 with the aid of an erection harness over Pier 13. It was cantilevered from Pier 13 to Pier 14 in one step without false bents. Photo No. 10 shows erection of the span in progress. The erection procedure is detailed on D.B.Co. Dwg. S3703/P1.

The first four panels of Span 5 were cantilevered off Span 4 to land on a false bent, numbered N4, at Panel Point 4. From N4 the span was cantilevered a further four panels to False Bent N5, using the permanent tie down arrangements of the cantilever anchor arm in Pier 14 as a tie down. From N5 the span was to be cantilevered an additional two panels to land on Pier 15.

Photo No. 19 shows the erection of Span 5 as it reached Panel Point 7. The Bent N4 is in place and the span will be landed on Bent N5 when steel is placed to Panel Point 8.

The erection procedure for Span 5 is detailed on D. B. Co. Dwg. S3703/P2.

A general layout of the superstructure showing the principal dimensions is given on D.B.Co. Dwg. S3701/E2 and S3702/E7. S/W Dwg. 2B, Job 133, shows the general arrangement of the bridge.

(b) **False Bents N4 and N5**

Bents N4 and N5 are founded upon steel pipe piles, concrete filled. The piles are capped with built up welded I-beams which support 36" WF cross-stringers. The cross-stringers carry the base plates of the legs. The legs themselves are sections of the truss verticals with their associated sway bracing.

The design of the bents, up to the underside of the base plates, is shown on D.B.Co. Dwg. S3703/E13 and S3703/E27A and ME28. The legs of bent N4 are detailed on D.B.Co. Dwg. S3702/141 and 142 and the sway bracing on Dwg. S3702/212, 213 and 214. The lower bearing plate at the top of the legs is shop welded to the leg, as detailed on Dwg. S3702/141, and the upper bearing plate, 309H, which is connected to the underside of the truss chord at Panel Point 4, is detailed on Dwg. S3703/E309. The base plate for the foot of the legs, 309G, is detailed on Dwg. S3703/E309. The bearing plates at the top of the leg form a rocker bearing. The upper bearing plate is machined to 2'-6" radius and this bears on the lower bearing plate which is machined to 2'-8" radius.

The piles for Bent N4 were driven by Greenlees Pile Driving Co. Ltd. on a sub-contract from Dominion Bridge Company Ltd. (3) The piles were 10¾" O.D. x .279 wall steel pipe, concrete filled.

All piles were driven with a Vulcan No. 1 hammer, using steam from a 40 H.P. boiler. This information was obtained verbally from Greenlees Pile Driving Co. Ltd. Pile driving records were kept by Dominion Bridge Company Limited. (4) A summary of these

(3) Purchase Order D.B. Co. to Greenlees Pile Driving Co. Exhibit 112

(4) Pile Driving Records.

Dominion Bridge Company Ltd. Exhibit 114
The piles were driven to 120-150 feet penetration and to a resistance of about 130 blows per foot.

The design load on the piles was 56.3 tons per pile. They were driven to a minimum resistance of 166 tons calculated by the Hiley formula. For this calculation energy losses were determined by measurement of the "quake" during driving. The required resistance corresponded to about 100 blows per foot.\(^5\)

In driving Bent N5 certain piles apparently punched through the hard layer on which the piles of Bent N4 had brought up. After punching through this layer driving resistance was low and the piles drove a considerable depth before fetching up. This raised doubts about the adequacy of Bent N4. An attempt was made to re-drive some of the piles in N4 to penetrate the hard layer. No further penetration could be obtained. The adequacy of the pile group was investigated for Dominion Bridge Company Ltd. by Paul M. Cook, P.ENG., Consulting Soils Engineer.\(^6\) Two piles were test loaded to 120 tons. Results of this load test are given in the report. His report states:

"The pile groups are safe for the loads intended both with respect to tipping as a mass and also with respect to the individual pile loads."\(^7\)

Each group of piles was braced with 5 x 3\( \frac{1}{2} \times \frac{3}{8} \) and \( \frac{3}{8} \) angles above low water and with 6" channel at 8.2 lb. below low water. Above low water the angle bracing was welded to the piles. It was placed longitudinally on the two outer rows of piles of each group, that is rows A and B, and transversely on lines 1, 3, 4 and 6. Below low water the channel bracing was placed transversely on lines 1, 3, 4, and 6. It was clamped to the piles. The two groups of piles were braced transversely by diagonal wire cables running from the top of the piles of one group to the piles of the opposite group at the mud line. There were 4 such diagonal cables. Longitudinally there were two wire cables for each group of piles running from Pier 14 to Bent N4, Bent N4 to Bent N5; and Bent N5 to Pier 15.\(^5\)\(^6\)

Between the grillage beams and the stringers, and between the stringers and the column base plate, plywood equalizer pads were used. These were strips of \( \frac{3}{4} \)" plywood on top of the flanges of the grillage beams, and a sheet of \( \frac{3}{4} \)" plywood between the stringers and base plate.

Design calculations for Bent N4 are difficult to follow in Dominion Bridge Company Ltd. design briefs. The bent legs appeared to have been designed in the Engineering Department\(^7\) and the grillage in the Erection Department\(^8\).

Construction of the bent and placing the grillage was completed April 1st, 1958\(^9\).

(c) Erection Procedure, Span 5

After cantilevering Span 5 off Span 4 to Panel Point 4, the span was landed on Bent N4.

The erection procedure for landing at Bent N4 required the tie down bars to be stressed to 250 tons per truss before landing. This was done by jacking down on the prestressing beams 115C from the roof of the chamber in Pier 14. The applied jacking load was determined by gauge pressure. So far as is known the jacks had not been calibrated for the job. The pre-stress of 250 tons corresponds to 6,700 psi on the gross section of the tie down bars which are B.S. 968 steel.

The tops of the tie down bars were blocked against the walls of the concrete well while jacking to land on
Bent N4. The blocking was timber and was removed after the landing was made.

To land on N4 the North end of Span 4 was raised by Jacks. After landing, the North end of Span 4 was further jacked to release the top chord tie plates between Spans 4 and 5. After these tie plates were disconnected Span 4 was jacked horizontally to its correct position. On Pier 13 the fixed shoe of Span 4 had 2" x 3" slotted anchor bolt holes in the bed plate to allow this horizontal adjustment. Also this bed plate had steel keys on its underside and chases and had been left in the concrete pad under the bed plate to be filled with grout after the span was positioned. Since the collapse it has been found that the 1 ¾" anchor bolts, which were set in pipe sleeves, were grouted in but the chases for the keys were not grouted. It is not known if the nuts on the anchor bolts were tightened down. It may be noted that the anchor bolt holes in the bed plate were slotted so that movement of this fixed end was possible.

After landing on Bent N4 the top chord tie plates between Spans 4 and 5 were re-bolted to provide a longitudinal tie for the anchor span. These tie plates are Mark 302A and 302K and are detailed on D.B.Co. Dwg. S3703/302. They connect from a 12½" diameter pin in the end of the top chord of Span 5 to an extension of the web plates of the top chord of Span 4. During cantilevering of Span 5 to Bent N4 the connection was made with two tie plates, 1-302A and 1-302K, connected to each web with 56—1" H.T. bolts in double shear. After landing on N4 the connection was re-made using tie plates 302A only and connecting with 10—1" H.T. bolts in single shear for each plate.

After landing and releasing Span 5 from Span 4 the holes in the tie plate and the webs of Span 4 top chord no longer matched since the tie plate holes had been laid out to “cock up” Span 5 while erecting to Bent N4. To re-connect tie plates 302A with 10 bolts new holes were burned.

It is possible that new holes were first burned after the landing on N4 but before Span 4 was finally positioned. A second set of holes were burned after Span 4 was positioned. This information came from Stroud, one of the connectors. It was not certain if he knew this for a fact or had assumed it to account for the rather mixed set of holes now observed in plates 302A.

There seems to be no doubt that the plates 302A were in fact connected. This was questioned by some of the steel workers but their opinion is no doubt due to the fact that plates 302A only were reconnected and plates 302K could be seen hanging free on the pin, and further that only 10 of the 56 holes in 302A were filled with bolts.

Erection beyond Bent N4 was by cantilevering using the permanent tie down on Pier 14. The erection procedure drawing calls for the roller nest at LO, Span 5, to be free at this stage. The chief Engineer of Dominion Bridge Company, R. A. McLachlan, has stated that on June 11th or 12th he ordered “soft” blocking placed between the tie down bars and the bearing blocks under the expansion shoe. This was done and it has been possible to find and to re-construct this blocking.

So far as can be ascertained the placing of the “soft” blocking around the tie down bars was the only departure from the procedure called for in the erection procedure drawings.

(d) Travellers

Three traveller cranes were used on the work. At the North end of the bridge a yard derrick, known as Traveller No. 3 was used to raise steel to the bridge deck from the delivery point on the ground beneath Span No. 1. This was a 120 ton stiff leg derrick.

At the front end the traveller used for placing steel was a 25 ton crane, known as Traveller No. 2, was located on Span 4 at the time of collapse. This traveller had been used to erect and dismantle the erection harness while erecting the approach spans, and was to be used to dismantle the legs of the false bents N4 and N5 in Span 5. It was not in use at the stage during which the collapse occurred.

The traveller No. 1 is detailed on D.B. Dwg. Y16/D1, D2, E1 to E4 (incl.), 1 to 30 (incl.).

(e) Steel Delivery

From the yard derrick, Traveller No. 3, steel was delivered to Traveller No. 1 at the front end over a service track located over the two most westerly roadway stringers.

Steel was carried on two cars, or bogies, pushed by a diesel locomotive. Sketch 1 gives the axle spacing and loads of the locomotive and cars. The locomotive was equipped with air operated train brakes capable of braking both locomotive and cars.

The service track was standard gauge, i.e. 4'-8½" with 60 lb. rail. The ties were planked to form a walkway about 12 feet wide.

5. SITUATION PRIOR TO COLLAPSE

(a) Progress of Erection

At the time of collapse erection of Span 5 was approaching False Bent N5 which was to be located at Panel Point 8.

Erection of East truss was complete to Panel Point 8. Erection of the West truss was complete except for placing the lower chord L7-L8. The No. 1 Traveller was at Panel Point 7, that is, it occupied Panel 6-7.

The top and bottom lateral systems had been filled in to Panel Point 7, and the sway braces were in at Panel Point 7. The floor beam at Panel Point 8 was not placed and no stringers were placed in Panel 7-8.
Erection of Span 5 had started March 25th and the span landed on Bent N4 on April 15th. Erection from N4 to N5 was temporarily discontinued on April 29th. At this time Panel 5-6 was partially erected with members U5-U6, and L5-U6 in place. It is understood the erection was closed down as erection had got ahead of fabrication and the shop was to be given a chance to catch up.

Erection was started up again on June 9, 8 days before the collapse.

(b) Situation at Time of Collapse

Immediately prior to the collapse the 51.6 ton lower chord L7-L8 was on the bogies beside the No. 1 Traveller ready for hoisting and placing. The slings were connected ready for lifting and the boom was either centred and falls snugged up just ready to lift, or possibly the boom was being topped down to centre over the load. In any case it appears to be agreed that everything was ready, or almost ready, to lift the chord. The tag line was in place but some staging was probably still being placed in position on the chord.

Below at Panel Point 7 the connectors had crossed over from the East bottom chord, which had been placed a short time previously. They were re-rigging their tugger lines to be ready to pull in the West chord. They had either completed re-rigging the tugger lines, or almost so.

Bolting-up crews were working from Panel Point 4 to Panel Point 7. Connections were being made with H.T. bolts and bolting up was keeping up within a panel or two of the front end. No erection bolts were used. Connections were made with H.T. bolts and pins and the bolting crew followed right behind the connectors.

A crew of painters was working on Span 4.

The locomotive with Chord L7-L8 had arrived at the front end 20-30 minutes before the collapse. The train had been stationary, presumably with train brakes set, since its arrival until the collapse.

The weather was clear and hot. Wind was 10-15 mph from the West. Temperature was in the eighties. The tide was flooding and running about 4½ knots. No earthquake shock is recorded prior to the collapse(10).

Nothing unusual is recorded. The crane operator had reported to the Project Engineer a short time before the collapse that his boom was difficult to swing to the East. He had noticed this when placing the East lower chord and considered that his rig had not been properly levelled. He did not consider it serious or attach any unusual significance to it.

No steel was on the span other than the chord section L7-L8. No unusual loads were on the span.

6. OBSERVATIONS SINCE COLLAPSE

(a) Movements of Bases of Piers 13 and 14

The chainage of marked points on the bases of Piers 12, 13, 14 and 15 were established after the collapse by chaining between these four piers. The results are given in Table III.

The original accuracy of the chainage set by S/W is not known but it may be assumed that the accuracy was good. The chainage measured after the collapse was made using standard tension and with temperature corrections to 68°F. The tape was calibrated against a standardized tape which is kept to calibrate working tapes.

From Table III it is seen that the distance from Pier 12 to Pier 15 checks very closely with the S/W chainage. Pier 13 has moved North .036 feet or about 36 inches and Pier 14 has moved South .048 feet or about 12 inches. The direction of these movements corresponds to the movements of the pier shafts.

No evidence was found of tilting of the base of either Pier 13 or Pier 14. No points of known elevation could be checked on the top of the pedestal of either pier but a check of elevations of various points around the perimeter of the pedestals gave no indication of any consistent differences of height for the two sides of the pedestal top. Sketch II shows elevations taken on the pedestal of Pier 13 and Pier 14.

(b) Shaft of Pier 13

The columns of Pier 13 are bent to the North. At the
top the West column is 3½" North and the East column 4½" North of their original position.

There are hair line cracks across the South face of each column spaced about 1'-0" apart from the top of the pedestal to a height of about 10 feet up the column. On the East and West column faces the cracks extend to the North about half-way across the face.

On top of Pier 13 the shoes of Span 4 are over the South edge of the pier top wedged against the South face. This is seen in Photo No. 83. During the first few days after the collapse the contraction of the steel of Span 4 each night allowed the shoes to slip slightly down Pier 13. The expansion of the steel the following day then pushed the pier slightly further to the North. To prevent collapse of the pier due to this wedging action posts were placed under the shoes and a "kicker" was placed from the top of the pier to the first panel point of Span 3. The posts and kicker show in Photo No. 83. They may also be seen in Photo 90.

On the pier top the anchor bolts of Span 4 bearing, which is a fixed bearing, are in the same condition for both East and West trusses. In both cases

1. The NE and NW bolts sheared off to the North.
2. The SW bolt is not sheared, but is bent over to the North.
3. The SE bolt is not sheared, but is bent over to the South. The concrete South of the bolt is torn and the bolt is driven into the concrete.
4. The two unsheared SW anchor bolts show a notch on their N sides at about the underside of the base plate.

The anchor bolts are 1½" diameter and were set in pipe sleeves. The bolts had been grouted into the sleeves.

On the underside of the base plate of the fixed shoe are steel keys. Chases had been left in the pad forming the bridge seat to receive these keys. The chases had not been grouted.

It is not known if the nuts on the anchor bolts were tightened down.

Photo No. 112 shows the bearing of the West truss. The SW anchor bolt may be seen bent over to the North and the remains of the pipe sleeve of the SE anchor bolt can be seen on the South face of the pier.

Photo No. 116 shows an almost exactly similar condition at the East truss bearing.

(c) Pier 14

The columns of Pier 14 have failed at the base and the columns are pulled over to the South and supported by the collapsed steelwork. The movement of the tops of the columns to the South was measured as follows:

<table>
<thead>
<tr>
<th></th>
<th>June 25th</th>
<th>July 2nd</th>
<th>July 3rd</th>
</tr>
</thead>
<tbody>
<tr>
<td>East Column</td>
<td>7'-1&quot;</td>
<td>7'-0½&quot;</td>
<td>7'-0¾&quot;</td>
</tr>
<tr>
<td>West Column</td>
<td>6'-7½&quot;</td>
<td>6'-7¼&quot;</td>
<td>6'-7¾&quot;</td>
</tr>
</tbody>
</table>

The concrete of the column shafts has completely failed near the base. The failure is in appearance a moment failure in a plain (un-reinforced) concrete section. Photos Nos. 39 to 43 (inclusive) show the concrete failure.

The failed cross section rotated about a line close to the South face of the column. The reinforcing steel appears to have been totally ineffective and to have failed in bond. It is evident that the large amount of steel in the region where column bars are spliced, together with the small area of concrete outside the bars prevented any large amount of stress being transferred to the steel.

The actual cover runs 3'-4" rather than 2" as called for in the drawings.

No fine cracks are visible above the major crack. There is no evidence that the reinforcing was effective in carrying stress.

The concrete in the South face, in the compression zone is spalled and has failed in compression.

At the top of the West column the bearing of Span 5 has crushed into the concrete of the pier top about 4". The bearing came forward during the collapse to the South edge of the pier top, crushing about 4" into the concrete. The concrete failed in shear out to the South face. Photo No. 46 shows the shear failure of the concrete.

(d) Bent N4

1. Piles. The piles of Bent N4 show little sign of damage. Elevations were taken on the tops of all piles that are accessible and undamaged. They are:

<table>
<thead>
<tr>
<th></th>
<th>West</th>
<th>East</th>
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<tbody>
<tr>
<td>North</td>
<td>8.035</td>
<td>8.052</td>
</tr>
<tr>
<td></td>
<td>8.036</td>
<td>8.040</td>
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<td></td>
<td>8.041</td>
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<tr>
<td></td>
<td>7.909</td>
<td>7.847</td>
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<tr>
<td></td>
<td>8.050</td>
<td>8.042</td>
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<tr>
<td></td>
<td>8.018</td>
<td>8.021</td>
</tr>
<tr>
<td></td>
<td>8.035</td>
<td>8.035</td>
</tr>
</tbody>
</table>

   | South    |             |              |
   |          |             |              |

D.B.Co Dwg. S3703E/ME28 shows the pile tops at Elev. 8.005. The elevations shown above are generally slightly higher than called for on the drawing. The maximum variation in elevation is .032' for the West group. For the East group the third row from the North is low slightly over 1'. This may be due to the fact that this row would receive the full impact of the bent legs during the collapse.

The pile bracing is generally intact. Very few of the welds attaching the bracing to the piles are broken. From a diver's examination it appears that the underwater bracing is as called for on the drawings.

As nearly as can be estimated the centre of the East pile group is .34 feet North and the West pile group, .22 feet North of the position shown on the drawings.

2. Grillage. The upper tier of grillage beams known as the "stringers" have failed completely by buckling of the webs. The lower tier of beams, known as the "grillage beams," have failed locally in web buckling.

The stringers collapsed to the North. When they collapsed the column base plate fetched up at about the level of the top flanges of the lower tier of beams.
leaving an impression of itself in the flanges of the stringers. The impression is clearest in the most Southerly beam.

The collapsing stringers wedged apart the lower tier of grillage beams. On the West leg the two outer grillage beams went overboard.

Photos No. 30 and 71 show general view of Bent N4. In Photo No. 30 it will be noticed that the two outer grillage beams of the West leg are missing.

Photo No. 67 is a view along the stringers facing towards the East leg. The timber blocking, ties, and strongback may be seen. Photo No. 49 shows the most Southerly stringer at near its mid-point. Photo No. 48 shows the top flanges of the stringers near their mid-point.

Photo No. 47 shows pile heads in the two most westerly rows of the West pile group. The piles shown are B1, B2, 1 (at extreme bottom right of picture) and 2. Also shown is the plywood sheet from beneath the column base plate.

Photo No. 53 shows the angles between the grillage beams and the outer end of the stringer webs. It also shows the plywood sheet between the grillage beams and stringers. This photo is of the East end of the East grillage.

Photo No. 54 shows the grillage beams with the flange of a stringer showing above. The plywood between grillage beams and stringers, keeper plates welded to the top flange of the grillage beam, and a diaphragm between grillage beams may all be seen.

Photo No. 70 shows a grillage beam in the East group. It is the third beam in from the East edge. The northwest stringer was originally located over the diaphragm seen in the background. The hole in the grillage beam web was presumably made by the corner of the column base plate.

Photo No. 69 shows the grillage beam immediately East of the beam shown in Photo 70.

Photo No. 55 shows the crippled stringers under the East bent leg. It is taken looking North. The impression of the base plate shows and also the hole in the web of the grillage beam (this is the hole seen in Photo 70). To the left of the photograph is a keeper plate that originally was welded across the top flanges of the two most northerly stringers. On the most northerly stringers are two 3/8" fills which had been welded to the flange beneath this keeper plate.

For some reason the keeper plate must have been 1/4" high over this flange. There is no evidence of fillers under other keeper plates in the same stringer.

Photos No. 50 and No. 66 show the westerly ends of the stringer beams. The impression of the column base plate may be seen.

Photo No. 56 is a view of the stringers under the West column leg. The base plate keeper shows in the centre of the picture.

On most of the pile heads there are shims between the pile head and grillage beam. In some cases these shims appear to have not covered the full area of the pile head.

