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Recovery and Repair of the Second Narrows Railway Bridge

Réhabilitation et réparation du pont ferroviaire de Second Narrows

Wiederherstellung und Reparatur der zweiten Eisenbahnbrücke von Second Narrows

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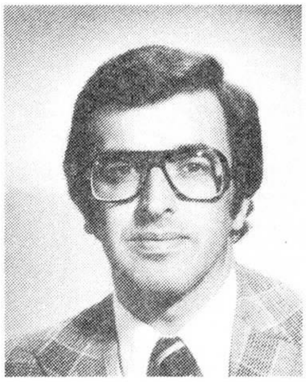

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SUMMARY

The paper describes the damage caused by a ship collision with the Canadian National Railway Lift Span in Vancouver, British Columbia. The theoretical and practical consideration of the stabilisation and recovery schemes are discussed and the equipment and procedures used in the twenty weeks it took to restore the train service are described in detail.

RÉSUMÉ

L'article décrit les dégâts causés par un navire entré en collision avec le pont de la société nationale des chemins de fer canadiens, à Vancouver, Colombie britannique. Les aspects théoriques et pratiques de la méthode de stabilisation et de réhabilitation sont abordés. Le matériel et les techniques mis en oeuvre durant les 20 semaines nécessaires au rétablissement des liaisons ferroviaires font l'objet d'une description détaillée.

ZUSAMMENFASSUNG

Der Artikel beschreibt den durch einen Schiffsaufprall auf die Brücke der Canadian National Railway in Vancouver, British Columbia, verursachten Schaden. Die theoretische und praktische Erwägung der Stabilisierungs- und Wiederherstellungspläne werden erörtert, und die verwendeten Anlagen und Verfahren werden genau beschrieben, welche zur Wiederherstellung des Zugverkehrs innerhalb von zwanzig Wochen benötigt wurden.

On October 12, 1979 the Canadian National Railway Bridge over the Second Narrows of Burrard Inlet, British Columbia, was extensively damaged when struck by the heavily laden vessel "Japan Erica". The ship was outward bound from Port Moody in a dense fog. The vessel struck the 77 m north tower span near its mid-point while the 152 m lift span was in the raised position at the top of the towers.

Although the ship was moving dead slow the impact knocked the north end of the span clear of the pier and displaced it laterally the width of the bridge (Fig. 1) by pivoting around the central wind post of the main pier.

The wind post prevented complete displacement of the span as the 12-64 mm anchor bolts in the pier members were sheared. The span was skewed approximately 9^{o} to the axis of the bridge and the north end dropped to a gravel bar at the foot of the pier.

The tower lost substantial support as the bearings were displaced to the extent that they were partially projecting over the N.E. and S.W. edges of the main pier. In this precarious state of equilibrium the lift span, which had been torn from the vertical tower guides (Fig. 3) was left hanging, hammock-like, from the lift ropes.

On the basis of a hastily prepared examination and submission the firm of Canron Inc., Western Bridge Division, was commissioned to restore the bridge to service. The plan, as submitted, envisaged three phases of the work: stabilization of the structure, recovery of the fallen span and restoration to service.

Appraisal of Damage

Figure 1

The initial appraisal of damage indicated a dramatic series of instantaneous failure mechanisms. The severe damage was limited to the bottom chord and web system between panel points (p.p.) L2 and L4 (see Fig. 2 for p.p. numbering). This damage permitted the remaining portion of the span to rotate against a plastic hinge which formed in the top chord at U2. An examination of the point of impact of the vessel at L3 failed to show any vertical striations in the paintwork indicating the explosive suddenness of the lateral displacement. Except for one member in the immediate area of impact, the top lateral system was undamaged, as was the portal bracing system.

All the bottom laterals which were visible from L0 to L6 displayed compression failure, although the floor beams and stringers in the undamaged panels did not exhibit any distress.