An attempt was made to measure the depth of the stringers. Due to their distortion this is difficult but the result is thought to be fairly reliable. Near mid-span where the flanges are quite straight and the web also appears straight the distance between the outer edges of the flange was measured. The average of these measurements is taken as the beam depth. Measurements taken were:

<table>
<thead>
<tr>
<th>Stringer No.</th>
<th>4</th>
<th>3</th>
<th>2</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth</td>
<td>36-1/32</td>
<td>36-3/32</td>
<td>36-1/8</td>
<td>36-3/32</td>
</tr>
<tr>
<td>Heat No.</td>
<td>309320</td>
<td>309320</td>
<td>348979</td>
<td>309320</td>
</tr>
</tbody>
</table>

Heat numbers as above were also found on the beams. No. 1 stringer is the most southerly.

The stringers are 36" WF @ 160 rolled by U.S. Steel. Mill certificates give the following tests:

<table>
<thead>
<tr>
<th>Heat No.</th>
<th>C</th>
<th>Mn</th>
<th>P</th>
<th>S</th>
<th>Yield</th>
<th>Ult</th>
<th>% Elongation</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. psi</td>
<td>in 8&quot;</td>
<td>psi</td>
<td>psi</td>
<td>psi</td>
<td>psi</td>
<td>psi</td>
<td>psi</td>
</tr>
<tr>
<td>309320</td>
<td>.26</td>
<td>.69</td>
<td>.022</td>
<td>.050</td>
<td>36,300</td>
<td>73,620</td>
<td>26.0</td>
</tr>
<tr>
<td>348979</td>
<td>.23</td>
<td>.67</td>
<td>.008</td>
<td>.034</td>
<td>36,460</td>
<td>65,100</td>
<td>30.5</td>
</tr>
<tr>
<td>348979</td>
<td>37,570</td>
<td>66,160</td>
<td>30.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(3) Legs and Bracing. The legs of the false bent N4 are straight and undamaged as far as may be seen above water. Below water they were examined by divers. Sketch III shows the legs according to the diver's report. The diver reports the base plates are visible as shown and appear little damaged. He thought the cross strut was still connecting the legs above the shoes but that it was badly damaged.

As far as can be determined the legs themselves are straight until they disappear in the mud.

The East leg is still attached at its top to the truss chord. The rocker bearing appears to be intact. The top of the West leg has moved North of its rocker bearing. The keys in the bearing appear to be sheared off. Photo No. 68 shows the top of the West leg.

(c) Main Trusses

(1) Span 4. The main steelwork of Span 4 is apparently undamaged from Pier 14 out to Panel Point 6. The last two panels, from Panel Point 6 to Panel Point 8, are a twisted mass of steelwork and are almost completely destroyed. The No. 2 traveller that was in Panel 7-8 is completely destroyed.

In falling, the truss caught the tops of the columns of Pier 14. The West column is scored to a depth of 3-4 inches for the upper 10 feet or so of its height. The east column was barely scratched at its top. Photo No. 63 shows the marks on the West column near the top. The photo is taken looking straight down the North face of the column. Photo No. 45 shows the scoring on both East and West columns.

The truss landed on the seal course of Pier 14. The end floor beam just caught on the top of the pedestal. The floor beam was torn from its lower flange which remained bolted to the top chords and top lateral plates. In ripping away from its bottom flange it ripped up the top cover plate of the East truss chord. This shows in Photo No. 82.
(2) Span 5. The main steelwork of the trusses out to Panel Point 6 is generally undamaged. On the East truss there is a compression failure (buckling) in the top chord in Panel 2-3, and in the West truss a compression failure in the diagonal LO-UI.

At L3 in the West truss a bosom splice plate had been removed and was hanging on a "come-along" beside the joint. A splice plate inside the chord was also removed. No damage is apparent at this point. It appears that at the time of the collapse an attempt was being made to bring up the milled ends at this splice, or possibly to insert shims in the joint.

The deck appears intact to Panel Point 4 at which point all the stringers pulled out of the expansion pockets. From U4 on the cantilevered ends of the floor beams have been twisted to the South.

In the upper lateral system all the diagonals running NW-SE are buckled.

The service track has completely disappeared from Span 5. Some of the rail pulled back onto Span 4 and is wrapped around the wreckage of Span 4.

(f) Bearings and Tie Down on Pier 14

On top of Pier 14 the shoes and tie down bars have been pulled over to the South as shown in Photo No. 24.

The rollers in both East and West shoes are collapsed to the South. On both shoes it appears that before collapsing to the South the rollers had been to the limit of their travel to the North and had locked and then slid a short distance to the North. The bar pf, Dwg. S3702/114 which is a $\frac{1}{2}'' \times \frac{1}{4}''$ stainless steel bar welded to the base plate to form part of the dust cover is partly torn loose on both shoes. On Photo No. 65 showing the East bearing the bar pf may be seen in the foreground of the picture just left of centre. It is immediately North of the roller pintles. Adjacent to the bar on the South, or roller, side, are two grooves cut across the full width of the base plate. They appear to be caused by the edge of the segmental roller. To confirm this the marks of the welds attaching pf to the base plate show plainly on the edge of the roller. In Photo 65 these marks are visible on the upper edge of the exposed roller. One mark is visible at the centre line of the roller and another just inside of the West pintle hole. On the West bearing the NW roller fell into the tie down well but the grooves cut by the roller's edge show in the base plate.

A movement to the North is also indicated by the damage the toggle arms caused in the end post of Span 5. The evidence indicates that the pin hole of the East toggle could not be made. A temporary bolt about $1\frac{1}{2}''$ diameter had been placed to secure the toggle until the holes were made fair. This pin sheared in the collapse leaving the toggle arm attached to Span 5 as shown in Photo No. 120. The toggle arm was forced up into the diaphragm and web plate of the end vertical of Span 5 as shown in Photo No. 104. On the West side the toggle had been set up with its 4'' pin in place. In the failure the 4'' pin held and the toggle arm was ripped off the 121\frac{1}{2}'' pin of Span 5 bearing. The whole toggle went down with Span 4. Prior to going down the toggle arm was forced up into the end vertical as shown in Photo 105. The damage to the end verticals could have been caused only by a movement of Spans 4 and 5 towards each other.

Further evidence of movement to the North is shown by the bearing blocks 114B. The East block moved 1\frac{1}{8}'' North and the West block 3 1\frac{1}{2}'' North. These blocks fetched up against metal packing that had been placed between the blocks and the pedestal for Span 4 to take the jacking leg of the toggle. This packing shows in Photo 120.

The "soft" blocking placed between the tie down bars and the bearing blocks 114B on June 11th or 12th was reassembled as shown in Photos 120 and 121 for the East bearing, and Photos 122 and 123 for the West bearing. This blocking was crushed by the tie bars during the northward movement of the bearings.

The final movement of the bearings was to the South. The bearings went South until the tie down bars fetched up against the walls of the concrete well. The rollers fell over to the South during this movement, the southern bearing blocks 114B were pushed South, and the southerly "soft blocking" was crushed. The tie down bars were bent and distorted. Photos 61 and 62 show the East bearing, Photos 58 and 59 the West bearing. The bent and distorted tie down bars are shown in Photos 57 and 60.

In the jacking chambers the steelwork is undamaged, and in the East chamber everything is in its original position except that the jack has fallen off the northern prestressing beam. Photos 96 and 99 (inclusive) show the East chamber. In the West chamber the tie down beams are displaced as shown in Sketch IV. The jack on the South prestress beam is in place, and the jack from the North prestress beam has fallen off the floor. The wedges and blocking are shown in Photos 100 to 103 (inclusive). On the SE corner the wedges have fallen out but the blocking is in place though displaced to the West. On the SW corner both wedges and blocking have fallen. On the NE corner the blocking and wedges are displaced South. Note that the blocking has pushed the concrete dust ahead of it. Presumably this dust and broken concrete resulted from the southward movement of the bearings so that the wedges and blocking were displaced after the bearing moved South. The SW corner is similar to the NE, but here the wedges are no longer on top of the blocking. One piece of blocking lies below the wedges and two pieces on top.

(g) Top Chord Tie Plates 302A

The bolts connecting the top chord tie plates 302A to the webs of the top chord of Span 4 sheared during the collapse. The tie plates were left hanging on the pin in the top chord of Span 5. They show in Photo No. 111.
They hit the top of Pier 14. Photo No. 82 is a view looking straight down on the end of the East top chord of Span 4. The buckled web plates may be seen curled inwards. There are 4 bolts still in the web plates and these will be examined when the piece is salvaged. Photo No. 26 is a close-up of the webs showing the 4 bolts in place.

The condition of the West chord is very similar to that of the East chord, but no bolts are left in place.

The tie plates 302A and 302K have been taken down and plates 302A examined by Professor W. M. Armstrong of the Department of Mining and Metallurgy, University of British Columbia, to determine if possible, the direction in which the bolts failed. The tie plates are shown in Photos 127 to 134 (inclusive). The plates were numbered 1 to 8 from West to East. Plates Nos. 1, 4, 5 and 8 were 302A's: plates Nos. 2, 3, 6 and 7 were 302K's. The plates 302A were bolted outside the webs of Span 4. The holes burned to reconnect the plates after landing on N4 may be seen in the photographs. At present it has not been possible to definitely establish which holes were filled.

On the West truss, the East face of tie plate No. 1 (Photo No. 127) and the West face of plate No. 4 (Photo No. 132) were in contact with the webs of Span 4 top chord. On the East truss the East face of plate No. 5 (Photo No. 129) and the West face of plate No. 8 (Photo No. 134) were in contact with the webs of Span 4.

Several "spatters" of metal from the burnt holes were noticed on the contact faces of the tie plates. These may have caused score marks on the connecting web plates. This cannot be determined until the web plates are salvaged.

Some holes have marks probably caused by "rags" on the bolts. These marks seem to point generally towards the pin, indicating a tension failure, but they are not conclusive. There is no evidence of failure due to a movement in the bolt group.

Additional information may be available when the connecting webs are salvaged and Professor Armstrong's investigation is completed.

The tie plates are BS968 steel and have been flame-hardened around the burnt holes.

The burnt holes are ragged, as may be seen in the photograph. It is possible that the sharp notches would cause early failure in the bolts, possibly soon after they slipped into bearing. Also the uneven holes would tend to cause the bolts to fail one at a time as they came into bearing.

(h) **Top of Pier 13**

The ends of the lower chords of Span 3 and Span 4 as seen on top of Pier 13 have run into each other. Span 4 moved North to run into Span 3 as is shown by the damage to the lower chords and also the evidence of the sheared anchor bolts of Span 4.

Photos 113, 114 and 115 show the corresponding damage to the ends of the West lower chords of Span 3 and 4 and Photos 116 to 119 (inclusive) show the corresponding damage to the East chord.

There is no evidence that Spans 3 and 4 were in contact at any point other than the lower chords.

(i) **Traveller No. 1**

The condition of traveller No. 1 is fully described in the report by R. N. McLellan, P.Eng.

(j) **Locomotive and Train**

The locomotive, bogies, and the chord section L7-L8 which was on the bogies are now in the water.

The locomotive was at about Panel Point 5 at the time of the collapse and the bogies with the chord L7-L8 on them, were one in panel 5-6 and one in panel 6-7. The present position of the locomotive and chord section on the bottom is shown in Sketch V.

(k) **Toggles on Pier 14**

The damage to the end verticals of Span 5 caused by the toggle arms has been described above.

On the West truss the toggle arm tore off the pin of Span 5. The 4" toggle pin held and both arms of the toggle went down with Span 4. The toggle arm that had been connected to Span 5 was dragged down the face of Pier 14 as Span 4 fell and was driven into the diaphragm at the end of the lower chord of Span 4. It may be seen in Photos Nos. 124 and 125.

The toggle arm was salvaged and is shown in Photos Nos. 135 and 136. In Photo 135 the marks on the bottom face of the toggle from its contacts with the pier face are clearly seen.

7. **CHECK OF PIER 14 DESIGN**

The design by Swan, Wooster and Associates of Pier 14 has been checked and the design found to be in accordance with the specifications and good practice.

8. **CALCULATION OF LOADS, REACTIONS AND STRESSES**

(a) **Loads at time of collapse**

The loads on the bridge at the time of collapse have been calculated and are summarized on Drawing 084-8. The basis of the various calculations is as follows:

1. **Weight of Structural Steel.** The dead load of structural steel was calculated and the weights compared with Dominion Bridge Company weights. Any discrepancies were discussed with Dominion Bridge Company engineers and agreement on correct values reached with them.

Final weights were calculated on the basis of the theoretical weight of shapes and plates, with cuts and cope deducted and with half the allowable over-run added to plates. Shop rivets were figured at the actual weight of rivet heads. Field bolts were figured at the
actual weight of head, nut, washers, and the shank less the grip. The number of bolts were estimated from the shipping bills. Shop paint was allowed for by adding 1/2 per cent to the total weight.

The steel weights were distributed to top and bottom chord panel points in accordance with the actual location of the members. In the case of main chords the distribution to panel points was made according to the actual position of the center of gravity of the chord. The distribution of steel to panel points is summarized in Dwg. 084-7.

(2) Weight of Traveller No. 1. The Dominion Bridge Company calculations of the weight and reactions of the No. 1 Traveller were examined and reviewed. It was established that the Dominion Bridge Company had used reasonable weights for all items of machinery and equipment and that their weight calculation had been adequately checked by their own organization. The Dominion Bridge Company weight calculations have been accepted as correct.

(3) Locomotive and Cars. The weight of the locomotive and cars was established by weighing the equipment after it was salvaged.

The location of the service train and the crane boom was established by inquiry and by considering the normal procedure for spotting loads under the crane.

(4) Miscellaneous Erection Equipment and Loads. An itemized list of all equipment on the span was made and the weights estimated as accurately as possible. The items of equipment and the method of estimating their weights were as follows:

(i) Gas Mains: Two 10 inch diameter gas mains extended out to P. Pt. 4. The weight of pipe and hangers was calculated.

(ii) Maintenance Tram Beams: Two tram beams provided for future maintenance had been placed out to P. Pt. 7. The weight of beams and connections was calculated.

(iii) Service Track: The service track consisting of two 60 lb. rails, ties, and planking was located outside the West Truss. The weights were calculated from the actual dimension of the material and estimated unit weights.

(iv) Service Walkway: A timber service walkway was located inside the service track. The weight was calculated from the actual dimensions of the material and estimated unit weights.

(v) Air Pipe: A 3" aluminum pipe carried air for erection equipment. Its weight was calculated from its actual dimensions and estimated unit weight.

(vi) Men: An allowance was made for the weight of workmen on the span by assuming that thirty men at 150 lbs. per man were concentrated at P. Pt. 7.

(vii) Miscellaneous Tools and Equipment: By inquiry it was established that miscellaneous tools and equipment such as small tools, welding equipment, gas bottles, small wrenches, barrels of oil, kegs of bolts, rigging etc. were on the span in the vicinity of Traveller No. 1. An estimate of the weights of these items was made and the weight taken to be located at P. Pt. 6.

(viii) Miscellaneous Erection Material at P. Pt. UO: The weights of top chord tie plates, traveller cross over plates, top chord pins, and end post ties were calculated from actual steel dimensions.

(ix) Toilet Shack: The weight of a toilet shack located near P. Pt 3 over the West Truss was estimated.

(x) Cat Walk: Along the lower chord a timer cat walk extended to P. Pt 6. The weight was calculated from the actual dimensions of the material and estimated unit weights.

(xi) Bolt Storage: A platform for bolt storage was located on the lower chord at P. Pt. 4. The weight of the platform and stored bolts was estimated.

(xii) Ladder: A ladder from the lower chord cat walk to the upper deck was located at P. Pt. 3. The weight of the ladder was estimated.

(b) Reactions at time of Collapse

The reactions at the time of collapse are shown on Drawing 084-8. These reactions have been calculated for the East and West Truss individually as if the trusses were not connected transversely. The effect of the sway bracing that connects the trusses transversely would be to decrease the reactions of the West Truss and to increase the reactions of the East Truss, that is it would tend to equalize the reactions of the two trusses. The effect would be small and would not be significant.

The reactions shown at Pier 14 are the reactions in the tie down chambers and include the weight of the tie down bars and their anchoring arrangements.

(c) Truss Stresses at time of Collapse

The stresses in all main truss members at the time of collapse are shown in Dwg. 084-9. For purposes of comparison the actual stress and the allowable stress is shown for each member. Allowable stresses are the basic unit stresses allowed by the AASHO Specifications.

9. SPECIAL INVESTIGATION AND TESTS

(a) Investigation of Prof. A. Hrennikoff

In order to evaluate the strength of the stringers in the grillage of bent N4 and to determine the effect of their strength and stability of the plywood pads used between the grillage beams and stringers, and between the stringers and column base plate, Prof. A. Hrennikoff of the University of British Columbia was requested to carry out a program of mathematical investigation and laboratory testing.
Prof. Hrennikoff has studied mathematically the state of stress and the elastic stability of the stringers at the time of the collapse. He also carried out tests to determine the stress-strain characteristics of plywood and tests to determine, if possible, the effect of plywood pads on the ultimate load carrying capacity of samples of beams similar to the beams used as stringers in the bridge.

The results of these studies and tests are contained in the report submitted by Prof. Hrennikoff.

(b) Investigation of Prof. Armstrong

Prof. Armstrong of the University of British Columbia was asked to study the physical properties of the steel in the stringers of bent N4. He also was asked to study the tie plates 302A to determine if possible the manner in which the bolts connecting these tie plates failed.

G. S. Eldridge & Co. Ltd. carried out physical tests on sections of the stringers under the direction of Prof. Armstrong. The results of Prof. Armstrong's studies are contained in Prof. Armstrong's report together with his comments upon the test results obtained by G. S. Eldridge and Co. Ltd.

The test results themselves together with comments on the steel and manufacture of the stringers are contained in a separate report by G. S. Eldridge & Co. Ltd.

(c) Investigation of Traveller etc. by R. McLellan, P. Eng.

Mr. R. McLellan, P. Eng., was asked to investigate the No. 1 Traveller, locomotive and cars to determine if any mechanical failure or fault might have caused or contributed to the collapse.

Mr. McLellan has submitted the results of his inquiry in a separate report. He found no evidence of any mechanical defect or failure in the travellers, crane or hoisting equipment nor in the locomotive or cars. It is concluded that this equipment in no way contributed to or caused the collapse.

10. SEQUENCE OF EVENTS DURING COLLAPSE

The sequence of events following collapse of the stringers at the base of Bent N4 may be followed in considerable detail as follows:

(1) The webs of the stringers at the base of bent N4 crippled and the stringers collapsed to the North (see Para. 1.6 (d) (2)).

(2) As the webs collapsed, the bent N4, and with it the whole superstructure, started to fall with the superstructure rotating about the bearing on Pier 14. (See Dwg. 084-11).

(3) The dynamic reactions at the bearing on Pier 14 would tend to move the bearing downward and to the North. The bearing was restrained from moving downward but was relatively free to move northward on the roller nest. (See Dwg. 084-4 and Para. 1.6 (f)).

(4) As the superstructure rotated on the bearing, the joint UO moved southward, shearing the total of 40-1" H.T. bolts in the four tie plates 302A. (These bolts probably sheared an instant after the collapse started and would likely shear with a loud "crack." (See Para. 1.6 (g)). Prior to failure, they probably pulled Span 4 slightly southward (see Para. 1.6 (b) (4)).

(5) The collapse of the stringers was arrested by the lower tier of grillage beams. When the stringers brought up on the grillage beams the base plate of the columns fetched up on the collapsed stringers. The base plates would be tilted somewhat, down to the north (see Dwg. 084-11, Diagram A.)

(6) When the base plates fetched up, the fall of the span was momentarily arrested. At this instant the fall at bent N4 had been 3' to 3½'. The corresponding drop of the outer end of the span would be about 6 to 7 feet, and at P. Pt. 7 would be 5½' to 6½' feet (see Dwg. 084-11, Diagram A.)

(7) Beyond a point close to P. Pt. 6, the vertical acceleration of the freely falling span would be greater than the acceleration due to gravity. Objects not connected to the span would in this case drop more slowly than the span. A man standing near the end of the span would have the impression that the span fell out from under him. When the fall was momentarily arrested as the column base plates fetched up on the grillage beams the freely falling objects probably "caught up" with the span.

(8) When the fall was arrested a very heavy load would come into the columns of bent N4. Since the base plates were tilted to the north the load would cause the columns to bend to the north and to buckle at a point about one quarter of their height, that is about 25 feet, up the leg. As soon as the legs started to buckle, they would cease to provide support for the span and the span would again start to fall freely, still rotating about the bearing on Pier 14. (See Dwg. 084-11).

(9) As the fall continued the legs of N4 would fold at the buckle about 25 feet up. The base plates would rotate with the lower part of the column leg until when the leg had bent through about 90 degrees so that its lower part became horizontal the base plates would be approximately vertical. Shortly before this time the base plates had slipped northward and on the east leg the N.W. corner of the base plate punched through the web of the grillage beam. (See Para. 1.6 d (2) and Photo No. 70). As the fall continued the horizontal cross strut between the two legs came to bear on the top flanges of the grillage beams. This prevented the base plates dropping further and accounts for the lack of damage to the piles and bracing. As the leg continued to drop the lower parts of the legs rotated about the strut supported on the grillage beams. This raised the base plates sufficiently so that when the strut finally slid off the ends of the grillage beams the base plate barely "nicked" the edge of the pile cluster as it fell into the water (see Dwg. 084-11, Diagram D).

(10) At this stage the column legs were falling through the water with the lower 25 feet bent back at a sharp
angle. When the legs finally hit bottom, they were driven into the mud and the lower ends, with the base plates, were further folded back along the upper part of the leg. (See Dwg. 084-11, Diagram F and Sketch III).

(11) When the stringers collapsed and the base plates landed at the end of the initial drop, a powerful wedging action was exerted by the stringers on the grillage beams. This forced the two easterly grillage beams east and the two westerly grillage beams west. On the west side the two westerly grillage beams were forced completely off the bent and fell into the water.

(12) The bearings on Pier 14 would continue to move north under the action of the dynamic reactions until the span had rotated about 3 degrees. During this northward movement the “soft” blocking around the tie down bars would be crushed but since the forces involved were not large, severe crushing would not take place. Evidence from the roller nest indicates a northward movement of about 7 inches at the pier (see Dwg. 084-4).

(13) As the span rotated past about 3 degrees, the movement of the bearing on Pier 14 reversed and the bearing moved south. As the fall progressed, the forces causing southward movement became very large. The bearings moved to the full limit of their travel in a very short time and the tie down bars came up against the concrete walls of the well. The dynamic reaction then increased rapidly and when the span had rotated through about 10 degrees the reactions were sufficient to cause failure at the base of the pier shafts.

(14) When the span had rotated about 13 degrees the outer end landed on the base of false bent N5. On the west truss the lower chord L7-L8 had not been placed so that the vertical punched through the bent and folded up as it did so. It offered little resistance. On the east truss all members were placed to P. Pt. 8 and considerable resistance would be developed while the main truss members failed. In effect the span hung up on its S.E. corner. This would cause a heavy reaction at the N.W. corner, on Pier 14. A measure of this reaction can be obtained from the local buckling in the diagonal L0-U1 West. It appears that the force was sufficient to cause the concrete failure beneath the west bearing in Pier 14. The same cause loaded the east top chord and the N.W.-S.E. running diagonals of the top lateral system in compression. All the diagonals buckled and the top chord buckled in Panel 2-3. (See Para. 1.6 (e) (2)).

(15) When the collapse started the train with chord L7-L8 was at the outer end of the span. The locomotive was about at the point where the vertical acceleration of the span equalled the acceleration of gravity and the cars with the chord sections were beyond this point. During the initial drop the cars and load fell more slowly than the bridge and the locomotive fell at about the same rate as the bridge.

The 51.6 ton chord section would fall more slowly than the bridge, and more slowly than the crane which was rigidly attached to the bridge. During the initial fall the chord section would rise a few inches relative to the span. Due to the rotation of the span, the head of the crane boom would move south relative to the chord. When the span momentarily paused after the initial fall, the chord caught up with the span and tightening the fall line jerked the boom back to the north where it ran into the western stiff leg. (See Photo 148). When the fall continued the chord again rose relative to the span so that the fall line was slack. Due to centrifugal action the boom swung till it pointed almost Southwest. When the span landed, the chord continued falling until the boom line tightened. Since the lead of the boom line was well to the north the line probably jumped some of the sheaves in the fall blocks and the jerk as the line came taut cut the line at the sheaves. The lower block must have swung in an anti-clockwise direction to land gently upon the timber access trestle. During this swing the fall line, which now had parted, was pulled through the blocks and partially unroved. The chord section would tend to fall vertically but the pull of the fall line started it rotating so that it hit the water in an almost vertical position. When the north end of the chord hit the bottom the whole member fell over to land upside down and reversed for end. At some stage of the fall, either when the span hit bent N5 or when the span finally landed, the chord crashed through the floor system.

The locomotive would have fallen at about the same rate as the span. Probably the locomotive slipped forward a short distance until its rate of fall equalled that of the span. It then fell vertically until the failure of the floor system due to the chord crushing through it toppled the locomotive over the side and into the water.

(16) When the base of Pier 14 failed, as outlined in 13 above, the top of Pier 14 moved south about 7 feet. This pulled the support from under the bearings of span 4. The pier top would move forward quite rapidly and the lower chord of span 4 almost cleared the north side of Pier 14.

On Pier 13 there would initially be a tendency for the bearings to move north but before the top chord had fallen to the level of the top of Pier 14, this tendency would be reversing to a southward movement. When the top chord hit the top of Pier 14 the whole of span 4 was forced to the north about 1'10" until the end of the top chord cleared the north face of Pier 14. (See Dwg. 084-10). This movement to the north sheared the two north anchor bolts in each bearing of span 4 on Pier 13 and lifted the bed plate off the south anchor bolts. The north end of span 4 ran into the south end of span 3 and shifted about 6 inches east. (See Para. 1.6 (b) and Dwg. 084-10).

(17) As span 4 continued to fall rotating about its bearings on Pier 13 the span started to move south under the action of the dynamic forces. During the southward movement the shoes on Pier 13 were dragged forward plowing into the concrete the two S.E. anchor bolts. Due to the shift of about 6" east the shoes cleared two S.W. bolts. The shoes finally slipped over
the edge of the pier top and ended up wedged between the span and the south face of the pier (see Para. 1.6 (b)).

(18) The south-west corner of span 4 finally landed on the seal course of Pier 14, and the S.E. corner cleared the seal and continued on to the bottom. The end floor beam hung up on the pedestal of Pier 14. A length of the bottom flange of the end floor beam has been torn away from floor beam and left attached to the east top chord. In tearing away, it tore loose a section of the top chord cover plate. This may have happened when the floor beam landed on the pier pedestal or may have happened at an earlier stage as the top chord passed the top of the pier. At this earlier stage some of the stub stringers would have hit the top of the pier and possibly sufficient force was developed to fracture the floor beam web at this time. The breaks have the appearance of high speed fractures and this favours the hypothesis that the damage occurred at the earlier stage.

(19) Spans 4 and 5 both finally came to rest with their north ends still supported upon Piers 13 and 14 respectively, and with their south ends on the harbour bottom (see Dwg. 084-3).

11. CAUSE OF COLLAPSE

(a) Investigations

As has been pointed out the sequence of events following the failure of the stringers in the grillage of bent N4 is clear and virtually all the observed damage can be accounted for as a consequence of failure of the stringers.

A careful search has been made for evidence of any unusual condition and for any prior failure elsewhere that could have caused failure of the stringers. No evidence has been found that suggests any failure or contributory cause prior to failure of the stringers in bent N4.

(b) Possible Displacement of Tie Downs on Pier 14

It has been suggested that possibly the wedges and blocking securing the tie down bars in the West Reaction Chamber of Pier 14 may have become displaced during the last passage of the locomotive and train.

As far as it has been possible to ascertain there is no definite evidence to either prove or disprove this possibility. While no proof exists that the wedges etc. were displaced, there is no evidence that is inconsistent with this supposition.

It is possible that if the stringers in bent N4 were already in a critical condition a slight change of stress or a sudden small displacement of the tops of the columns of N4 due to a movement of the west tie downs could have been the immediate cause of the stringer failure.

(c) Grillage Failure, Bent N4

There is no evidence of any failure prior to the collapse of the stringers in the grillage of bent N4 other than possibly the displacement of the west tie downs on Pier 14. It remains only to determine if these stringers were in fact in a critical condition and if it is probable that failure would occur spontaneously or due to a minor disturbance from some external cause.

Prof. Hrennikoff has shown in his report that analysis of the stringers in the light of information obtained from the various tests performed indicate that the stringers were in a highly critical state and that failure was not only likely but almost inevitable.

The results of Prof. Armstrong’s investigation and the tests carried out by G. S. Eldridge and Co. Ltd. on the physical properties of the stringers reinforce the conclusions reached by Prof. Hrennikoff. The fact that the second stringer from the north was slightly higher than its neighbours and at the same time was of marginally low strength would aggravate the conditions found by Prof. Hrennikoff to be already critical.
### TABLE I

<table>
<thead>
<tr>
<th>Time</th>
<th>Height</th>
</tr>
</thead>
<tbody>
<tr>
<td>H.W. 0509/17</td>
<td>11.4</td>
</tr>
<tr>
<td>L.W. 1254/17</td>
<td>0.3</td>
</tr>
<tr>
<td>H.W. 2024/17</td>
<td>12.8</td>
</tr>
<tr>
<td>L.W. 0132/18</td>
<td>8.9</td>
</tr>
</tbody>
</table>

_Tides, Vancouver Harbour_

<table>
<thead>
<tr>
<th>Time</th>
<th>Max. Velocity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Turn to flood 0102/17</td>
<td>0323/17 1.4 kn.</td>
</tr>
<tr>
<td>Turn to ebb 0533/17</td>
<td>0937/17 4.6 kn.</td>
</tr>
<tr>
<td>Turn to flood 1304/17</td>
<td>1638/17 4.8 kn.</td>
</tr>
<tr>
<td>Turn to ebb 2038/17</td>
<td>2259/18 2.0 kn.</td>
</tr>
</tbody>
</table>

_Current, Second Narrows_


All times—Pacific Daylight Time.

### TABLE II

<table>
<thead>
<tr>
<th>Location</th>
<th>Ultimate Strength psi</th>
<th>Cement sacks per yd.</th>
<th>Mix Slump</th>
<th>Entrainment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pedestal</td>
<td>3285</td>
<td>3340</td>
<td>5½</td>
<td>3</td>
</tr>
<tr>
<td>&quot;</td>
<td>3265</td>
<td>—</td>
<td>5</td>
<td>3</td>
</tr>
<tr>
<td>&quot;</td>
<td>3535</td>
<td>3360</td>
<td>5</td>
<td>3</td>
</tr>
<tr>
<td>Columns</td>
<td>3185</td>
<td>3155</td>
<td>5</td>
<td>3½</td>
</tr>
<tr>
<td>&quot;</td>
<td>3130</td>
<td>3265</td>
<td>5</td>
<td>3</td>
</tr>
<tr>
<td>&quot;</td>
<td>3845</td>
<td>3360</td>
<td>5</td>
<td>3</td>
</tr>
<tr>
<td>Cross Beam</td>
<td>3185</td>
<td>3240</td>
<td>5½</td>
<td>3</td>
</tr>
</tbody>
</table>

28 day compression tests on 6' dia. x 12' cylinders—MacDonald and MacDonald Ltd. Exhibit 144.

### TABLE III

<table>
<thead>
<tr>
<th>Pier</th>
<th>Set by S/W before Collapse</th>
<th>Chainage Measured after Collapse</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>67 + 75.450</td>
<td>67 + 75.450</td>
<td>—</td>
</tr>
<tr>
<td>13</td>
<td>70 + 57.110</td>
<td>70 + 57.074</td>
<td>.036 N</td>
</tr>
<tr>
<td>14</td>
<td>73 + 42.030</td>
<td>73 + 42.078</td>
<td>.048 S</td>
</tr>
<tr>
<td>15</td>
<td>78 + 08.405</td>
<td>78 + 08.402</td>
<td>.003 S</td>
</tr>
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</table>

Chainage, Piers 12, 13, 14, 15
# APPENDIX A

## BENT N4—PILE DRIVING RECORDS

### North

<table>
<thead>
<tr>
<th>Pile No.</th>
<th>Penetration</th>
<th>Resistance</th>
<th>Pile No.</th>
<th>Penetration</th>
<th>Resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td>B11</td>
<td></td>
<td></td>
<td>B13</td>
<td>123</td>
<td>164</td>
</tr>
<tr>
<td>B12</td>
<td>128</td>
<td>190</td>
<td>B14</td>
<td>122</td>
<td>120</td>
</tr>
<tr>
<td>B13</td>
<td>128</td>
<td>80(1)</td>
<td>B15</td>
<td>127</td>
<td>127</td>
</tr>
<tr>
<td>B14</td>
<td>122</td>
<td>60(2)</td>
<td>B16</td>
<td>122</td>
<td>60/2&quot;</td>
</tr>
<tr>
<td>B15</td>
<td>129</td>
<td>250</td>
<td>B17</td>
<td>122</td>
<td>250</td>
</tr>
<tr>
<td>B16</td>
<td>128</td>
<td>326</td>
<td>B18</td>
<td>6</td>
<td>120</td>
</tr>
<tr>
<td>B17</td>
<td>120</td>
<td>150</td>
<td>B19</td>
<td>7</td>
<td>150/6&quot;</td>
</tr>
<tr>
<td>B18</td>
<td>120</td>
<td>154</td>
<td></td>
<td>8</td>
<td>154/6&quot;</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>157</td>
<td></td>
<td>10</td>
<td>159/6&quot;</td>
</tr>
<tr>
<td></td>
<td>11</td>
<td>152</td>
<td></td>
<td>12</td>
<td>152/6&quot;</td>
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### South

**NUMBERING OF PILES IN BENT N4**

<table>
<thead>
<tr>
<th>Pile No.</th>
<th>Penetration</th>
<th>Resistance</th>
<th>Pile No.</th>
<th>Penetration</th>
<th>Resistance</th>
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<tbody>
<tr>
<td>B7</td>
<td>120</td>
<td>150</td>
<td>B7</td>
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<tr>
<td>B8</td>
<td>120</td>
<td>154</td>
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<tr>
<td>B9</td>
<td>122</td>
<td>127</td>
<td>B9</td>
<td>122</td>
<td>127</td>
</tr>
<tr>
<td>B10</td>
<td>122</td>
<td>126</td>
<td>B10</td>
<td>122</td>
<td>126</td>
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<tr>
<td>B11</td>
<td>121</td>
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<tr>
<td>B14</td>
<td>124</td>
<td>132</td>
<td>B14</td>
<td>124</td>
<td>132</td>
</tr>
</tbody>
</table>

(1) Increased to 279 blows per foot after 11 days.

(2) Increased to 340 blows per foot after 12 days.

Resistance given as blows per foot for last foot of driving unless shown otherwise. Thus "134" indicates 134 blows for the last foot of driving. "105/4" indicates 105 blows for last 4" of driving.

Records taken from Dominion Bridge Co. Ltd. records. Exhibit 114.
Axle Spacing

Load per Axle

SERVICE TRAIN

Revised Aug 30/58  SKETCH I
ELEVATIONS ON PEDESTAL - PIER 13

ELEVATIONS ON PEDESTAL - PIER 14

SKETCH II
EAST

Elev. +8.0'
W.L.?

West

3'-0"
Elev. -27.0

FORESHORTENED VIEW
LOOKING SOUTH

POSITION OF LEGS - BENT N4

SKETCH III
PRESENT POSITION OF TIE-DOWN BEAMS
WEST CHAMBER

SKETCH IV
PLAN OF LOCATION
OF SERVICE TRAIN AND LOAD AFTER FALL

Revised Sept. 3, 1958

SKETCH IV
WEST TRUSS

TOP CHORD

Security Train Load
- Uplift to Concrete

TOTAL AT PANEL

RECO NSTRUCTED

May 17, 1974

SECOND HARRI SONS BRIDGE INVESTIGATION

NORTH ANCHOR SPAN

LOCATION AND WEIGHTS OF ALL LOADS AT TIME OF COLLAPSE

A.B. SANDERSON AND COMPANY LTD.
CONSULTING ENGINEERS

BENT N4

DETAILS

No. DATE DESCRIPTION

SECOND HARRI SONS BRIDGE INVESTIGATION

NORTH ANCHOR SPAN

LOCATION AND WEIGHTS OF ALL LOADS AT TIME OF COLLAPSE

A.B. SANDERSON AND COMPANY LTD.
CONSULTING ENGINEERS

BENT N4

DETAILS

No. DATE DESCRIPTION

084-8
### Table 1: Member Loads Actual Stress

<table>
<thead>
<tr>
<th>Member</th>
<th>Load Description</th>
<th>Material</th>
<th>Actual Stress</th>
<th>Section</th>
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</thead>
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### Table 2: Tension and Compression

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<th>Load Description</th>
<th>Material</th>
<th>Actual Stress</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
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### Table 3: Miscellaneous

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<thead>
<tr>
<th>Member</th>
<th>Load Description</th>
<th>Material</th>
<th>Actual Stress</th>
<th>Section</th>
</tr>
</thead>
<tbody>
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</table>

### Table 4: Service Loads

<table>
<thead>
<tr>
<th>Member</th>
<th>Load Description</th>
<th>Material</th>
<th>Actual Stress</th>
<th>Section</th>
</tr>
</thead>
<tbody>
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<td></td>
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</tbody>
</table>

### Table 5: Erection Stresses at Time of Collapse

<table>
<thead>
<tr>
<th>Member</th>
<th>Load Description</th>
<th>Material</th>
<th>Actual Stress</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**
- Tension
- Compression
ROYAL COMMISSION
SECOND NARROWS BRIDGE INQUIRY

JOINT REPORT
by
F. M. Masters J. R. H. Otter
J. R. Giese R. Freeman

Engineering Consultants and Advisers
to the Commission

30 September, 1958

Law Courts
Vancouver, B.C.
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<td>III</td>
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II Description of accident as observed by eyewitnesses .......... 61
III Condition of the structure after the accident ................. 62
IV Discussion of possible causes of the accident .................. 63
V Theoretical reconstruction of the progress of the accident based on assumed initial failure of the top tier grillage at Bent N4 West ........... 68
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SECTION I
INTRODUCTION AND SUMMARY

1.1 We, the Engineering Consultants and Advisers appointed by the Commissioner, having completed our investigations into the collapse during erection of the Second Narrows Bridge, are unanimously agreed upon the cause of the collapse and now present this joint account of our work.

1.2 Our conclusion as to the cause appears in paragraph 4.30 of this report and is as follows:

The primary cause of the accident is elastic instability of the webs of the stringer beams of the N4 grillage, accentuated by the plywood packings above and below the beams. The instability was due to the omission of stiffeners and effective diaphragming in the grillage, and this in turn was basically due to an error in the calculations. Such diaphragming as was provided was inadequate.

1.3 Immediately upon our appointment at the end of July, 1958 we travelled to Vancouver to inspect the collapsed structure prior to the commencement of salvage work. At that time we examined the exhibits filed with the Commission, with particular reference to the transcripts of eye-witness testimony and to the design and erection drawings and calculations. We conferred with Commission Counsel with regard to procedure; and with Mr. A. B. Sanderson with regard to supervision of salvage work on behalf of the Commission. We also had the benefit of informal discussions with representatives of the British Columbia Toll Highways and Bridges Authority, with Messrs. Swan and Wooster their Consulting Engineers, with representatives of the Dominion Bridge Company and with other interested parties.

1.4 Following our first visit we proceeded to conduct two entirely separate and independent investigations based upon all the data then available to us, one by Messrs. Masters and Giese and the other by Messrs. Otter and Freeman, at our respective offices in Harrisburg, Pennsylvania, and London, England.

1.5 We reassembled in Vancouver on 25 September, 1958, since when we have reviewed our computations and findings and have taken into account all the additional information now available; in particular the following reports:

(a) Report on Investigation, dated 22 September, 1958, by Mr. A. B. Sanderson, P.ENG.

(b) Report on Role of Grillage of the Falsework Bent N4 (undated) by Professor A. Hrennikoff, Sc.D.

(c) Report on Investigation of the 110-ton travelling crane (Traveller No. 1), dated 14 August, 1958, by Robert N. McLellan, P.ENG.

(d) Report on Examination and Testing of the Falsework Top Grillage Stringers Nos. 2 and 3, dated 19 September, 1958, by G. S. Eldridge & Co. Ltd.

(e) Comments on Results of Mechanical and Metallographic Tests on Top Grillage Beams from Falsework N4, dated 19 September, 1958, by Professor W. M. Armstrong.

(f) Report on Examination of Connecting Plates 302A and on Single Shear Tests on Bolts similar to those used in plates 302A, dated 23 September, 1958, by Professor W. M. Armstrong.

1.6 We have reviewed the salvage work accomplished to date. We are satisfied that all material likely to throw any light on the cause of the collapse has been recovered; we have examined that material and find that its condition is consistent with our findings and does not affect or alter our conclusions in any way.

1.7 A narrative is first given, based on the eyewitness accounts, outlining the probable sequence of events during the accident. Further details are then deduced from the condition of the span after the accident. The loading of the structure both before and at the time of the accident are considered, together with the ability of the structure to carry the loads. This is followed by a discussion of possible causes of the accident, finishing with the conclusion as to the cause. Finally, a theoretical reconstruction is given of the course of the accident following on an initial failure of the top tier grillage at Bent N4, with the object of accounting for the whole of the evidence at present before the Commission.

1.8 Except where there are special reasons for not doing so, all loads and forces stated in this report are expressed in Kips (short for Kilopounds, i.e. units of 1,000 lbs.) and unit stresses in Kips per square inch (ksi). Throughout the report the north anchor span is described as Span 5 and the adjacent approach span as Span 4. The piers are numbered as in the relevant contract drawings. All other references to parts and locations are in accordance with the nomenclature used in Mr. Sanderson's report.