The top chord members from U7 to U2 were virtually undamaged except for two significant areas. The first was the plastic hinge in the chords and the second was the compression failure of the diagonal brace from U3E to U2W. The latter was of great importance during truss separation.

The tower span was thus acting as a bent shore resisting the over-turning thrust from the northwest corner of the lift span which was bearing against the west leg of the tower. The intensity of bearing was a matter of considerable conjecture. No visible distortion of the tower leg was apparent nor were there any abrasion scars on the south face. The jacking girder in the north tower diplayed some damage due to the reaction on the wind post. Otherwise, the main members in the north tower were undamaged.

There were no outward signs of distress in the winding machinery. The sheaves were free of galling and the enamelled mechanical components did not exhibit any paint crazing. Couplings, drives, pinions and gears were inspected and were found to be free of any external indication of strain.

Examination of the lift span was not possible during the initial appraisal due to inaccessibility. Inspection the following week, however, indicated that while there was substantial crushing of the 60 mm diameter lift ropes where they passed through the lift girder, and some surface abrasion caused as the ropes were torn through the weather skirting of the towers, they were otherwise capable of carrying the 1000t reaction of the span during the salvage operations. The top flanges of the lift girders, however, which were normally stayed against lateral deflection by a tie to the top lateral system, were bowed outward by the horizontal component of the 40 lift ropes at each end. This reaction sheared the connection of the girder tie and bowed the north girder outward 76 mm and the south girder 44 mm. The top flange of the lifting girder, fortunately, was the tension flange and such distortions did not create a buckling problem. The condition of the north girder was considered to be inimical to the safety of the structure.

Immediately following the accident the Harbour Master closed the passage to heavy marine traffic but permitted smaller vessels such as tow boats and pleasure craft to use the channel under the south tower span. The wisdom of this order became apparent during the initial damage survey when it was noticed that as each vessel made the transit of the south channel the vibration of the vessel's screw could be felt in the higher elevations of the south tower. It was abundantly clear that the equilibrium of the system was extremely sensitive, so on the second day after the accident a series of survey check points was established on the north and south towers and main piers. Concurrently with the survey, divers were engaged to evaluate and report on the conditions of bearing of the north end of the fallen tower span on the gravel bar.

At this time the philosophy of the salvage and recovery system was firming up and it was determined that the first stage of the work should be the stabilization of the structure.

STABILIZATION MEASURES

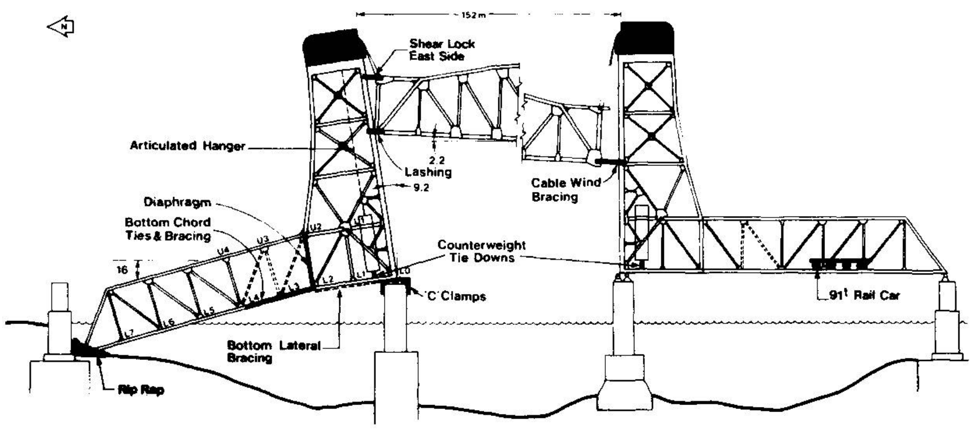

Figure 2

Stabilization

Figure 3

Upon receipt of the instruction to undertake the work the Canron engineering group directed their efforts toward the predominant need to stabilize the structure and to prepare recovery procedures. Stabilization included the immediate installation of wire rope diagonal bracing between the lift span and the south tower and shear devices between the lift span and the north tower. Concurrently with this work the north lift girder was stayed against further movement and the 1000t counterweight in the north tower was prevented from further downward movement by the installation of articulated platework hangers capable of carrying the total weight. Upward movement of north and south counterweights was prevented by the rigging of heavy down-haul tackle.

To ensure that the south tower span did not tip under any sudden increase in the horizontal component of cable forces, a 91t railcar was spotted over the south end of the south tower span.

In the meantime crews were installing heavy C-clamp type weldments between the foot of the tower and the main pier while others were clearing away the span locking equipment of the north main pier and removing the deck and damaged steel in the way of temporary works.

Due to the loss of the web members in panel L2 to L4 it was necessary to inhibit any further tendency to hinge at U2. Accordingly, heavy structural sections were cut and fitted on site to make the bottom chord continuous from L2 to L4. A temporary vertical U3-L3 was installed and the vertical bracing in the plane of the north face of the tower was extended down through the truss to L2. The bottom lateral bracing system was restored from L0 to L2 and so transformed the tower portion of the structure into an integrated vertical box truss, a condition which was to present some problems during subsequent operations.

Upon completion of stabilization the passage was re-opened to deep water shipping and the upper docks resumed normal activity. From the outset of the work divers carried out a daily check on the stability of the gravel bar on which the north end of the span rested. The bar was subject to heavy tidal currents of the flood tide and hence destructive scouring. To prevent scouring, rip-rap was placed at L8. The underwater survey was carried out daily until completion of the salvage operation.

During the work of stabilization a final appraisal of the damaged structure provided the information necessary to complete the engineering analysis and design.

Engineering

The scope of the work and complexity of analysis engaged not only Canron's staff of seventeen engineers and draughtsmen but also required the services of nine consulting engineers of various disciplines.

Paralleling the work of stabilization, loads and stresses were being analysed from calculations made from the original centres of gravity of the structure, physical measurement of loads in the lift ropes and, by inference, from the effect of the bearing of the bottom chord against the west leg of the north tower.

These calculations were verified in sense and in degree by a series of readings of residual stresses in sensitive members and joints which were obtained by the "blind hole" drilling method of photo-elastic strain analysis.

Design of falsework and procedures, however, did not await the receipt of this information but, instead, were designed on the basis of upperbounds and verified by model studies.

The scope of this paper does not allow for a detailed description of the analytical steps. However, the interrelation of model tests, computer analysis, field measurement and upper and lower bound scenarios was germane to the fast tracking of the recovery scheme. The total elapsed time from accident to first train crossing was only 20 weeks and, from the amount of falsework and equipment that had to be designed and fabricated, it is obvious that engineering was on the critical path.

Figure 4 shows the accuracy that the various analyses achieved by plotting the jacking forces at L2 as the tower rotated to the vertical. The lower bound calculations were derived from measured forces and theoretical analysis and, as can be seen, are very close to the forces predicted by model analysis.

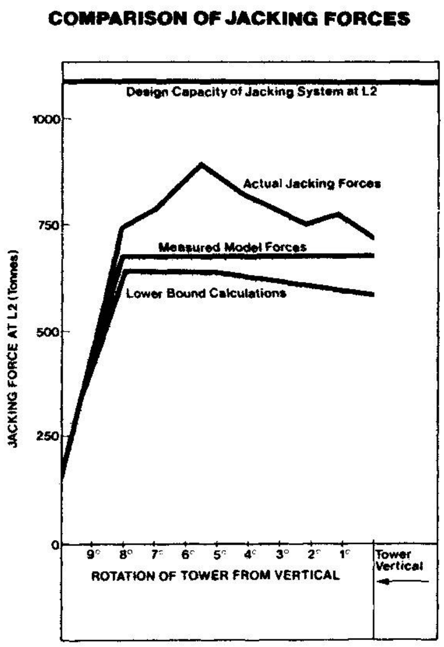

Figure 4

The actual values had a disturbing peak in them which was caused by a jammed shear lock. Without this occurence it is believed that the measured values would have paralleled the model values closely. Even with the jamming, the maximum jacking forces did not come close to the upper capacity provided in the recovery scheme.