SECTION II—DESCRIPTION OF ACCIDENT AS OBSERVED BY EYEWITNESSES

2.1 The course of the accident, as interpreted by us from the recorded evidence of eyewitnesses, is set out below in continuous narrative form. The evidence supporting the various statements is referred to by letters in brackets, e.g. (a); and in the Appendix attached are listed the page numbers in the Transcript of Evidence relating to the indicating letters.

2.2 Before the accident the construction of the bridge had reached the stage indicated in exhibit No. 296-19B. The erection of the steelwork in all spans up to and including span 4 was effectively complete and these spans were supported on their permanent bearings, although the fixed bearings of span 4 had not been finally bolted down and grouted. A day or so before the accident the erection crane (known as Traveller No. 1) on span 5 had been moved from panel 5-6 to panel 6-7, and properly connected to the struc-
ture. At 2 a.m. on June 17th, i.e. during the night before the accident, one witness stated that he heard a sharp noise like a rifle shot from span 4 (a). On the morning of the accident one witness said that the front end of the bridge seemed to take a "duck" or "dip" (b). During the day of the accident one bottom chord, one top chord, one diagonal and two verticals were erected in panel 7-8 of span 5. Fifteen to thirty minutes before the accident occurred a train, consisting of chord BC8W (weighing 52 tons) resting on two bogie cars and pushed by a locomotive, had travelled from the north end of the bridge to a position near the end of span 5 suitable for the chord to be lifted by the crane (c). The chord had been prepared for lifting by the attachment to the crane hook of slings which were themselves connected to spreader beams bolted to the chord; and the chord falls were made taut but without lifting the chord (d).

2.3 Very soon after the making taut of the crane falls to the chord, span 5 started to fall, whilst still supported on pier 14, and tilted slightly down to the west at first (e). Almost simultaneously a noise or noises were heard. The noises were variously described, but the probability seems to be that there were two distinct noises, a loud cannon-like explosive crack above the top of pier 14 (f) and a rumbling and not well localized noise similar to thunder (g). At about the time of the beginning of the collapse, the bottom chord BC8W swung southwards on the hook of the crane (h). After a short initial fall, estimated at different positions on the span as about three to six feet (i), the falling span appeared to hesitate (j). After the hesitation the span continued to fall, still remaining supported at the north end on pier 14. The columns of N4 were not bent at this stage (k).

2.4 During the second stage of the fall of span 5 after the hesitation, the column legs of falsework N4 were observed entering the water north of the pile nest (l), appearing straight to one witness and buckled about one-quarter of the way down the legs to another (m). (This latter observation is clearly incorrect; almost all the upper half of the bent was visible above water after the collapse and this part was still straight). The buckling of the legs let the span down further (n). At the same time the top of pier 14, which had been vibrating (o), began to move southwards (p), breaking the bottom of the pier and removing the support from under the south end of span 4 which then began to fall whilst remaining supported on pier 13. Span 5 fell before span 4 (q). The south end of span 4 scraped down the top of pier 14 (r). Span 5 entered the water and came to rest whilst span 4 was still falling (s). Span 3 vibrated slightly for a short time whilst span 4 was falling (t). During the collapse there was a cloud of dust around pier 14. At certain positions in the spans, mainly towards their southern ends, the spans fell faster than the men standing or sitting on them (u). One witness stated that about the time of the hesitation in the fall of span 5, the ends of the two spans over pier 14 appeared to lift (v). (This appears to be unlikely from other considerations.)

2.5 Evidence was given with regard to certain structural features of the bridge. The tie-plates 302A over pier 14 between the south ends of the top chords of span 4 and the north ends of the top chords of span 5 were stated to have been connected with a number of bolts thought to be ten in number per plate (w). The conditions of the toggles connecting the bearing pins of spans 4 and 5 at pier 14 were described (x).

2.6 It is thought that the description given above represents the probable course of events before and during the collapse as far as it can be deduced from the evidence. Little reliance can be placed on the various estimates of time during the collapse, although the general impression that it was very short is undoubtedly correct. Further particulars of the collapse can be deduced from the condition of the bridge after the collapse, as described in the following section.

SECTION III—CONDITION OF STRUCTURE AFTER THE ACCIDENT

3.1 The structure after the accident has been examined in detail on behalf of the Commission by Mr. A. B. Sanderson, P.ENG., who has described its condition in his Report. Photographs of the structure and of structural details thought to be important have been taken and incorporated as exhibits before the Commission. The Advisers inspected the bridge between 28 July and 7 August. All the above took place when the collapsed structure was in the condition it assumed immediately after the collapse. Salvaged material has been examined by the Advisers in the period immediately preceding the date of this Report.

3.2 Mr. Sanderson's Report and the photographs give particulars of the collapsed structure in considerable detail which were confirmed by the inspections of the Advisers and which will not be repeated here. Features requiring particular note in determining the progress of the collapse in more detail than could have been observed by eyewitnesses are listed below:—

(a) The foundations of pier 14 and of falsework N4 are practically undamaged. They have not settled or moved appreciably since before the accident.

(b) The lower beams of the N4 grillages have been crushed from above, and the girders driven outwards from the longitudinal centre lines of the pile groups.

(c) The upper beams (stringers) of the N4 grillages have collapsed northwards, the upper flanges being approximately at the level of the bottom flanges, but north of them. The ends of the two outer stringers have torn away from the vertical double-angle members which connected them to the web stiffeners of the lower beams, and the connection bolts have disappeared.

(d) The two columns forming the legs of N4 have
both buckled about 23 feet above their base plates, and the bottom ends are turned up south of the upper parts. The remainder of the lengths of the columns are straight. The top end of the west leg is disconnected from its bearing on the bottom chord of the span and displaced northwards.

(e) Pier 14 has been pulled over southwards, the top now being approximately 6'-10" south of the bottom. The pulling-over of the pier has caused the concrete at the base of the pier shafts to fail in tension and the reinforcement to suffer bond failure at the splice above the pier pedestal.

(f) The north face of pier 14 is scored near the top in positions corresponding to the ends of the span 4 trusses.

(g) The pins at LO of span 5 have been pulled approximately 3'-9" southwards from their proper positions relative to the present position of pier 14, and the top ends of the tie-down links are bent southwards correspondingly.

(h) The northern base-plates of the span 5 bearings have been displaced approximately one inch north, and the southern base-plates approximately 3'-6" south, of their proper positions relative to the top of pier 14. The southern base-plate on the west side has been driven into the concrete, and the concrete below it has sheared on a steep diagonal plane passing downwards to the southern face of the top of the pier.

(i) The south ends of the distributing beams of the span 5 bearings are displaced about 8" north of their proper position relative to the southern base-plates, having kicked off the knife-edges on the base-plates.

(j) The "soft blocking" assemblies placed between the tie-down links and the opposing sides of the base-plates, consisting of 4" x 4" WF beams, steel packings and 4" timbers, have been crushed on both the north and south faces of the links (but more severely on the south).

(k) The condition of the toggle assemblies originally joining the bearing pins of spans 4 and 5, is described and photographed.

(l) Tie-plates 302A, which originally joined the ends of the top chords of span 5 and span 4 are still attached to the UO pins of span 5. The projecting web plates at U8 of span 4, to which the tie-plates had been bolted, have been driven inwards to the north.

(m) The south end floor beam of span 4 has been torn upwards away from its bottom flange, at its eastern end. The upper cover plate of the top chord below this beam has been torn from the rest of the member and bent upwards.

(n) Conditions in the tie-down chambers are described and photographed.

(o) The anchor bolts of the bearings on pier 13 at the north end of span 4 have been sheared off, except for the southwestern bolts in both bearings which are bent over northwards. These bolts show nicks on their north faces just above the concrete.

(p) The south ends of the bottom chords of span 3 and the north ends of the bottom chords of span 4 at pier 13 have been damaged in a way which proves that during the collapse span 4 was driven northwards into contact with span 3. This damage also indicates that when this contact occurred span 4 had dropped to a level which would bring the south ends of its top chords into contact with the top of pier 14.

(q) The severe and confused structural damage at the south end of span 4 is consistent with its south-west corner having hit, and its south-east corner having missed, the north edge of the underwater foundation of pier 14.

(r) Similarly, the havoc at the south end of span 5 and the damage exhibited by traveller No. 1 are consistent with free fall of the span and subsequent stoppage by contact with the grillages and pile nests of falsework N5 and with the harbour bottom.

(s) There are several noteworthy structural compression failures in span 5, namely diagonal LO-U1 west, which has been crippled near its top end; top chord U3-U4, which has been similarly damaged; and the top laterals, the NW-SE members of which have buckled in compression.

(t) The upper and lower grillage beams at N5 are severely damaged, the upper beams having been crushed over to the south, in a manner consistent with slanting contact of the extreme south end of span 5 during its fall.

(u) The western web splice plate in the L3 west joint span 5 was not in place. This plate had been removed prior to the accident. Its absence had no relevance to the collapse.

3.3 It is believed that all the features of the structure concerned in the collapse have been covered in the eyewitness accounts, in the Advisers' examination of the structure subsequent to the collapse, and in the detailed scrutiny and field reports made thereon by Mr. Sanderson and his staff.

SECTION IV—DISCUSSION OF THE POSSIBLE CAUSES OF COLLAPSE

Sequence of Collapse

4.1 It is known that two spans, Nos. 4 and 5, were brought down and that pier 14 was tilted to the south. Before starting to search for the cause of the mishap
it is necessary to be sure of the sequence in which these three main events occurred.

4.2 The weight of eyewitness testimony suggests very strongly, with no real dissenting evidence, that span 5 dropped first, then pier 14 tilted, then span 4 dropped, in that order.

4.3 Our examination of the visible parts of the two spans and of the pier, and of the drawings constituting their design, provides no indication of anything weak or faulty about these permanent parts of the bridge which could have initiated the collapse.

4.4 Within the field of the known circumstances, only one event could possibly have caused the fracture and tilting of pier 14, namely downward rotation of span 5, about its tie-down pins at the top of the pier, towards the inclined position which it assumed after the collapse. Such rotation would produce a very large horizontal force acting southwards at those pins; and the character and position of the shaft fracture is consistent with such a force. Thus there is the strongest technical evidence that the tilting of pier 14 followed, and did not precede, the drop of span 5.

4.5 Span 4 was structurally complete and on its permanent bearings. The stresses in its members were much less than they were designed to carry, due to the absence of the concrete deck slab and traffic load; and many of these members had already been subjected to higher stresses during the cantilever process of erection. The bearings of span 4 on pier 14, being roller bearings, were incapable of transmitting any significant horizontal force to the pier. (Similarly, they could not allow span 4 to help to any degree to prevent horizontal movement of the top of the pier.) There is in fact nothing whatever to suggest that span 4, by virtue of any structural weakness or instability, fell first or initiated the collapse of span 5. In making this assertion, it must be remembered that some of the members of the south end of this span are still underwater and cannot yet be inspected. The span could conceivably have been brought down by structural failure of some of these members. But for the span to have assumed the collapsed position it did assume, structural failure would have had to occur simultaneously at virtually identical points in both the east and the west truss. If this had happened, it is most unlikely that it would have produced damage at the top of pier 13 consistent with what actually occurred there. And even if it had happened, the fall of span 4 would not of itself have brought about the fracture and tilting of pier 14. Finally, to accept a theory that span 4 collapsed first would be to reject outright the weight of eyewitness testimony.

4.6 Could the two spans have been brought down by two separate and distinct causes which operated, one on one span and the other on the other, almost simultaneously? Such a hypothesis would be stretching probabilities beyond all reasonable bounds; and moreover there is no supporting evidence for so unlikely a combination of circumstances.

4.7 It can therefore be confidently concluded that the sequence of motion of the two spans and the pier was as stated in 4.2 above, namely span 5—pier 14—span 4.

Possible Causes of Collapse

4.8 Evidence has been given that no earthquake, high winds, collision or similar disturbance occurred on the day of the collapse. The velocity and direction of the tide were normal.

4.9 The system of support of span 5 will now be closely examined, for all evidence available so far points to a weakness somewhere in that system as the cause of the disaster.

4.10 At the time of the collapse, eight of the ten panels of span 5 had been erected (save for the west bottom chord and the bracing members of the eighth panel) to a length of 345 feet southwards from pier 14. Apart from its own weight, this structure carried the travelling erection crane (in the seventh panel), the railway track for delivering members to the crane for erection, and various temporary walkways, scaffolding and other light equipment. The weight of the permanent structure was much less than it will be when the bridge is completed, owing to the absence of the concrete roadway deck. The weight of the structure and everything on it at the time was 4149 kips. Its centre of gravity was approximately 192 feet horizontally to the south of pier 14.

4.11 This weight was supported by vertical reactions at the two permanent roller bearings and anchorages at panel point LO on the top of pier 14, and at the two columns of falsework N4, located under panel point L4, 168 feet south of pier 14. Owing to the fact that the centre of gravity of the span was 24 feet south of panel point 4, the vertical reaction at pier 14, that is to say the vertical force applied by the span to the pier, was an upward force (uplift) of 604 kips, and the vertical reaction at bent N4 was a downward force of 4753 kips. Owing to the disposition of the traveller and of the erection train (loco, cars and member BC8W) these reactions were not quite equally shared by the two shafts of pier 14 and the two columns of N4.

4.12 The reactions would be modified hardly at all by the effect of the estimated 15 MPH wind blowing at the time. At this speed the wind pressure would be only about 2 per cent of the 30 pounds per square foot allowed for in the erection calculations. Wind forces are therefore ignored in computing the reactions and stresses at the time of the accident.

4.13 The total vertical forces present at the time in the N4 columns including the weight of the columns themselves, were:

- East leg — 2282 kips
- West leg — 2699 kips

4.14 The span had also to be stabilized and held in position longitudinally. Tie-plates 302A were provided for this purpose. They formed rigid connections be-
tween the opposing ends of spans 4 and 5. Span 4 had fixed bearings at its north end on pier 13. Any horizontal forces on span 5 in the north-south sense would be transmitted via the tie-plates through span 4 to those fixed bearings and absorbed by pier 13.

4.15 Horizontal forces in this sense on span 5 could arise from four sources, viz:

(a) Wind blowing in any direction other than truly right angles to the line of the bridge.
(b) Acceleration or braking of the erection train when moving along the span.
(c) Slewing motion of the traveller boom.
(d) Expansion and contraction of the structure due to change of temperature, causing the columns of Bent N4 to be moved out of plumb.
(e) Reactions corresponding to compressive forces in the soft blocking, described in paragraph 3.2 (j) above. Such compressive forces would result from temperature expansion and contraction of span 4 because, on account of the tie-plate connection 302A, the temperature movements would be transmitted to the north end of span 5 and thence, through the top ends of the tie-down links, to the blocking. The blocking would be squeezed, thus offering some resistance to the temperature movement. The amount of the resistance, and therefore of the force in the tie-plates, would be roughly proportional to the amount of the movement. The arrangement of the blocking was such that, for the small range of minimum and maximum temperatures indicated by the relevant meteorological records, and the proximity of this range to the design mean temperature of 60°F., the squeezing forces would be insignificant.

As explained in paragraph 4.12, item (a) (wind) was negligible at the time. Eyewitness testimony indicates that items (b) and (c) were not operative. Items (d) and (e) would produce only minute forces, and the forces due to these two items would in any case be opposite to one another in direction and tend to cancel out.

The reactions on tie-plates 302A were therefore virtually zero at the time of the accident.

4.16 We now consider the ability of the various units to resist the forces imposed upon them.

Firstly, the partly completed permanent structure of span 5 itself. The forces in the members and joints have been recomputed and found to be within safe limits in every case. This conclusion is supported by the appearance of the wreckage, which shows no sign either that any part was under-designed or that the material or construction of any part was defective or faulty.

Secondly, Pier 14. This was designed to deal with permanent forces many times larger than those it was called upon to resist by the erection scheme and had, in fact, been more heavily loaded during the erection of the first four panels of span 5. Our own examination, coupled with the Report of Phillips and Barratt, shows it to have been soundly constructed in accordance with the design.

Thirdly, the Tie-Plate connection. The weakest part of this connection was the group of 10 bolts fastening each tie plate to the end of Span 4. There were four such groups altogether, giving a total safe load of the order of 420 kips.

Forces in the tie-plates would be transmitted through Span 4 to the fixed bearings on Pier 13. The condition of fastening of those bearings was such that they were in fact capable of about 2° of north or south movement if they were subjected to longitudinal forces sufficient to overcome the frictional resistance between them and the pier top. We estimate this frictional resistance conservatively at 180 kips (90 kips per bearing).

As shown in paragraph 4.15, there was virtually no force on the tie-plates at the time.

Fourthly, Falsework N4. This temporary structure has been fully described elsewhere. It consisted of the two columns and their interconnecting bracing, the upper tier of grillage beams (stringers), the lower tier of grillage beams, and the two pile nests.

4.17 In the following discussion, when comparisons are drawn between actual stress (immediately before collapse) and permissible stress, figures quoted for permissible stress are the appropriate figures, recommended by the AASHO Standard Specification for Highway Bridges, 1953 edition, increased by 25% for erection stress, including full erection wind stress, as adopted by Dominion Bridge in their calculations. The 1953 edition of the AASHO Specification does not mention any such increase of permissible stress under erection conditions nor does the Contract Specification, but an increase of up to 30% is normal practice for maximum combined loading conditions.

4.18 The actual compressive stress in the more heavily loaded west column of N4 was 16.7 ksi compared with a permissible stress of 24.9 ksi. Stresses in the diagonal bracing between the two columns were negligible. The likelihood that failure of these columns initiated the collapse can therefore be ruled out.

4.19 The pile nests do not appear to have contributed in any way to the collapse. The Contractor's records of pile driving indicate that the loading on the piles, even allowing for some maldistribution, was well within their safe load capacity. This is borne out by the report of Paul M. Cook, P.ENG., (Exhibit No. 101) and reinforced by the virtually undamaged and undistorted condition of the pile nests after the collapse. The severe local damage sustained by the inner lower grillage beams is conclusive proof that the inner piles of each nest, and especially the four central piles, received during the first stage of the collapse a rapidly-applied load of the order of two to three times the load they were carrying before the collapse. Yet the tops of the inner piles have not been driven down below the outer piles by any significant amount. The possibility that the piles caused the collapse can be dismissed.
4.20 The actual stresses in the lower grillage beams were everywhere within the permissible stresses. The damage to these beams is only consistent with severe abnormal loading having occurred after the collapse started. There is no indication of weakness in the material, or error in design or in fabrication, that would account for this damage.

4.21 The next consideration is the upper grillage beams or stringers; all the evidence, the state of the wreckage generally, reconstruction of the process of collapse from eyewitness statements, and the present appearance of the beams in particular, focusses attention on these beams as the most probable cause of the collapse.

4.22 There is no indication of defective or substandard material. On the contrary, the manner in which these beams have folded over each other without cracking (except in a few instances of very severe edge loading) shows the steel to be of satisfactory quality. Measurements of the undamaged portions indicate reasonably accurate rolling and fabrication.

4.23 The design of this group of beams, used as a grillage, was faulty. Using normal methods of computation, and remembering that the real stresses may well have been larger due to maldistribution, actual and permissible stresses for the load in the west leg of the bent are compared as follows:
(a) Actual tensile and compressive stresses in the flanges were 19.7 ksi compared with 22.5 ksi permissible.
(b) Actual maximum shear stress in webs was 14.4 ksi compared with 10.3 ksi permissible for webs of rolled beams without stiffeners.
(c) Actual web bearing stress was 22.7 ksi compared with 24 ksi permissible (not including erection allowance).
(d) Actual buckling stress in the webs at mid-height of beam under the column base-plates was 14.2 ksi compared with a permissible stress of 15.7 ksi, assuming the top flanges were properly restrained laterally (i.e. at right angles to their length) and including the 25% erection allowance. The loaded length of web is taken in this case on a conventional basis as twice the beam depth, i.e. as 72 inches (some specifications allow only 60 inches in this case).
(e) Actual maximum equivalent stress in the webs just below the edge of the column base-plate, calculated as specified in British Standard 153 was 29.3 ksi compared with the permissible of 31.3 ksi allowed by that specification.

4.24 Even though there was virtually no wind at the time of the collapse the beams were overstressed in shear, and would have been even more overstressed if there had been a high wind as is specified when the 25% overstress is used. Stiffeners should therefore have been provided. Furthermore, the quoted permissible limit for web buckling stress is subject to the provision of effective lateral restraint for the top flanges; but this restraint was not provided.

4.25 The customary method of providing lateral restraint in a grillage of this kind is to set rigid transverse diaphragms between the beams and connect the diaphragms rigidly to the beam webs or to stiffeners attached to them. Sound practice dictates that such diaphragms be placed at the most highly stressed points of the beams (in this case within the 3'-6" loaded length corresponding to the width of the column base-plate) and, in order to maintain straightness of the compression flanges, at the outer reaction points. In this grillage no diaphragms conforming to the above description were provided. Instead, four sets of wooden blocks were inserted between the beam webs and clamped there by external clamping bolts. Two sets were located near the ends of the beams (outside the loaded length referred to above), and the other two some 20 feet in from the ends, also well away from the highly stressed part of the beam. These blocks were not positively secured to the beams, nor were they made to fit tightly at top and bottom between the flanges. Friction between the blocks and beam webs was all that could render the blocks effective as diaphragms. Calculations indicate that such friction force as might have been developed would have been inadequate. There is some likelihood that the blocks may have shrank in position and thus relaxed the clamping force. Considering all the circumstances there is a distinct possibility that shrinkage occurred; but even if it did not, it is very doubtful whether the blocks, badly located and insecurely attached as they were, made any significant contribution to the stability of the upper tier of the grillage.

At this point it may also be recorded that there are inconsistencies between the centre-to-centre distances of the stringers and the thicknesses of the blocks, as detailed on the Contractor's working drawings, and between those dimensions and the measured thicknesses of unbroken blocks recovered from the wreckage. These inconsistencies indicate errors in checking and also lend support to the shrinkage theory mentioned above.

4.26 The stability of the upper grillage beams must therefore be considered in greater detail. Assuming that each of the four beams received one quarter of the total column load, because of the compressibility of the plywood packings, for the west column this was 675 kips per beam. Since the beams had no vertical stiffeners anywhere near the location of the column base plate or near the tops of the lower beams on which they stood, the whole of this load on each beam had to be transmitted downwards by its web which was 0.653 inches thick. The load entered the web over a length of about 46 inches under the column base and left it via four lengths each of about 16 inches; these four lengths were centred 24 inches and 60 inches each side of the centre of the column base. The rigidity of the pile nests and the lower grillage beam assembly,
coupled with the lateral rigidity of the bottom flanges of the upper beams themselves, would hold the lower edges of the beam webs firmly in position in space and in direction. The top flanges of the beams were not effectively restrained in space against lateral displacement by any external agency, although the base plates of the columns would maintain the upper edges of the webs in vertical planes. The degree of effectiveness of the double angle members at the ends of the two outer beams was negligible. Considered as struts, the webs must therefore be taken as restrained in position and direction at the bottom, and restrained in direction only at the top. Structurally, this is equivalent to a strut having a length equal to the full depth of the web, hinged top and bottom. The slenderness ratio of the webs considered as struts in this manner is 171, which is higher than is normally permitted for structural members carrying main loads, the limit in the AASHO specifications being 120. Even if the plywood packings were sufficiently incompressible (which they were not—see 4.27 below) to prevent any twisting of the flanges, the failure stress of a perfect strut of this slenderness ratio is 10 ksi, which would act on approximately the full length of the web over the bottom tier grillage, namely 11 feet. The failure load would therefore be 857 kips, compared with the actual load of 675 kips. The theoretical failure load would tend to be reduced by any eccentricities or inequalities of loading and by normal dimensional inaccuracies in the beams themselves; but a major cause of reduction in the failure load is the presence of the plywood packings above and below the beams coupled with the absence of stiffeners and of effective diaphragms.

4.27 The plywood packings may be thought of as extensions of the webs, and if the "stiffness" of the plywood were the same as that of the webs, the effective depth of the web considered as a strut would be increased by the depth of the two packings. By "stiffness" in this connection is meant the product of Modulus of Elasticity (E) and Moment of Inertia (I) of the webs and packings respectively. In general, these stiffnesses would not be equal and the effective depth would be increased by the depth of the two packings multiplied by the stiffness of the webs and divided by the stiffness of the packings. The stiffness of the packings is proportional to the modulus of elasticity of the plywood under the conditions of loading. Professor Hrennikoff has carried out tests to determine the plywood modulus and has demonstrated that under the intensity of load obtaining at the time of the collapse (1.34 ksi) the modulus is very low. Further he has shown that the full compression of the plywood does not develop immediately on the application of an increment of load; an initial compression first takes place and then further compression ("creep") develops under the same load. The stiffness of the packing would therefore decrease as time passed; the effective depth of the combined beam and packing unit, as a strut, would increase, and the failure load would correspondingly decrease. Using the results of Professor Hrennikoff's tests, it is found that the calculated failure load would be about 785 kips immediately after chord BC8W was brought on to the span and about 670 kips 30 minutes later. Again, these failure loads relate to a perfect strut and would be further reduced in practice by the imperfections mentioned before.

In this connection it is noted that measurements were recorded at the site (see Exhibit 111) of the level of the N4 column base-plates, and that these levels indicate continued settlement of the base-plate during the night preceding the accident. The most probable explanation of this is that the plywood was "creeping" as described above.

4.28 The reason for the non-provision of stiffeners in the beams is to be found in the only calculation sheet for these beams available to us. This sheet is exhibited before the Commission by Dominion Bridge Co. in their calculation file No. 2 (Exhibit 117). The sheet is unnumbered but bears the title "design of caps and distributing beams using 36WF160 beams between pairs of bents" and is dated 29 June 1957. Under the heading "CHECK SHEAR," to determine the shear stress the area has been taken as the gross area of the whole beam, including flanges and webs (47.09 sq. in.) instead of the gross area of the webs (23.5 sq. in.), as is required by accepted elastic theory and by the AASHO and all other design specifications. The shear stress has therefore been wrongly calculated to be 6 ksi instead of 12 ksi. If 6 ksi had actually been the shear stress, it might have justified the use of the adopted beams without stiffening, as is the conclusion recorded on the calculation sheet; but this would not have been the case if the correct shear stress had been calculated. Stiffeners and diaphragms would then have been provided, and the accident would not have occurred. There is a second error on the same sheet, under the heading "CHECK FOR WEB STIFFENERS," in which the flange thickness (1") has been used instead of the web thickness (0.653"); the higher stresses properly calculated would have called attention to the high general stressing of these beams even though they would still have been permissible in this case.

4.29 Mention should be made of two other factors which may have had some effect in reducing the failure load. In most rolled steel beams there are "locked-up" stresses in the roots of the webs, and these tend to reduce the effective limit of proportionality when deflections under high stress fall to be considered, and this may have been the case here. Further, a calculation of the lateral bending stresses in the unstiffened flanges acting as cantilevers under the load shows these stresses to be very high indeed. Professor Hrennikoff has also dealt with this aspect in his Report.

4.30 Our conclusion is therefore that the primary cause of the accident is elastic instability of the webs of the stringer beams of the N4 grillage, accentuated by the plywood packings above and below the beams. The
instability was due to the omission of stiffeners and effective diaphragming in the grillage, and this in turn was basically due to an error in the calculations. Such diaphragming as was provided was inadequate.

SECTION V—THEORETICAL RECONSTRUCTION OF THE PROGRESS OF THE ACCIDENT BASED ON ASSUMED INITIAL FAILURE OF THE TOP TIER GRILLAGE AT BENT N4 WEST

5.1 Dynamic calculations have been made in an attempt to reconstruct the progress of the accident, based on the assumed initial failure of the top tier grillage at Bent N4. The narrative is divided into sections corresponding to the main stages of the collapse. Nine sketches showing some of the more important stages, and a timing diagram of the fall, are attached.

The main stages are as follows:
1. Up to the end of the crippling of the webs of the top tier grillage, divided into the following sub-stages:
   A. the initial free fall of span 5 during the early stages of the crippling;
   B. the locking of the span 5 expansion bearing and the breaking of the bolts at UO;
   C. the later stages of crippling of the webs.
2. Crippling of the columns of N4.
3. The second stage in the fall of span 5 after the hesitation, in the following sub-stages:
   A. LO bearings at north end of their travel;
   B. OL bearings moving south until they reached the south end of their travel;
   C. anchor links bending south and the south base-plates being pushed south, finishing with the breaking of pier 14;
   D. the final fall with pier 14 moving over to the south. The span 4 support was removed during this stage;
   E. effects in the anchorage tie-down chambers of pier 14.
4. The collapse of span 4, including damage to the end of span 3, divided as follows:
   A. the initial fall and northward motion of the span as its south support was removed by the tilting of pier 14;
   B. further fall to the point where the upper south end of the span grazed the north face of the tilted pier 14;
   C. remainder of the fall.

The calculations are necessarily approximate. Certain essential data, which would be needed for exact analysis, are not available. Nevertheless the results obtained do show that the progress of the collapse can be expected to have accorded with the evidence of the eyewitnesses and with the observed post-collapse condition of the structure.

5.2 Stage IA

The initial failure is taken to have been in the webs of the top tier of the N4 grillage. This may have started under the west column, because the west column loading was the greater and the top of the west column was at least \( \frac{3}{4} \)" south of the vertical centre line at the bottom of the columns. The base-plate and the stringer top flanges then started to move north. The load in the west column became small almost immediately, and the resulting transfer of load to the east column then caused similar collapse of the east top tier grillage.

The columns moved downwards owing to the removal of their support, and the base-plates moved north as they were clipped to the top tier grillage beam top flanges.

Span 5 then began to move downwards at N4. It remained supported on its expansion bearings at pier 14, the load on which was changed from an upward force of about 300 kips per truss before the collapse to a downward force of 500 kips per truss during the fall of span 5. The bearing at this time was under a pre-tension of approximately 500 kips per truss in the tie-down link, the total load on each bearing being about 1000 kips during the fall.

Panel points UO of span 5 were at this stage connected by pins to the tie plates which were themselves connected to panel points U8 of span 4. Span 5 was at first constrained to rotate about the UO pin. During this initial movement, the combined centre of gravity of span 5 and Traveller No. 1 (tied down to it), which was about 190 feet south of the UO-LO vertical, accelerated at about three quarters of gravitational acceleration, i.e. at about 24 feet per second per second. Points at 253 feet south of the UO-LO vertical moved with acceleration equal to that of gravity (32 feet per second per second). Therefore any men or materials, not held down to the span, located more than 43 feet south of panel points 5 would fall more slowly than the span itself. This happened to certain of the eyewitnesses and doubtless also to chord BC8W and the outer bogie on which it was supported.

During this initial fall, panel points LO of span 5 moved northward over the expansion rollers, taking with them the top ends of the tie-down links. They, in turn, crushed the soft blocking. No large force would be required to crush the blocking, and it is thought that it had no real influence on the subsequent course of events.

The top chord of span 5 before the collapse sloped upwards to the south at \( 3^\circ \). At the end of Stage 1B it had rotated through about \( 1^\circ \) to about \( 2^\circ \) upwards.

Stage 1A lasted about 0.4 seconds.

5.5 Stage 1B

When panel points LO of span 5 had moved approximately \( 6^\circ \) north, the expansion bearing rollers "locked" as the parallel faces of adjacent rollers came into contact with one another. The diagonally opposite corners of the bearings were then in line contact with the bearing plates above and below, and the downward force then existing of about 1000 kips drove the corners of the rollers into the plates, thus completing the locking action.
Span 5 was then forced to rotate about the bottom chord pins below LO; and the continuing fall of the span thus generated a large horizontal impulse on the tie-plates connecting span 5 to span 4, operating against the reaction of the locked expansion bearings and the inertia of span 4.

The 1" high strength bolts were in clearance holes in the tie plates. The holes in the tie plates had been burnt out with jagged and very hard edges. The locking of the expansion bearings under the vertical load would have provided the horizontal reaction needed at LO to account for the failure of the UO tie-plate bolts. Markings on the tie-plates confirm that they were being dragged horizontally southwards at the time of the failure of these bolts.

The forces in the tie-plates, due to the crushing of the blocking and the locking of the expansion bearings, tended to pull span 4 southwards, and there is evidence of this in the nicking of the north faces of the anchor bolts of the bearings on pier 13. The breaking of the bolts could not materially affect the fall of the span.

The breaking of the bolts would cause a very loud cannon-like noise within half a second after the beginning of the collapse.

5.4 Stage 1C
Stage 1B took place when the top flanges of the top tier grillage beams at N4 had moved about 2 feet downwards and about the same distance northwards. After the breaking of the tie-plate bolts, span 5 continued to fall and the top tier grillage beams to collapse until the beams were flattened. At that stage the steel-to-steel contact of adjacent webs, particularly the more southerly webs, crushed as they were between the base plate and the upper flanges of the bottom tier of the grillage, then began to exert a large upward force on the column legs. The almost free fall of span 5 was thus arrested, and the span appeared to hesitate in its fall.

This may be considered to end the first stage of the collapse. Up to this point there had been no serious physical damage to any part of the structure except the top tier grillage beams of N4, the bolts in the UO tie-plates and the soft blocking at the tie-down links. Bottom chord BC8W and its outer bogie, the traveller boom and men working near the south end fell freely, and more slowly, than the span below them. The boom of the traveller with the chord hanging from its hook would be expected to be swinging southwards as Stage 1 ended. This would explain the evidence of Mr. and Mrs. Phillips.

5.5 Stage 2
The reaction force under the base plates of the N4 columns increased rapidly as the velocity of the falling span was arrested. This force crushed the webs of the top tier grillage beams together, driving their bottom flanges like a wedge between the two pairs of beams of the bottom tier of the grillage and forcing them outwards from one another, breaking the bolts in the connecting diaphragms and tearing the welds holding the beams to the pile caps. In the case of the western pair of the western bottom cap beams, the wedging action was sufficient to drive the beams right off the pile cap and into the water. The outer ends of the top tier beams were turned upwards by the wedging action. During this stage the base plates of the N4 columns, and points L4 of span 5, moved downwards further distance of about 1/2 feet while the velocity was being retarded by the resistance force now created by the crushing of the grillage.

This force acted on the base plate about 1'-3" south of its centre line causing a large moment on the bottom of the column. The combination of the direct thrust, the moment and the horizontal dynamic force due to the arrest of the northward rotation of the columns caused them to cripple at a point about one-quarter of the distance up the column from the base plate. Calculation indicates that the force to cripple the column under these conditions was of the order of 3000 kips.

The force in the column then rapidly reduced to the small force necessary to continue the crippling, and span 5 began to fall almost freely again. The columns continued to bend with knee-action, the base plates rotating upwards about their north edges until they were lifted away from the upper grillage beams by the gussets of the bottom cross lateral of the bent contacting the ends of the inner lower grillage beams. Further movement then caused the gussets to slide northwards with little damage to the bottom tier top flange plates, and the folded-up bent to drop into the water. Before its final disengagement, the corner of the east column base plate was driven into the web of the west inner bottom tier beam and tore it. The collapse of the columns and the wedging outwards of the bottom tier beams account for all of the damage caused to these components.

It is difficult to estimate the length of time for Stage 2, but it was probably of the order of a tenth to a quarter of a second, and the total elapsed time from the start of the failure to the end of Stage 2 would not be more than about \( \frac{3}{4} \) second.

During Stage 2 the reaction at LO changed momentarily from a downward reaction to an upward reaction, but this would not have been sufficient to damage the tie-down links or to move the tie-down girders and wedges. The rollers under the expansion bearings of span 5 remained in the position they assumed during Stage 1B, but there would be little horizontal force holding them in that position.

Up to the end of Stage 2 there would be no real physical damage to either span 5 or span 4, or to pier 14.

5.6 Stage 3, General
During its further fall span 5 remained supported on the top of pier 14, and subsequent events are related to the dynamic forces arising from the fall acting on the span itself and on pier 14 through the anchor link and bearings. The vertical fall of the span was, to a close approximation, independent of the movement of the
top of the pier and of the LO panel points of the span, and to this approximation the centre of gravity of the span fell at about 24 feet per second per second. The amount of the horizontal force at the top of pier 14 could be calculated accurately if the centre of rotation of the span had remained fixed in space, but this was only true during the early part of the fall. After that the LO pins moved south over the expansion rollers, the anchor links were bent southwards, and the pier broke and tilted southwards. The time for this complex sequence of events can be estimated reasonably, and a fair approximation obtained for the horizontal movement of the various components at the top of pier 14 during the course of the fall. The forces vary considerably from those which would occur if the centre of rotation were fixed; nevertheless, calculations provide adequate proof that horizontal forces were developed large enough to fracture pier 14.

At the beginning of Stage 3 the level of the centre of gravity of span 5 was about 23 feet above the level of the LO pins; that is to say, an imaginary line through the centre of gravity and the LO pins sloped upwards to the south at an angle of 7° above the horizontal corresponding to an upward slope of the top chord of 2°. The elapsed time at the beginning of Stage 3 from the commencement of the accident was about \( \frac{1}{2} \) of a second.

5.7 Stage 3A

At the beginning of Stage 3A the horizontal force at the LO pins due to the rotary fall of span 5 acted in a northerly direction and held the expansion rollers at the northern end of their travel. This force decreased in amount as the fall continued and became zero when the span had fallen through about 2.3° in a time of \( \frac{1}{4} \) second since the beginning of Stage 3. By this time the centre of gravity of the span had acquired a southward horizontal velocity of 1.5 feet per second.

5.8 Stage 3B

The LO pins and the upper parts of the expansion bearings then began to move southwards over the rollers with the span maintaining the horizontal velocity reached at the end of Stage 3A, since during the forward motion of the bearings virtually no horizontal force could be exerted on the span. The total movement of the LO pins during this stage was about 11 inches and they reached the southern limit of their travel 1.6 seconds after the beginning of Stage 3. During Stage 3B the span fell through an angle of 7.7° and the top chord at the end of this stage sloped 8° downwards.

5.9 Stage 3C

After the bearings locked at the southern end of their travel, the horizontal force acting through the LO pins dragged the bearing distributing girders southwards over the pier and with them the south baseplates. The friction force between the base-plate and the top of the pier during the dragging forward is difficult to evaluate but it tended to reduce the horizontal motion of span 5 without having an appreciable effect on its vertical fall. The horizontal force was not at this time of sufficient magnitude to break the pier, but towards the end of Stage 3C the tie-down links were bent forward over the edges of the anchorage shafts. The bent-over top ends of the links then acted with the displaced bearing assemblies to form a roughly triangular "bracket." The formation of this bracket now arrested southward movement of LO relative to the pier top, causing a rapidly increasing horizontal force to be applied to the pier itself, which then became sufficient to break its shafts at the base. The approximate angle of the top chord when the pier broke was about 15° below the horizontal and the elapsed time since the beginning of Stage 3 about 2 seconds.

5.10 Stage 3D

After pier 14 had broken, it began to tilt southwards under the influence of the horizontal force which accelerated the pier top in a southerly direction. When the span had fallen to an angle of about 20° below the horizontal, the movement of the top of the pier was sufficient to remove the base plates from below the expansion bearings of span 4 and this span then began to fall, as described under Stage 4. This took place about 2.2 seconds after the beginning of Stage 3.

Span 5 continued to fall and the pier to pull over until the southern ends of its bottom chords drove into the river bed causing violent and heavy damage to the two end panels. Whilst its fall was being slowed down and finally stopped by the river bed, there was a large increase of force downwards on the bearings at pier 14, and the momentum of the pier was reduced to zero by the force operating through the locked bearings. At this stage the top of the pier had moved about 6'-2" south of its original position. In the earlier stages of the slowing down the main reaction was at first exerted through the southern end of the east truss which was triangulated in bay 7-8, whereas the western truss was not. This caused the span to twist anti-clockwise (looking north) at its south end, crippling the NW-SE diagonal members of the top lateral system. At the same time the reaction of the span at pier 14 was transferred mainly to the western bearing on the pier, crippling diagonal LO-U1 west, driving the south baseplate into the top of the west pier shaft and shearing the concrete there.

During the course of its fall the L8 panel points of span 5 drove through the northern edge of N5, damaging the lower tier grillage there; but this first impact probably caused only superficial damage to the south end of the span and could not significantly have altered the course or rate of its fall. Later, the top chord material at U8 hit the upper tier grillage at N5, crippling the webs of the beams southwards and doing other severe damage to the N5 foundation generally.

Span 5 came to rest with its top chord about 26° below the horizontal, the elapsed time from the beginning of Stage 3 being about 23\( \frac{1}{4} \) seconds.

5.11 Stage 3E
The final stages of the fall of span 5 caused certain effects in the anchorage chambers of pier 14. The downward reaction of the span lowered the LO pins relative to the top of the pier and at some late stage reduced the tension in the anchor links to zero. This released the load from the tie-down girders at the bottom of the chamber and these then swung on the twisted portion of the tie-down links as a pendulum. It would first appear to have swung to the north with considerable force, knocking the northernmost jack off the jacking girder when the side of the jack hit the wall. At this stage the wedges on the north end of the cross girder were pushed inwards to rest on the top flange of the girder. The pendulum then swung southwards more slowly, displacing the south jack northwards when it came into contact with the south wall. Finally, the pendulum swung gently northwards again, by which time the girder was tilting up to the north due to the out-of-balance weight of the jack at its south end. This final movement pushed some of the packing plates on to the top of the wedges, giving the peculiar effect of plates, which were originally below the wedges, finally resting on top of them.

The foregoing reconstruction of events relates in the main to the west anchorage chamber. The movement in the east chamber was much less in extent, being sufficient only to topple the northern jack off the jacking girder.