The design capacity had been conservatively estimated by considering the tower weight and ignoring the restraining forces from the inclined cables.

Recovery

The philosophy of the recovery procedure envisaged isolating the lift span and tower from the partially submerged portion of the span. This required the provision of falsework under the back legs of the tower of sufficient capacity to carry the heaviest vertical reaction and provide a base large enough to carry a jacking frame, and a transversing system which would be required to re-align the tower after plumbing (Fig. 10). Support for this falsework, which was designed for 1091t, was provided by two groups of 12 - 500 mm dia. pipe piles supporting a transverse girder system of adequate size to

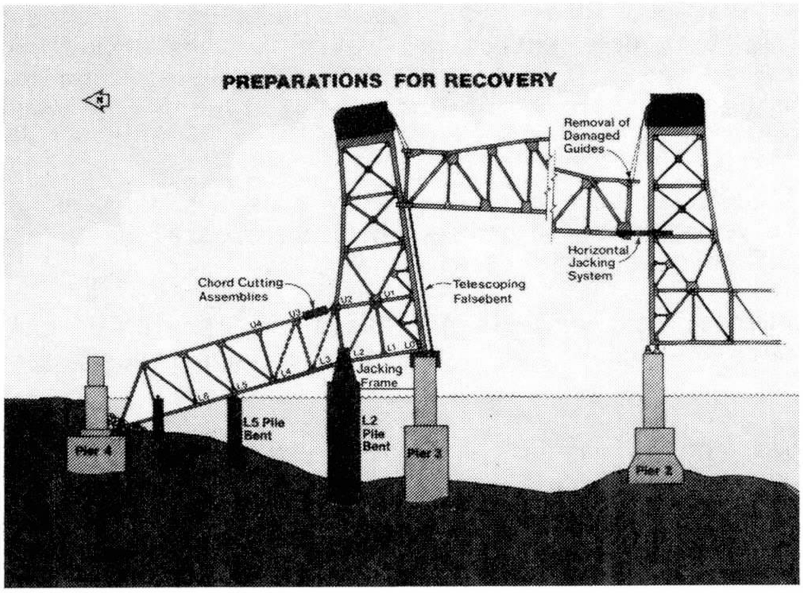

Figure 5

carry the jacking-skidding system (Fig. 11). This platform braced to Pier 3 with 610 mm dia. pipes. The attachment to the pier was made with Dywidag bars in holes cored through the foundation legs. 914 mm dia. pipe piles supported falsework of 455t capacity at L5, and 91t capacity at L7. The L5 and L7 bents were necessary to support the partially submerged span after separation from the tower. Bent L5 was designed to act as a pivot bent on which the severed truss span would rotate as it was raised to the horizontal for subsequent dismantling in large sections.

Construction of Pile Bents

Installation of the piles by a vibratory hammer was elected due to the sensitivity of equilibrium of the system which could have been upset by the dynamic action of a reciprocating pile hammer.

During the driving of the 500 mm dia. pipe piles the tides produced currents of up to 8.4 km/h which engendered destructive vortex shedding in the piling and produced several modes of vibration.