5.12 Stage 4A

As described in Stage 3D above, the support of span 4 on pier 14 was removed by the southward movement of the top of pier 14 about 2.4 seconds after the beginning of Stage 3. Span 4, which remained supported at its north end throughout its fall, then began to fall, rotating about the pier 13 bearings. These bearings, which had been pulled forward to the limit of the slotted holes during Stage 1B, were then driven northwards over the top of pier 13, probably tilting upwards at the same time, shearing off the northern anchor bolts, and bending the southern anchor bolts northwards. The southern end of the bottom chord of span 4 then came into contact with the north face of pier 14. The toggle arms which had been torn apart when span 5 bearings on pier 14 moved south, were turned upwards, and in the case of the western toggle, driven into the end batten plates of the lower chord. Relatively little force was required to cause this.

5.13 Stage 4B

Span 4 continued to fall until the top chords at the south end reached the level of the top of the pier, which took place about 1.3 seconds after the beginning of Stage 4, that is, about 3.7 seconds after the beginning of Stage 3, by which time span 5 and pier 14 would have stopped moving; span 4 at this stage would be sloping 3° downwards to the south. The distance between the LO pins of span 4 and the extreme southern end of its top chord was sufficient to bring the latter into slanting contact with pier 14, and the force and motion produced by this contact drove span 4 northwards and the top of pier 14 southwards, gouging out slots in the north face of the pier near the top and bending in the projecting ends of the U8 web plates. The further southward movement of the top of pier 14 produced at this stage drove the south base-plates from under the distributing girders of the bearings a distance of about 8 inches, accounting (with the 6°-2° movement mentioned in Stage 3D above) for the total observed tilt of 6°-10° of the pier.

The northward movement of span 4 brought the north ends of its bottom chords into contact with the corresponding south ends of the bottom chords of span 3, damaging both in a manner well shown by the photographs. The angle of span 4 when the spans came into contact is shown by the damage to the diaphragms of the chords which also indicates the limit of relative movement of the spans. The northern end of span 4 moved slightly east whilst it was coming into contact with span 3, and this was accentuated by the lateral forces produced during the contact.

5.14 Stage 4C

The rotation of span 4 about the top of pier 13 soon took the top chords at the south end out of contact with the pier and the span then continued its free fall. The horizontal force arising from the fall then moved the bearings southwards over the top of pier 13, shearing off the south-eastern anchor bolts in the process. The easterly movement of the north end of the span was sufficient for the base plates in their southward movement to miss the south-western bolts, which remained bent over to the north.

Finally, the fall of the span was arrested partly by the pedestal and partly by the base of pier 14, damaging the two end panels extensively.

The top chord of span 4 came to rest sloping down at an angle of about 30°, and the total time for Stage 4 was 3.2 seconds.

ACKNOWLEDGMENT

We record here our appreciation for the assistance and courtesy extended to us by Messrs. Sanderson, Hrennikoff, Armstrong, McLellan and Eldridge and their respective staffs, and by Mr. R. Wilson and the Commission Secretariat. Their efforts have greatly facilitated our work.

Respectfully submitted

by the Engineering Consultants and Advisers

F. M. Masters
J. R. H. Otter
J. R. Giese
R. Freeman

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ROCKER BEARINGS BECOME FREE TO MOVE SOUTH

TEMPORARY BENT BUCKLES TO NORTH

BEARINGS LOCKED TO SOUTH
ANCHOR TIES BEGIN TO BEND

TIME 14 SEC
0° 41' DEG

TIME 23 SEC
0° 30' DEG
NORTH

SPAN 3  SPAN 4

DATUM LINE  PIER 13

BEFORE COLLAPSE

ANCHOR SPAN

TIME: 00 SEC 0 - 8 DEG

PIER 14

TEMPORARY BENT N4

PILED FOUNDATION N5

MUD LINE

NORTH

SPAN 3  SPAN 4

U6 TIE BREAKS

ANCHOR SPAN

TIME: 05 SEC 0 - 7 DEG

PIER 14

TEMPORARY BENT N4

WEBs BUCKLE TO NORTH

ROCKER BEARINGS LOCKED TO NORTH

PILED FOUNDATION N5

MUD LINE
SPAN 4 FORCED OFF NORTH BEARING

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NORTH
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REPORT

SECOND NARROWS BRIDGE COLLAPSE

MODJESKI AND MASTERS
Consulting Engineers
SCHEDULE 9

(c) Modjeski and Masters Report dated September 30, 1958. Copy of Exhibit 319.

REPORT
SECOND NARROWS BRIDGE COLLAPSE

MODJESKI AND MASTERS
Consulting Engineers
September 30, 1958

John L. Farris, Esq., q.c.
Commission Counsel,
Royal Commission
Second Narrows Bridge Inquiry
Court House
Vancouver, B.C., Canada

Dear Sir:

In response to the request of the Honourable The Chief Justice Sherwood Lett, Commissioner, Royal Commission of Inquiry, Second Narrows Bridge Collapse, we submit herewith our report on the cause of the collapse, on June 17, 1958, of the north anchor span and adjacent approach span of the highway bridge being constructed over the Second Narrows of Burrard Inlet between Vancouver and North Vancouver, B.C.

The report has been prepared from field data furnished by Mr. A. B. Sanderson, P.ENG.; from information obtained from copies of Exhibits and Testimony assembled by the Commission; and subsequent to a visit to the site by J. R. Giese, Partner of Modjeski and Masters, during the period July 28 to August 2, 1958 and a later visit by both J. R. Giese and the writer. All calculations have been made and checked by the members of the staff of Modjeski and Masters in accordance with the accepted procedure of a Professional Engineer's organization. All results and conclusions have been reviewed by and approved by the signers of this report.

It is our opinion that the collapse was caused by the failure of the upper tier of the grillage of Falsework Bent N4, due to critical stress conditions in the beams induced by the loads which existed at the time of collapse.

Respectfully submitted
MODJESKI AND MASTERS
Engineers

Dr. F. M. Masters
REPORT OF THE INVESTIGATION
AS TO THE CAUSE OF THE COLLAPSE
OF THE SECOND NARROWS BRIDGE

SECTION I—INTRODUCTION

The Royal Commission investigating the collapse of a highway bridge during its construction over the Second Narrows of Burrard Inlet, has requested engineering advice, with respect to the circumstances surrounding, leading to or having any causal connection with the collapse. Pursuant to this request we have examined the various exhibits assembled by the Commission and the transcripts of eyewitness testimony taken during the hearings July 21, 22, 23, 1958; made a general inspection of the site of the collapse; studied the plans and specifications for the design of the bridge; examined pertinent contractor’s erection drawings; and examined material which has been recovered from the wreckage at this time.

We have also reviewed reports of special investigations in connection with the inquiry, as follows:

1. A. B. Sanderson, P.Eng.—concerning the detail observations and measurements made of the wreckage and the assembly of all technical data in any way pertinent to the investigation of the collapse.

2. Robert N. McLellan, P.Eng.—covering the investigation of the 110 ton travelling crane used to erect the superstructure.

3. G. S. Eldridge & Co., Ltd.—covering the testing of the metal in the grillage beams, Bent N4.

4. W. M. Armstrong, Professor of Metallurgy—commenting on the Results of Mechanical and Metallographic Tests made by G. S. Eldridge & Co. Ltd. and reporting on investigation of tie plates 302A and their bolts.

5. A. Hrennikoff, D.Sc.—covering the testing of plywood and a mathematical analysis of its use as cushions above and below the grillage stringers.

We have made pertinent independent computations and calculations based upon these data.

Facts and circumstances pertinent to the description of the project, the history and progress of the construction and the technical information with respect to conditions existing immediately prior to and subsequent to the collapse are covered in detail in Engineer Sanderson’s report. References to parts of the structure and locations of parts contained herein are in accordance with the nomenclature used by Mr. Sanderson.

SECTION II—FINDINGS AND CONCLUSIONS

A. Our general examination of the contract plans and specifications reveals no departure from sound engineering practice normal for the design of a bridge structure of this magnitude. The specifications and design criteria are quite similar to and for the most part identical with those used in the design of similar structures in the United States. Our detail examination of the design of Pier 14 reveals no inherent weakness. We believe that this pier could not have contributed to the collapse of the bridge, its present state of rupture having been a result of rather than a cause of the collapse.

B. We have made an analysis of the stresses in the various members of Span 5, based upon the computation of weights and loads furnished by A. B. Sanderson and Company, Ltd., under date of August 29, 1958, giving an estimate of the loads to which the span was subject immediately prior to its collapse. This analysis reveals that no condition of overstress existed in any structural member of the bridge itself due to the erection procedure, all critical members having been amply reinforced where needed.

C. Our review of the contractor’s erection drawings and shop details of pertinent parts of the structure indicates that the general methods of erection and the proposed equipment were adequate (with the exception of the beams comprising the upper tier of the grillage which was a component part of the temporary erection support known as Falsework Bent N4) to construct the bridge with a satisfactory rate of progress and to produce a structure in conformity with the contract plans and specifications.

Our computation of the stresses in the upper tier of grillage beams reveals that, at the time of the collapse, there existed a critical stress condition leaving no margin of safety to provide for permissible irregularities in the mill-produced beams. The lack of an ample safe load capacity and adequate means of providing lateral support and stiffeners for the beam flanges and webs permitted these beams to buckle in the webs, causing the destruction of the span.

SECTION III—DETAIL OF FINDINGS

In the following paragraphs our findings with respect to the various parts of the work are set forth in more detail.

A. FALSEWORK BENT N4

This falsework bent was located at Panel Point 4 of the north anchor arm of the main bridge structure. After landing the first four panels of superstructure on this bent by cantilevering from Pier 14 and the adjacent approach span, cantilever erection was to have continued for four additional panels, at which point the load was again to be relieved by landing upon Falsework Bent N5 under Panel Point 8 of the north anchor arm. During the course of this cantilevering and just prior to landing on Bent N5 the entire weight of the eight panels of anchor arm superstructure and the erection equipment was to have been supported by Bent N4.
1. Pile Foundations

An examination of the records of pile driving, types of piles provided, and the conditions of the pile groups after being subjected to high dynamic loadings during the collapse, reveals that these piles were carefully and accurately driven and were adequate to withstand the erection loads to be placed upon them.

2. Pile Frames

The groups of 24 individual piles forming the east and west foundations of the falsework bent were adequately braced to provide effective distribution of loads.

3. Lower Grillage Beams

The beams comprising the lower tier of the falsework grillage were heavy welded I sections, specially designed and fabricated for their purpose, and were adequate to transmit the estimated loads from the tops of the piles to the upper tier of the grillage with a satisfactory margin for eccentric overload.

4. Upper Grillage Beams

The upper grillage is comprised of four 36WF160, (Mark 129A) I-beam stringers placed parallel with the flanges nearly touching. These stringers run transverse to the bridge centerline and are supported by the four lower grillage beams under each leg of the bent, thus serving as a necessary horizontal tie between the two pile clusters as well as constituting the bearing area upon which the column legs of the bent rest. These beams were to be used in the permanent structure as stringers in the floor system of the cantilever arm of the main span, (refer to D.B.Co. Dwg. S3702/129) and were made of steel meeting A.S.T.M. Specification A7. Results of mill test reports for compliance of this material with the specification are included in the A. B. Sanderson and Company, Ltd. report. Similar results were obtained on samples cut from the beams used, as reported by G. S. Eldridge and Co. Ltd.

D.B. Co. Drawing S3703E/E13 indicates the position of these stringers in the falsework bent construction.

Normal safe design would have required that these beams, to serve as a grillage, should have had top and bottom bearing plates, structural steel diaphragms between the beams and web and top and bottom flange stiffeners. Because of an error, the design computations indicated that such stiffeners and diaphragms would not be required. In lieu thereof, inadequate timber blocking between the webs of the beams was inserted at four (4) points, two at about the third points of the beams where connection angles (for future diaphragm connections) were available and at two points each between the ends of the beam and the near edge of the column bearing plate. The timber blocking and the beams were clamped together at this point by means of a strongback angle and long bolts, over and under the beams. This arrangement provides the only restraint tending to hold the flanges of the beams in the proper transverse alignment, except immediately at the column bearing plate, where metal keepers were welded to pairs of the beams, thus providing a slight additional restraint close to the bearing area.

The extreme ends of the outer beams were bolted to extensions placed on the exterior stiffeners of the lower tier of beams to serve to position the upper tier and to provide a horizontal tie between the two pile clusters. These connections, which were made through the ends of the webs of the beams, would not be effective in restraining the flanges from transverse movement. Additional keeper plates were welded to the top flanges of the lower tier beams and to the underside of the upper tier beams along the exterior edges of the lower tier components.

No other restraints or supplementary stiffening plates were provided.

Using the maximum anticipated loads estimated by the contractor (D.B.Co. Dwg. 3703E/D4) consisting of erection stress, 30 pound wind, and locomotive and cars, to which has been added the weight of the bent. Locomotive (56 k at Panel Point 6) and 103 k Cars (10 k at Panel Point 7)

Load for Deflection Calculation 2580 k
Wind 30 lb. per sq. ft. on truss and traveller 982 k
Leg of Bent N4 114 k

3779 k

we calculated the anticipated stresses in the beams and obtained the following:

Bending —27,500 psi  
Web Crippling—31,700 psi  
Web Shear —20,100 psi

By any accepted design criteria, these stresses are much too near the minimum yield point of the steel, 33,000 psi., to be acceptable, even for temporary construction, and certainly not for such a critical point in the erection of a structure of this magnitude.

The Standard Specifications for Highway Bridges of the American Association of State Highway Officials (pg. 65, paragraph 1.6.8) requires that the calculated stress due to shear in the web of an unstiffened beam shall not exceed 75%, of 11,000 psi, or 8,050 psi. The American Institute of Steel Construction Specification for Allowable Loads on Beams states (paragraph titled “Crippling Value of Beam Webs,” page 172, 1957 Edition).

“Lack of proper lateral support for the top flanges of beams at the reaction point so decreases the crippling strength of the webs as to render such practice inadmissible.”

The long period of time elapsing from the installation of the timber blocking between beams in April to June 17 would have further reduced, by shrinkage of the timber, any effective lateral support furnished by this blocking, which in itself is of questionable effectiveness.

In order to evaluate as accurately as possible the actual stresses within these beams due to the loads which are estimated to have existed at the time of collapse, we have made calculations, taking into account the column bearing plate, where metal keepers were welded to pairs of the beams, thus providing a slight additional restraint close to the bearing area.

The extreme ends of the outer beams were bolted to extensions placed on the exterior stiffeners of the lower tier of beams to serve to position the upper tier and to provide a horizontal tie between the two pile clusters. These connections, which were made through the ends of the webs of the beams, would not be effective in restraining the flanges from transverse movement. Additional keeper plates were welded to the top flanges of the lower tier beams and to the underside of the upper tier beams along the exterior edges of the lower tier components.

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In order to evaluate as accurately as possible the actual stresses within these beams due to the loads which are estimated to have existed at the time of collapse, we have made calculations, taking into account
relative deflections of the piles and the beams, which indicate that stresses within the web material in certain localized areas immediately under the edges of the west column bearing plate and over the supports of the two interior lower grillage beams were in the range of 30,000 psi to 32,000 psi which is so near the yield point that the margin of safety, required by the specifications to protect against irregularities in both quality and dimension of the beams, did not exist. Both the Report of G. S. Eldridge & Co. and the Report of Professor A. Hrennikoff indicate that such irregularities did exist.

Under these conditions the inequality of load transfer due to the permissible variation in beam manufacture would cause yielding within the plastic range of the material which in turn would cause further eccentric loadings and result in collapse of these grillage beams by buckling in the webs of the beams.

The failure of the grillage is therefore attributable to the instability of the grillage stringers because of the lack of provision for lateral restraint of the top flanges of the beams and the lack of structural steel stiffeners which would effectively transfer the loads from the base of the column legs and distribute them to the foundation piles below.

The use of wood cushions to take up small variations in load distribution in falsework bents of this nature is not uncommon, but the use of plywood, both on top and immediately below the beams comprising this tier of the grillage, is highly questionable, particularly in view of the other lack of provision for transverse restraints. The loads on the plywood were of an intensity sufficient to produce plastic deformation and creep of the material as shown by Professor A. Hrennikoff's report. No lateral rigidity should have been anticipated to have been developed by this type of construction. The top and bottom flanges of the beams were susceptible to lateral movements which, under the conditions of critical stress, would produce eccentricities resulting in additional stress sufficient to cause collapse.

5. The Legs of the Bent

The legs of the bent consisted of two strengthened and braced members of the permanent structure. Our calculations indicate that the ultimate capacity of these legs was not less than 5600 kips per leg which is safely in excess of any static loads existing just prior to the collapse.

Upon failure of the grillage an eccentric load was placed upon the columns. In view of the large dynamic forces to which they must have been subjected in stopping downward movement of the span momentarily after its initial drop, failure by buckling occurred, possibly some twenty to thirty feet above the base of the column.

B. MOTIONS OF COLLAPSE

With the failure of the upper tier grillage beams under the west column of Falsework Bent N4 an immediate concentration of load was placed upon the east leg of the bent causing it also to begin to fail. During this initial failure the span moved down approximately 2½ to 3 feet at Panel Point 4 which resulted in downward movements at Panel Point 8 of five to six feet, which is in agreement with much of the eyewitness testimony.

Upon the release of support at Panel Point 4 there was an immediate reversal of the vertical loads at Pier 14 where the total vertical reaction of 604.4 kips tension changed to nearly 1000 kips compression. At the same time, a horizontal pulling was exerted upon tie plates (Mark 302A) tieing Span 5 to Span 4 at the ends of the top chords. This pull, together with rotation of Span 5 about the pins in these tie plates resulted in moving Span 4 southward so that the fixed shoes on Pier 13 exerted a force on the north sides of their anchor bolts, and the rollers of Span 5 on Pier 14 moved northward an amount sufficient to cause their locking and to cause the crushing of the 4-inch H-beams and "soft blocking" placed between the tiedown bars and the bearing plates of the bridge shoes bringing the toggles between the shoe pins on Spans 4 and 5 together an amount sufficient to damage the diaphragm and cover plate in Span 5 at this point.

The stopping of motion of the roller nest, the additional pressure exerted by the hold-down bars against the H-beams and the "knifing" of the toggles, together with the release of internal energy of Span 5, must have been sufficient to shear the bolts fastening Plate 302A to the top chord of Span 4. Although a total of 40 bolts had been placed in these four plates (10 in each), the conditions of the rough burned holes, slotted holes and the lack of any knowledge as to the degree to which these bolts had been tensioned, makes it very doubtful that anything approaching the full shearing capacity of all 40 bolts could possibly have been attained.

After the grillage folded over there existed a brief period where vertical support was again restored at Falsework Bent N4 through the columns coming in contact with crushed and nested beams, but this support was now eccentric to the columns, the span was in motion, adding its dynamic load, thereby exceeding the ultimate capacity of the columns which failed very quickly. Again both vertical and horizontal reactions on Pier 14 reversed, this time at much higher magnitudes with additional releases of internal energy of the span members.

It is possible that under these almost instantaneous reversals the wedging devices in the tiedown chambers were dislodged. Little definite information has been obtained as to the actual extent of the required pre-stressing in these chambers and the condition of the wedging devices.

After the failure of the columns the span continued downward with very little support provided at the falsework bent, causing an ever increasing pull southward on Pier 14 which, by the time the span had fallen
SCHEDULE 10
LIST OF EXHIBITS

Exhibit
2. The Honourable Chief Justice Sherwood Lett's Commission as Sole Commissioner.

DOCUMENTS FILED BY B.C. TOLL HIGHWAYS AND BRIDGES AUTHORITY, NUMBERED FROM 5 TO 99 INCLUSIVE

5. Contract No. 1 between B.C. Toll Highways and Bridges Authority and Peter Kiewit & Sons Co. and Raymond International Co. Ltd., January 9th, 1956.
6.— (1) U-4886—General Arrangement.
6.— (2) U-4887—Log of Drill Holes, Test Piles and Test Pits.
6.— (11) U-4896—North Approach Road—Profiles—Keith Road Intersection.
6.— (14) U-4899—North Approach Roads—Keith Road Intersection—Typical roadway Cross Sections.
6.— (17) U-4902—North Approach Viaduct—Bent No. 1—Concrete.
6.— (18) U-4903—North Approach Viaduct—Bents No. 2, 3, 4—Concrete.
6.— (19) U-4904—North Approach Viaduct—Bents No. 5, 6—Concrete.
6.— (20) U-4905—North Approach Viaduct—Bents No. 7, 8, 9—Concrete.
6.— (21) U-4906—North Approach Viaduct—Bents No. 10—Concrete.
6.— (22) U-4907—North Approach Viaduct—Bents No. 11—Concrete.
6.— (23) U-4908—North Approach Viaduct—Bents No. 12—Concrete.
6.— (26) U-4911—Cantilever Span—North Anchor Pier Concrete Outline.
6.— (27) U-4912—Cantilever Span—North Anchor Pier Footing and Concrete Reinforcing.
6.— (28) U-4913—Cantilever Span—North Anchor Pier Miscellaneous Metal.
6.— (29) U-4914—Cantilever Span—South Anchor Pier Concrete Outline—Sheet 1.
6.— (30) U-4915—Cantilever Span—South Anchor Pier Concrete Outline—Sheet 2.
6.— (31) U-4916—Cantilever Span—South Anchor Pier Concrete Outline—Sheet 3.
6.— (32) U-4917—Cantilever Span—South Anchor Pier Concrete Reinforcing Sheet 1.
6.— (33) U-4918—Cantilever Span—South Anchor Pier Reinforcing Sheet 2.
6.— (34) U-4919—Cantilever Span—North Main Pier Cofferdam and Concrete Outline.
6.— (35) U-4920—Cantilever Span—North Main Pier Concrete Reinforcing.
6.— (36) U-4921—Cantilever Span—South Main Pier Design Data and Concrete Outline.
6.— (37) U-4922—Cantilever Span—South Main Pier Concrete Reinforcing.
6.— (38) U-4923—Cantilever Span—South Main Pier False Piers, Caisson and Water Stops.
6.— (39) U-4924—Cantilever Span—Main Piers—Unwatering and ventilating.
7.— Contract No. 2 between B.C. Toll Highways and Bridges Authority and Dominion Bridge Co. Ltd., dated 7th Aug 1957.
8.— Approved Drawings for Second Narrows Bridge, on Contract No. 2 A.
8.— (1) General Plan and Elevation.
8.— (2) Plan, Elevation and Geometry.
8.— (3) Deck Framing Plan, Typical Section.
8.— (4) Stringers.
8.— (5) Floor Beams.
8.— (6) Reinforced Concrete Deck, Sheet 1.
8.— (7) Reinforced Concrete Deck, Sheet 2.
8.— (8) Deck Control Joints.
8.— (9) Deck Expansion Joint, Piers 10, 11, 12, 13.
8.— (10) Deck Expansion Joint, Piers 14, 17.
8.— (11) Deck Expansion Joint, Suspended Span.
8.— (12) Fence and Bracket.
8.— (13) Miscellaneous Drainage Details.
8.— (14) Anchor Span, Cross Section.
8.— (15) Anchor Span, Truss Members.
8.— (16) Anchor Span, Cross Sections.
8.— (17) Anchor Span, Truss End Bearing.
8.— (18) Suspended Span, Stress Sheet.
8.— (19) Suspended Span, Truss Members.
8.— (20) Suspended Span, Cross Section.
8.— (21) Stress Sheet, Anchor Span.
8.— (22) Stress Sheet, Cantilever Span.
8.— (23) Anchor and Cantilever Span, Lateral Bracing.
8.— (24) Anchor and Cantilever Span, Cross Bracing.
8.— (25) Anchor Span, Joint L-0 Details.
8.— (26) Anchor Span, Joint L-3.
8.— (27) Anchor Span, Joint U-2.
8.— (28) Anchor Span, Joint U-5.
8.— (29) Anchor Span, Joint L-5.
8.— (30) Main Span—Joint L-10.
8.— (31) Main Span—Joint U-10.
8.— (33) Cantilever Span—L18 and U18 North.
8.— (34) Cantilever Span—L18 and U18 South.
8.— (35) Main Pier Bearing.
8.— (36) Anchor Span, Out-Down and Bearing.
8.— (37) Wind Anchor, North Anchor Pier.
8.— (38) Wind Anchor, South Anchor Pier.
8.— (39) Wind Anchorage at Suspended Span.

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Exhibit
8.—(40) Main Span, Williot Diagram.

CORRESPONDENCE WITH B.C. TOLL HIGHWAYS AND BRIDGES AUTHORITY

SECTION 1
10. " No. 1061732— June 9th 1955
11. " No. 1065229— July 4th 1955
12. " No. 1066678— July 11th 1955
13. " No. 1067519— July 18th 1955 & enclosure (Rocanen report)

SECTION 2
15. " No. 1078162— Sept. 16, 1955

SECTION 3
17. Letter No. 1087957—dated Nov. 25, 1955 & Drawing 019—1
18. " No. 1089568 dated Dec. 8, 1955
19. cc from Mr. J. E. Moore, Dept. comptroller to Peter Kiewit, dated Dec. 13/55.

SECTION 4
22. " No. 1098891— Feb. 9, 1956
23. " No. 1100046— Feb. 17, 1956
24. cc from Mr. C. K. Saunders, Bridge Engr. to Swan, Wooster & Partners dated Feb. 27/56.
26. " No. 1102682— Mar. 6, 1956
27. " No. 1104992— Mar. 20, 1956

SECTION 5
28. Letter No. 1106916—dated April 2, 1956
29. " No. 1107621— April 6, 1956

SECTION 6
31. Letter No. 1124207—dated July 6, 1956
32. " No. 1126732— July 23, 1956
33. " No. 1126729— May 28, 1956
34. " No. 1130113— Aug. 8, 1956 (report)
35. " No. 1130348— Aug. 10, 1956 & enclosure
37. cc (no No.) dated Aug. 20, 1956 from Kiewit Raymond to San, Wooster.
40. " No. 1133762— Aug. 31, 1956
41. " No. 1133796— Aug. 24, 1956
42. " No. 1134842— Sept. 6, 1956
43. cc No. 1137323— Sept. 21, 1956 from Swan, Wooster & Partners to Kiewit Raymond
44. " No. 1139513— Oct. 4, 1956 & enclosure
45. " No. 1140668— Oct. 11, 1956 (report)

SECTION 7
47. Letter No. 1145118—dated Nov. 7, 1956 & enclosure
48. " No. 1145324— Nov. 7, 1956
49. " No. 1150184— Dec. 6, 1956 & enclosure
50. cc "Approach Roads, Tenders for sub-surface investigation" and "proposed Construction Schedule"

Exhibit
51. " No. 1151239—dated Dec. 12, 1956
52. " No. 1154352— Jan. 4, 1957 & enclosures
53. cc" No. 1154615— Jan. 7, 1957
54. cc" From Mr. J. E. Moore, Dept. Compt'r to Dominion Bridge, dated Jan. 23, 1957.
55. " No. 1159013—dated Feb. 4, 1957 & enclosures
56. cc" (no No.) — Jan. 7, 1957 from Swan, Wooster & Partners to Mr. Fisher.
57. " No. 1160188—dated Feb. 11, 1957

SECTION 8
58. Letter No. 1163845—dated Mar. 4, 1957 & note from Mr. Wilkins
59. " No. 1164900— Mar. 8, 1957 & enclosures
60. " No. 1164379— Mar. 6, 1957 from Swan, Wooster & Partners to Mr. Brown.
61. " No. 1167892—dated Mar. 25, 1957 & enclosures
63. " No. 1169404— Apr. 4, 1957 & enclosures
64. " No. 1169887— Apr. 5, 1957
65. " No. 1170454— Apr. 10, 1957
66. " No. 1170797— Apr. 12, 1957
67. " No. 1171580— Apr. 16, 1957
68. " No. 1175458— May 7, 1957 & enclosures
69. " "Federal Insurance Company Contract Bond" (no No.) Bond No. 798 3970
70. " No. 1182215—dated June 10, 1957 & enclosures.

SECTION 9
73. " No. 1191931— July 30, 1957
74. " No. 1192726— Aug. 6, 1957 & enclosures.
75. " "Toronto General Insurance Company Contract Bond" dated Aug. 7, 1957
76. " No. 1197392—dated Sept. 4, 1957
77. " No. 1197393— Sept. 4, 1957 & enclosures
78. " No. 1198683— Sept. 11, 1957
79. " No. 1199218— Sept. 16, 1957

SECTION 10
80. Letter No. 1206958—dated Oct. 31, 1957
82. " No. 1210693— Nov. 22, 1957
83. Letter No. 1211000—dated Nov. 25, 1957
84. " No. 1211612— Nov. 28, 1957 & enclosures.
86. cc from Mr. F. T. Brown, to Swan, Wooster & Partners, dated Dec. 4, 1957
88. " No..1213619— Dec. 10, 1957

SECTION 11
91. cc" "Rayint No. 23"—Kiewit Raymond, Second Narrows Bridge, N. Vanc.
92. cc" (no No.) Letter from Eakins Const. to B.C. Toll Highways and Bridges Authority dated June 25, 1957.
93. cc" (no No.) Letter from Eakins Const. to B.C. Toll Highways and Bridges Authority dated Feb. 6, 1958.

SECTION 12
96. " No. 1231904— Apr. 16, 1958
DOCUMENTS FILED BY DOMINION BRIDGE CO.

EXHIBIT

100. Daily Journal (Oct. 10/57 to Dec. 20/57, Jan. 2/58 to date of collapse).


FILE 1

102. Revised erection schedule—May 28/58.

103. Removal N4 procedure.

104. Joint L5 shims required.

105. Summary of gaps measured in bottom chord splices
     —North Anchor Span.

106. Tie down left leg 35 ton S.L. and miscellaneous
calculations on 35 ton S.L.—not in operation (may
     not be relevant).


108. Calculations for lengths of wire rope for 35 ton S.L.

109. Laboratory calibration of hydraulic jacks.

110. False bent bracing at N4 and N5. Pile driving report and settlement
     record during erection.

111. Bents N4 and N5. Pile driving report and settlement
     record during erection.

112. Quantities of bracing N4 and N5.

FILE 2

113. Rebound test cards for bent pile driving.

114. Pile driving records Bent N4 and N5.

115. Analysis of pile driving results and wind check on
     pile bracing.


117. Calculation sheets for preliminary work on Bent
     N4 grillage.

118. Calculations and data on 110 ton S.L. (probably
     preliminary).

119. Summary of maximum erection loads N4 and N5.

120. Erection calculation brief in folder.

121. Erection calculation brief in folder.

122. Live load stress calculation sheets.

FILE 3

123. Check calculations for N4 and N5.

124. Dead Load stress calculations.

125. Wind stress calculations in erection tie U0.

126. Calculation sheets for Approach Spans 1, 2, 3,
     and 4.

127. Calculation sheets for erection deflections.

128. Calculation sheets for roller shoes, Main Span
     (Pier 14).

FILE 4

129. Field Books—Pier 14 set out and levels.

130. Field Books—Chainage P14, P15 and N4 and N5.

131. Field Books—Set out pier 14.

132. Drawings—110 ton Stiffleg Derrick.

132.—(1) Drawing D1 General Design.

132.—(2) " D2 Derrick Cap on Trolley.

132.—(3) " E1 Diagram, Undercar.

132.—(4) " E2 Diagram, Derrick.

132.—(5) " E3 Loading Diagram.


132.—(7) " 1 Bottom Section Boom.

132.—(8) " 1A Middle Section Boom ( & Bottom).

132.—(9) " 2 Stiffleg.

132.—(10) " 3 Sill.

132.—(11) " 4 Middle & top Section Mast.

132.—(12) " 5 Base & Bottom Mast.

132.—(13) " 6 Base & Bottom Mast.

FILE 5

133.-(1) Drawing E1 Erection Marking Diagram—
     Span 1 & 2

133.—(2) " E2 Erection Marking Diagram—
     Span 3 & 4

133.—(3) " D1 Stress Sheet.

133.—(4) " 1 Expansion Pier Member.

133.—(5) " 2 Fixed Pier Member.

133.—(6) " 3 Floor Beams.

133.—(7) " 4 Floor Beams.

133.—(8) " 5 Floor Beams.

133.—(9) " Drawing 6 Bottom Chord

133.—(10) " 8 " "

133.—(11) " 9 " "

133.—(12) " 11 " "

133.—(13) " 12 Top Chord.

133.—(14) " 14 " "

133.—(15) " 17 " "

133.—(16) " 19 " "

133.—(17) " 21 " "

133.—(18) " 22 Vertica .

133.—(19) " 24 Diagonals.

133.—(20) " 25 Diagonals.

133.—(21) " 26 Diagonals & Vertica ls.

133.—(22) " 27 Jacking Girders.

133.—(23) " 28 Bottom Laterals.

133.—(24) " 29 Top Laterals.

133.—(25) " 30 Sway Bracing.

133.—(26) " 32 Deck Expansion Joints.

133.—(27) " 33 Catch Basins & Drain Grate.

133.—(28) " 34 Stringers Span No. 4.

133.—(29) " 35 Stringers Span No. 4.

133.—(30) " 36 Stringers Span No. 4.

133.—(31) " 37 Stringer Diaphragms.

133.—(32) " 38 Fence Brackets.

133.—(33) " 39 Fence Brackets.

133.—(34) " 40 Cast Iron Drains.

133.—(35) Drawing E6 Erection Harness Assembly.

133.—(36) " P1 Erection Process.

133.—(37) " 300 Harness Frame Details.

133.—(38) " 301 Harness Components.

133.—(39) " 302 Erection Eyebars & links.

133.—(40) " 303 Harness Components.
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DOCUMENTS FILED BY KIEWIT-RAYMOND
NUMBERED FROM 190 TO 224 INCLUSIVE

EXHIBIT

202. Copy of Bond for $1,500,000.00 in favor of B.C. Toll Highways, dated April 1st, 1957.
203. Memo pile driving data, undated.
204. Safety Discussion report by Mr. Francis of Workmen's Compensation Board, dated June 24th, 1957.
208. Unapproved copy of Second Narrows Bridge Contract No. 1 Drawings.
209. Index file to Field Engineering Books.
210. Field Books.
   L1, L2, L3, L4, L5, L6, L7, L8, L9, T1, T2, T3, T4, T5, T6, T7, T8, T9, T11, T12, T13, T14.
212. Drawing Bracket supports and form, dated April 26th, 1957 (Details).
214. Drawing Progress Schedule 1 to 14, dated May 4th, 1956.

EXHIBIT

216. Drawing Original Soundings of Pier 14, dated August 9th, 1956.
218. File Scouring Pier No. 15 containing:—
   (a) Soundings Outside Cofferdam, and copy.
   (b) Soundings for Scour at Pier 15 and copy.
   (c) Layout for taking Soundings at Pier 15.
220. File of Weekly Information Letters commencing January 23rd, 1956 to September 13th, 1957. (No. 1 to 87)
221. Wood Pile Sub-Contract containing:
   (a) Timber Pole Layout for Pier No. 12.
   (b) Layout of Piles for Piers 7, 8 and 9.
   (c) Statement of costs involved on pile driving and preliminary preparations for Pier No. 10.
   (d) Letter date September 22nd, 1956 from Eakins Construction Ltd., to Kiewit-Raymond, attention Mr. Judson Howell re pile driving operations.
222. File—Progress Schedules containing:
   (a) Second Narrows Bridge Contract No. 1—Progress Schedule, dated January 9th, 1956.
   (b) Plan Raymond Concrete Pile—Progress Schedule dated March 30th, 1956.
   (c) Plan Raymond Concrete Pile Progress Schedule, (undated).
   (d) Plan, Raymond Concrete Pile Progress Schedule, dated February 18th, 1956.
   (e) Plan Raymond Concrete Pile Progress Schedule (undated).
223. File—Borings:
   (a) Documents marked: “Locations” and Gow Boring Locations and reports from Foreman R. Price of Raymond Concrete, dated October 26th, October 27th, October 29th, November 5th, October 31st, November 1st, November 3rd, November 7th, November 8th and November 9th, 1956.
   (b) Reports from Rocanen Engineering Co. Ltd., for Hole No. 1, 2 and 3.
   (c) Reports for Engineering Drillers Ltd., covering: Hole 1, Pier 7, Hole 2, Pier 8, Hole 3, Pier 10, Hole 4, Pier 3, Hole 5, Pier 2, Hole 6, Pier 6.
224. Files:
   (a) Plan, soundings on west side of work tressel.
   (b) Plan, soundings at Bent 15.
   (c) Diagram—soundings at Pier 13, dated January 9th 1957.
   (d) Diagram—soundings at Pier 12, dated October 16th, 1956.
   (e) Diagram—soundings after final excavation, October 4th, 1956.
   (f) Diagram—soundings at Pier 12, dated October 28th, 1956.
   (g) Final soundings after excavation on Pier 13 (undated).
   (h) Sections Pier 13, January 9th, 1957. After Driving Bearing Piles.
224A. Progress Photograph Folders (1).
224B. Progress Photograph Folders (2).

DOCUMENTS FILED BY WORKMEN'S COMPENSATION BOARD

225. First Air Service Booklet.
226. General Accident Prevention Regulations Booklet.
   (a) Safety Committee Reports.
   (b) Inspection Reports.
   (c) Inquest Reports.
   (d) Correspondence.
DOCUMENTS FILED BY GREENLEES
PILEDRIVING COMPANY LIMITED

Exhibit
227. Letter from Dominion Bridge Company Limited to Greenlees Piledriving Company Limited re Second Narrows Bridge Falsework, dated November 5th, 1957.
228. Specifications for steel pipe pile footings for Bents N4 and N5.

DOCUMENTS FILED BY MCDONALD AND MCDONALD LTD.

NUMBER FROM 231 TO 240 INCLUSIVE

240. Report of analysis of aggregates for period:—Mar. 20, 21/57 Report No. 15P
   " " " Mar. 26/57, " " " 17P
   " " " Mar. 28/57, " " " 18P
   " " " Apr. 1/57, " " " 19P
   " " " Jun. 4/57, " " " 24P
   " " " Jun. 14/57, " " " 29P
   " " " Jun. 14/57, " " " 30P
   " " " Jun. 20/57, " " " 33P
   " " " Jun. 20/57, " " " 34P
   " " " Jun. 27/57, " " " 36P
   " " " Jun. 27/57, " " " 37P
   " " " Jul. 8/57, " " " 40P
   " " " Jul. 8/57, " " " 41P
   " " " Jul. 15/57, " " " 45P
   " " " Jul. 15/57, " " " 46P
   " " " Jul. 18/57, " " " 48P
   " " " Jul. 31/57, " " " 59P

DOCUMENTS FILED BY ROCANEN ENGINEERING COMPANY LIMITED

241. File containing test borings for proposed bridge at Second Narrows Burrard Inlet, accompanied by sketches, and letter from Rocanen Engineering Co. Ltd., to Swan, Wooster & Partners.

DOCUMENTS FILED BY COAST TESTING LABORATORIES LTD.

242. Physical test report of reinforcing steel: File S-6
   Report No. 1-1, dated May 30, 1956
   " " 2-2, " " May 31, 1956
   " " 3-3A, " " June 7, 1956
   " " 4-4A, " " June 15, 1956
   " " 5-5A, " " July 17, 1956
   " " 6-6A, " " July 23, 1956
   " " 7-7A, " " July 31, 1956
   " " 8-8A, " " July 31, 1956
   " " 9-9A, " " Aug. 7, 1956

Exhibit
243. Cast Steel Sole Plates (File S-8)
   Report No. 1 & 1A—June 8th, 1956
   " " 2 & 2A—June 12th, 1956
   " " 3 & 3A—June 18th, 1956
   " " 4 & 4A—June 20th, 1956
   " " 5 & 5A—June 22nd, 1956
   " " 6 & 6A—July 3rd, 1956
   " " 7 —July 3rd, 1956
   " " 8 & 8A—July 10th, 1956
   " " 9 & 9A—July 23rd, 1956
   " " 10 —July 23rd, 1956
   " " 11 —Aug. 8th, 1956
   " " 12 & 12A—Aug. 17th, 1956
   " " 13 & 13A—Aug. 24th, 1956
   " " 14 —Aug. 27th, 1956
   " " 15 & 15A—Aug. 31st, 1956
   " " 16 —Sept. 6th, 1956
   " " 17 —Sept. 14th, 1956
   " " 18 —Sept. 25th, 1956
   " " 19 and final—Oct. 9th, 1956

Beam Base Plates for south retaining wall of Second Narrows Bridge as detailed on drawings supplied by Kiewit-Raymond:
   Report No. 1 and Final Oct. 31st, 1956

Base Plates for beam seats at south approach retaining wall of Second Narrows Bridge as detailed on drawings supplied by Kiewit-Raymond:
   Report No. 1 and Final Oct. 31st, 1956

DOCUMENTS FILED BY G. S. ELDRIDGE & COMPANY LIMITED.
NUMBERED FROM 244 TO 295 INCLUSIVE

244. G. S. Eldridge Inspection Order No. 106857, dated April 4th, 1957.
246. Warnock Hersey Co. Ltd., Inspection Report, dated August 28th, 1957, (No. 1.)

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EXHIBIT


260. Eldridge Inspection Report No. 1

261. Eldridge Inspection Report No. 5

262. Eldridge Inspection Report No. 8

263. Eldridge Inspection Report No. 9


265. Eldridge Inspection Report No. 5

266. Eldridge Inspection Report No. 8

267. Eldridge Inspection Report No. 9


271. Eldridge Inspection Report No. 10

272. Dominion Bridge Company Ltd., Physical Inspection Reports Nos. 1 to 25 inclusive.


277. Eldridge Inspection Report No. 11

278. Eldridge Inspection Report No. 12

279. Eldridge Inspection Report No. 13


283. Eldridge Inspection Report No. 14

284. Eldridge Inspection Report No. 15

285. Sketch—main bearings.

286. Eldridge Inspection Report No. 19 and attached sketches.

287. Eldridge Inspection Report No. 20

288. Eldridge Inspection Report No. 21

289. Eldridge Inspection Report No. 22

290. Eldridge Inspection Report No. 23

RE: ROYAL COMMISSION
SECOND NARROWS BRIDGE INQUIRY

DESCRIPTION OF PHOTOGRAPHS CONTAINED
IN EXHIBIT NO. 296—NUMBERED 1 TO 200

1. Progress picture No. 1 dated Nov. 26th, 1957 No. 34985

2. " " " 2 Nov. 26th, 1957 " 34986

3. " " " 3 Dec. 9th, 1957 " 34997

4. " " " 4 Dec. 9th, 1957 " 34999

5. " " " 5 Dec. 23rd, 1957 " 35079

6. " " " 6 Jan. 20th, 1958 " 35097

7. " " " 7 Jan. 31st, 1958 " 35100

8. " " " 8 Feb. 15th, 1958 " 35157


10. " " " 10 March 11th, 1958 " 35218

11. " " " 11 March 25th, 1958 " 35290

12. Extra Progress picture

No. 12 dated March 28th, 1958 No. 35308

13. Progress picture No. 12 dated April 8th, 1958 No. 35318

14. " " 13 " April 22nd, 1958 " 35334

15. " " 14 " April 22nd, 1958 " 35335

16. Picture after collapse of Spans 4 & 5

No. 15 dated June 18th, 1958 No. 35476

17. Picture after collapse camera facing north.

No. 16 dated June 18th, 1958 " 35477

18. Aerial photo taken before collapse.

dated June 15th, 1958 " 58353

19. Aerial photo, taken before collapse, camera pointing easterly.

dated June 15th, 1958 No. 58353

20. Aerial photo taken after collapse.

dated June 17th, 1958 " 58374

21. Aerial photo taken after collapse, camera pointing north east.

dated June 17th, 1958 No. 58379

22. Aerial photo of Pier 14, and Span 5, camera facing south westerly.

dated June 17th, 1958 No. 58378

23. Aerial photo after collapse, camera facing easterly, of both Spans 4 and 5.

dated July 1st, 1958 No. 58386

24. Aerial photo after collapse of top of Pier 14 camera facing easterly.

dated July 1st, 1958 No. 58387

25. Aerial photo taken after collapse, closer view of top of Pier 14.

dated July 1st, 1958 No. 58388

26. Photo of splice plates South end, East top chord Span 4 camera facing directly down towards the water, taken at the north east end of Pier 14.

dated June 18th, 1958 No. 35512

27. Photo taken from top of Pier 14 looking down under side of Span 5 from westerly top of Pier 14.

dated June 18th, 1958 No. 35508

28. Photo showing general view of outer falsework, camera facing east.

dated June 18th, 1958 No. 35509

29. General view of outer falsework, camera facing north.

dated June 18th, 1958 No. 35510

30. General view of Pier N4, camera facing east.

dated June 18th, 1958 No. 35511


(R. C. M. P. PHOTO)

32. General view of collapsed structure camera facing north east.

(R. C. M. P. PHOTO)

33. General view of collapsed structure, camera facing north east.

(R. C. M. P. PHOTO)

34. General view of damaged derrick and head end of Pier after collapse, camera facing south east.

(R. C. M. P. PHOTO)

35. General View of Pier N4, camera facing south east.

(R. C. M. P. PHOTO)

36. General view of derrick boom after collapse, camera facing South East.

(R. C. M. P. PHOTO)

37. General view of derrick boom after collapse, camera facing north.

38. General view of structure after collapse, camera facing south east.

(R. C. M. P. PHOTO)

39. View of fractured concrete Pier 14, on the east face of the west leg of Pier 14.

dated June 18th, 1958 No. 35493

40. Photo of fractured concrete east face of the east leg of Pier 14.

dated June 18th, 1958 No. 35494

41. Photo of fractured concrete, east face of east leg of Pier 14.

dated June 18th, 1958 No. 35495

42. Photo of fractured concrete, west face of west leg of Pier 14.

dated June 18th, 1958 No. 35496

43. Photo of fractured concrete, west face of the east leg of Pier 14.

dated June 18th, 1958 No. 35497
44. Photo showing fractured concrete on the west face of the west leg of Pier 14.
45. Photo showing the north side of the top of Pier 14, camera facing south easterly.
46. Photo showing top of Pier 14 showing south side of Pier 14, camera facing north easterly.
47. Photo showing Pier N4—falsework piles on west side.
48. Photo of Pier N4, showing general view of supporting beams and tie rod. (?)
49. Photo of Pier N4, showing tie rod, different view.
50. Photo of Pier N4, showing deformed base on westerly side.
51. Photo of Pier N4, showing splinters spacers between falsework base beams.
52. Photo of Pier N4, showing deformed falsework base beams.
53. Photo of Pier N4, showing stiffeners at east end of falsework.
54. Photo of Pier N4, showing plywood bracing under base beams.
55. Photo of Pier N4, showing deformed base beams on easterly end of falsework pier (also shows drift pin).
56. Photo of Pier N4, showing deformed beams on westerly end.
57. Photo from top of Pier 14, view of interior of Pier—West Anchor.
58. Photo from top of Pier 14, showing West Anchor—general view, camera facing east.
59. Photo from top of Pier 14, view of corner of the west anchor, camera facing east.
60. Photo from top of Pier 14, of East Anchor, looking down into Pier.
61. Photo from top of Pier 14, of East Anchor, camera facing east.
62. Photo from top of Pier 14, of East Anchor, showing detail, camera facing west.
63. Photo from top of Pier 14, camera showing easterly corner of north side of Pier. dated June 18th, 1958 No. 35503
64. Photo from top of Pier 14 taken from top of easterly side of Pier showing damage to concrete on north side.
65. Photo from top of Pier 14 showing toggle arms on easterly side of Pier. dated June 18th, 1958 No. 35506
66. Photo—camera on westerly end of Pier N4, inner falsework facing west, showing deformed beams.
67. Photo—camera on westerly end of Pier N4 facing east showing deformed base plates.
68. Photo—camera on westerly end of Pier N4, facing upwards so as to show the lower west chord and upper end of falsework.
69. Photo—camera at North East corner of Pier N4, showing deformed base posts. dated June 23rd, 1958 No. 35516
70. Photo—camera at North East corner of Pier N4, showing deformed base beams. dated June 23rd, 1958 No. 35518
71. General view of North East corner of Pier N4.
72. General view of North side of Pier N4, camera facing due south.
73. General view of North side of Pier N4, camera facing due south.
74. General view of North West corner of Pier N4, camera facing south east.
75. General view of westerly side of Pier N4.
76. General view of south west corner of Pier N4, camera facing easterly.
77. General view of south west corner of Pier N5.
78. General view of southerly side of Pier N5, camera facing north.
79. General view of south east corner of Pier N5.
80. General view of crane, camera facing north westerly.
81. General view of crane, camera facing south easterly.
82. View of member located at north side of Pier 14 (East).
83. Profile shot of Pier 13, camera facing east.
84. Profile shot of Span 4 camera facing east (Continuation of No. 35531).
85. Profile shot of Span 4, camera facing east. (Continuation of No. 35532).
86. Profile shot of Pier 14, camera facing east. (Continuation of No. 35533).
87. Profile shot of Pier 14, camera facing east. (Continuation of No. 35534).
88. Profile shot of Pier 14, camera facing east. (Continuation of No. 35535).
89. Profile shot of Span 5, and Pier N5, camera facing east. (Continuation of No. 35536).
90. Profile shot of Pier 14, camera facing west. (Continuation of No. 35537).
91. Profile shot of Pier 14, camera facing west. (Continuation of No. 35538).
92. Profile shot of Pier 14, camera facing west. (Continuation of No. 35539).
93. Profile shot of Span 5, camera facing west. (Continuation of No. 35540).
94. Profile shot of Span 5 and Pier N5, camera facing west. (Continuation of No. 35541).
95. Corner view of member located at north side of Pier 14.
96. Photo in easterly tower of Pier 14, camera facing south east, showing wedging.
97. Photo in easterly tower, camera facing south west, showing wedging.
98. Photo in easterly tower of Pier 14, facing north east, showing wedging.
99. Photo in easterly tower of Pier 14, facing north west, showing wedging.
100. Photo in westerly tower of Pier 14, facing south east, showing partial wedging.
101. Photo in westerly tower of Pier 14, facing south west.
102. Photo in westerly tower of Pier 14, facing north east.
103. Photo in westerly tower of Pier 14, facing north west.
104. Photo at north side of easterly tower of Pier 14, facing south.
105. Photo at north side of westerly tower of Pier 14, facing south, showing toggle pin.
106. Photo, camera at north west corner of Pier 14, looking down, facing north.
109. Photo, camera at centre of north side of Pier 14 facing north of structure. (Continuation of No. 35557).
   dated June 26th, 1958 No. 35558
110. Photo, camera at south easterly corner of Pier 14, looking down and side of easterly side of Span 5.
   dated June 26th, 1958 No. 35559
111. Photo, camera at north east corner of Pier 14, facing south west, taking north westerly end of the upper chord of Span 5.
   dated June 26th, 1958 No. 35560

ADDITIONAL PHOTOGRAPHS FROM LEONARD FRANK STUDIOS RECEIVED JULY 23/58

112. No. 47 (35581) Photograph of Southwest corner of Pier 13 —camera pointing Northwest.
113. No. 48 (35582) Photograph of Southwest corner of Span 3 —camera pointing Northwest.
114. No. 49 (35583) Photograph of the top of the Southwest corner of Pier 13, showing a portion of the bottom of the Northwest corner of Span 4 —camera facing Southwestely.
115. No. 50 (35584) Photograph of Southwest corner of Pier 13 showing Northwest corner of Span 4 —camera pointing due South.
116. No. 51 (35585) Photograph of Southwest corner of Pier 13 showing Northeast corner of Span 4 —camera facing East (North east).
117. No. 52 (35586) Photograph of Southwest corner of Span 3 —camera facing due North.
118. No. 53 (35587) Photograph of Southwest corner of Pier 13 —camera facing downwards in a southeasterly direction.
119. No. 54 (35588) Photograph of the Southeasterly corner of Pier 13 and the Northeasterly corner of Span 4 —camera facing due South.
120. No. 55 (35589) Photograph of Easterly side of Pier 14 showing the toggle arms on the easterly side and the northeasterly corner of Span 5 camera facing almost due East.
121. No. 56 (35590) Photograph of the Northeasterly corner of Pier 14 —camera facing Northwest.
122. No. 57 (35591) Photograph of the Northwesterly corner of Pier 14 —camera pointing in a Northwesterly direction.
123. No. 58 (35592) Photograph (close up) of the Southwest corner of Pier 14 showing the Northwest corner of Span 5 —camera facing in a Southeastern direction.
124. No. 59 (35593) Photograph of the Southwestern corner of Span 4 —located at the Northwest corner of Pier 14 —camera facing due West.
125. No. 60 (35594) Photograph of Southwest corner of Span 4 located at Northwest corner of Pier 14 —camera facing due West.
126. No. 61 (35595) Photograph of Southwest corner of Span 4 located at Northwest corner of Pier 14 —camera facing Northeast.
127. No. 62 (35611) Photograph of Tie Plate No. 1 east face, dated July 21st, 1958.
128. No. 63 (35612) Photograph of Tie Plate No. 4 east face, dated July 21st, 1958.
129. No. 64 (35613) Tie Plate No. 5, east face, dated July 21st, 1958.
130. No. 65 (35614) Tie Plate No. 8, east face, dated July 21st, 1958.
132. No. 67 (35616) Tie Plate No. 4, west face, dated July 21st, 1958.
133. No. 68 (35617) Tie Plate No. 5, west face, dated July 21st, 1958.
134. No. 69 (35518) Tie Plate No. 8, west face, dated July 21st, 1958.
135. No. 70 (35519) Toggle from east bottom chord, Span 4 dated July 22nd, 1958.
137. No. 72 (35621) Block in position on west side of wooden pier, dated July 22nd, 1958.
138. No. 73 (35622) Boom of head end crane, camera facing up in northerly direction, dated July 22nd 1958.
139. No. 74 (35623) Close up of boom and cables of head end crane, camera facing south easterly, dated July 22nd, 1958.
140. No. 75 (35624) Photo of head end crane, camera facing downwards in a southeasterly direction dated July 22nd, 1958.
141. No. 76 (35625) Front end of the traveller—panel point 7 upper east chord dated July 22nd 1958.
142. No. 77 (35626) Front end of the traveller—panel point 7 upper west chord dated July 22nd, 1958.
143. No. 79 (35628) Telephoto, close up of the top of North face of East tower—Pier 14.
144. No. 80 (35629) Telephoto, close up of the top of North face of West tower—Pier 14.
145. No. 81 (35711) Bottom West chord recovered from water —West face.
146. No. 82 (35712) Bottom West chord recovered from water —East face.
147. No. 83 (35713) Bottom West chord recovered from water —close up of sling attachment.
148. No. 84 (35714) West stiff leg of traveller—damage to angle.
149. No. 85 (35715) North railway flat car recovered from water.
150. No. 86 (35716) South railway flat car recovered from water.
151. No. 87 (35717) Tie plate between cars recovered from water.
152. No. 88 (35718) Locomotive recovered from water.
153. No. 89 (35719) Locomotive recovered from water.
154. No. 90 (35720) Locomotive recovered from water.
155. No. 91 (35721) West inner grillage beam—west pile group of N4—East face.
156. No. 92 (35722) West inner grillage beam—west pile group of N4—West face.
157. No. 93 (35723) West outer grillage beam—west pile group of N4—West face.
158. No. 94 (35724) West outer grillage beam—west pile group of N4—East face.

PLYWOOD PHOTOGRAPHS

159. (35671) Bent N4 Plywood Pad below west shoe above stringers 129A—upper side.
160. (35672) Bent N4 reverse side of Plywood Pad (No. 1) below west shoe above stringers 129A, lower side.
161. (35673) Bent N4 Plywood Pad below east shoe above stringers 129A—upper side.
162. (35674) Bent N4 reverse side of Plywood Pad No. 3 below east shoe above stringers 129A—lower side.
163. (35675) Bent N4 Plywood Pad marked No. 7 from top of grillage beam MK 311 of west pile group upper side.
   Bent N4 Plywood Pad marked No. 5—on east grillage beam MK 311 of east pile group, upper side.
   Bent N4 Fragment from Pads marked No. 9 on top of grillage beam MK 311, upper side.
164. (35676) Bent N4 Reverse side of Plywood Pad (No. 7) from top of grillage beam MK 311 of west pile group, lower side.
   Bent N4 reverse side of Plywood Pad (No.
5) on east grillage beam MK 311 of east pile group, lower side. Bent N4 reverse side of fragment from Pad (No. 9) on top of grillage beam MK 311, lower side.

WEST TOWER—PIER 13

165. No. 95 (35769) Camera at south west end of Pier 13 facing east. Lower shot of approximately 4 ft. of south end of west face—West tower.

166. No. 96 (35770) Camera at south west end of Pier 13 facing east showing approximately an additional 4 ft. of part of south end of west face—West tower, to show crack in concrete. (Complementary to photograph No. 95).

167. No. 97 (35771) Camera at south west end of Pier 13 facing north east. Lower shot of south face of west Tower.

168. No. 98 (35772) Camera at south west end of Pier 13 facing north east showing part of the south face of west tower. (This photograph is complementary to No. 97).

169. No. 99 (35773) Camera located near south end of Pier 13 facing north west and showing lower part of south east corner of west Tower.

170. No. 100 (35774) Camera located near south end of Pier 13 facing north west and showing part of south east corner of west Tower. (Complementary to photograph No. 99).

171. No. 101 (35775) Camera located near south end of Pier 13 facing west showing south end of east face of west Tower.

EAST TOWER—PIER 13

172. No. 102 (35776) Camera near south east end of Pier 13, facing west showing lower part of south end of east face, East tower.

173. No. 103 (35777) Camera located near south east end of Pier 13 facing north west showing continuation on south face of crack in concrete shown in photograph 102.

174. No. 104 (35778) Camera located near south east end of Pier 13 facing north west showing upper part of east face of east Tower.

175. No. 105 (35779) Camera located near south end of Pier 13 facing north east showing lower part of south west corner of east Tower.

176. No. 106 (35780) Camera located near south end of Pier 13 facing north east and showing the upper part of the south west corner—east Tower. A continuation of photograph 105.

177. No. 107 (35781) Bottom west chord recovered from the water—damage to cover plates on south end. Head on view.

178. No. 108 (35782) Bottom west chord recovered from water. Damage to plates on south end. Side view.

179. No. 109 (35783) Bottom west chord recovered from water—damage to cover plates on south end. Close up detail view.

180. No. 110 (35784) Joint U8 from top East chord Span 4—General view.

181. No. 111 (35785) Joint U8 from top East chord Span 4—Side view.

182. No. 112 (35786) Joint U8 from top East chord Span 4—close up—with part of plates cut off.

183. No. 113 (35787) Joint U8 from top East chord Span 4—close up showing the three cut off plates at detail.

184. No. 114 (35788) Tie Plate No. 4 west face (T302A) after cutting out of samples.

185. No. 115 (35801) Bent N4—lower section west leg (after salvage).
327. General conditions of contract and forms of tender, agreement and bond for use in Civil engineering construction filed by R. Freeman.
329. Corrections in Transcripts Volumes 5 to 9 inclusive.
330. Letter from Phillips Barratt & Partners to Dominion Bridge Company—October 16th 1956, and Drawing ME1—filed by John Prescott.
331. Letter from Phillips Barratt & Partners to Dominion Bridge Company—June 22nd 1956, approval of sketch 507—filed by John Prescott.
332. Letter from Phillips Barratt & Partners to Dominion Bridge Company—August 8th, 1956, Drawing ME 3B also Province Photograph August 7th, 1956. Filed by John Prescott.
333. Professor Armstrong's Report re possibility of explosion.
335. W. Pryde's Report re possibility of explosion.
SCHEDULE II

Reports submitted on behalf of interested parties.

(a) Report by Phillips Barratt & Partners to Kiewit-Raymond. Exhibit 312.
(b) Report by H. H. Minshall to Local Union No. 97 of the International Association of Bridge, Structural and Ornamental Iron Workers. Exhibit 321.

The Reports listed in this schedule appear in Volume II.
SCHEDULE 12

Reports on examination and tests of special features.

(c) Professor William M. Armstrong's Report. Selection and testing of steel samples from upper grillage beams 2 and 3. Examination of connecting plates 302A and single shear tests on bolts. Exhibit 314.
(d) Professor Alexander Hrennikoff's Report. Investigation, tests and calculations regarding Grillage Bent N4—including stringers, use of wood in diaphragms and plywood pads. Exhibit 315.

The Reports listed in this schedule appear in Volume II.
Reports dealing with possibility of explosion on east leg or column of false bent N4.

(e) Telegram from Modjeski and Masters dated 16th October 1958, concurring in joint report of Messrs. Otter and Freeman. Exhibit 337.

The Reports listed in this schedule appear in Volume II.
Chief Justice Sherwood Lett,
Commissioner,
Royal Commission Second Narrows Bridge Enquiry,
Court House,
Vancouver, B.C.

Dear Sir:-

Below mentioned is the text of the recommendation concerning the Workmen's Compensation Board and made by the Building Trades Council to the Commission on Thursday the 14th of October, 1958:

"That without criticizing the Workmen's Compensation Board and its inspectors, but, in an endeavour to point out the importance of basic safety precautions, the Workmen's Compensation Board employ or otherwise acquire the services of a consulting engineer or consulting engineers to examine and check all critical points of stress, harnesses, ties, calculations, for safe loading, etc., on all major structures under construction or minor structures where an unseen safety hazard might exist at any point in the Province of British Columbia where such construction may be or is taking place or such other place as may be necessary to determine that the plans, practices and material, will permit a safe method of carrying out the erection or construction work".

In answer to the above the Workmen's Compensation Board takes the position that the recommendation is impracticable and unworkable for the following reasons:-

1. If such a program were adopted the Board would be interfering in a matter of management responsibility and attempting to encroach upon management's prerogatives.
2. The Board would be duplicating engineering services within the Province and competing with management for the best engineering brains on the market since there would be no purpose in acting on the recommendation at all unless the Board's engineers were better trained than consulting engineers on the job and better equipped to give opinions.

3. A large staff of such personnel would be required to handle all the details of all the heavy construction within the Province, the cost of which would place an unnecessary burden upon the industry concerned, more so by reason of the fact that it would be a duplication of services.

4. The net result would be that management would be relieved of responsibility for such accidents and that the Board would be in the consulting engineering business. Such a result hardly appears to have been contemplated by the Workmen's Compensation Act.

Yours truly,

[Signature]

CWP/ES

Acting Chairman, Workmen's Compensation Board.
VOLUME II

VOLUME II OF THE ROYAL COMMISSION REPORT HAS NOT BEEN REPRODUCED IN PRINTED FORM, BUT IS AVAILABLE FOR REFERENCE, IN THE PROVINCIAL LIBRARY, VICTORIA, BRITISH COLUMBIA.

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