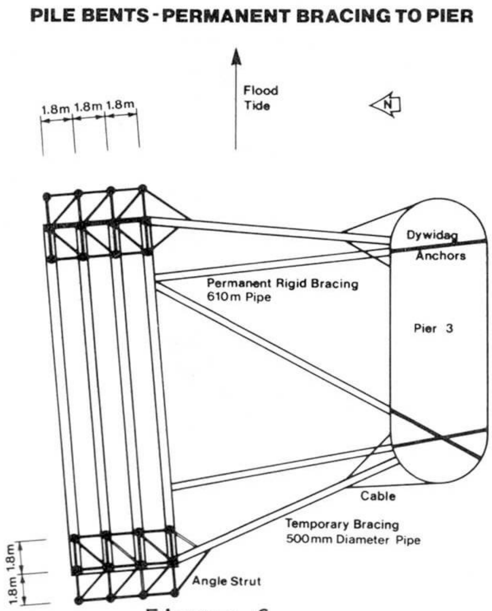

Figure 6

Figures 6 and 7 show the final configuration of the L2 bent. The table beside Figure 7 lists the variety of oscillations observed in the bent during construction. The amplitudes in most cases are visual estimates and therefore, are not too accurate. The measurement that is guaranteed, however, is the Stage 1 plus or minus 900 mm value. In this case the piles were driven at 1800 mm centres and during oscillations actually made contact with each other. Tidal flow and frequency were accurately measured. Vibrations only occurred for about 1 to 2 hours during every other tide during a two week period. It was unfortunate that the construction schedule did not fully coincide with the tidal cycle. Attempts to damp out the oscillations with ropes attached to the piers were unsuccessful. It was fortuitous that the oscillations almost always occurred after the dayshift was complete (7 p.m. to 9 p.m.) or before it started (3 a.m. to 5 a.m.) and work could always continue on bent completion during daylight hours with no oscillations. The Stage II oscillations came as a considerable surprise since group oscillation was considered to be unlikely. The east pile group survived the first set of oscillations, but top bracing of the west group was partially destroyed and was subsequently re-installed.

Temporary 500 mm dia. tube struts were installed from the bents to the piers which eliminated the violent crossflow vibrations but replaced them with plus or minus 76 mm in-line vibrations.

The jacking bent was completed as designed with 600 mm dia. pipe bracing and no serious vibrations were detected thereafter.

TYPICAL SECTION THROUGH PILE BENT

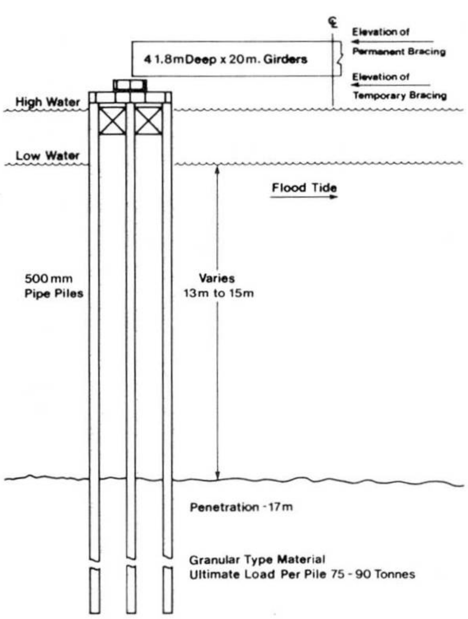

Figure 7

SUMMARY OF PILE VIBRATIONS

STAGE	HALF AMPLITUDE	DIRECTION	CYCLES/ MINUTE	CURRENT (KNOTS)
I	900 mm	N/S crossflow	35	4 to 6
II East	220 mm	N/S crossflow	50	4 to 5
II West	450 mm	Do	35	4 to 5
III	75 mm	E/W in line	50	2 to 5
IV	75 mm	Do	50	2 to 5
V	7.5 mm	E/W	180	5.4
	2.5 mm	Bowstring	180	4.0
	.75 mm	E/W Random	Random	4.0

The jacking bent was capped by 4 - 1800 mm deep plate girders which supported the jacking frames during the jacking stage and later supported the skidding frames as the tower was re-aligned. The jacking frames each contained two welded channel members braced in position. A box beam which spanned between these members supported the truss which was raised through the box beam by two 318t jacks in each jacking frame.

Structural Separation and Re-alignment (Fig. 9, 10, 11)

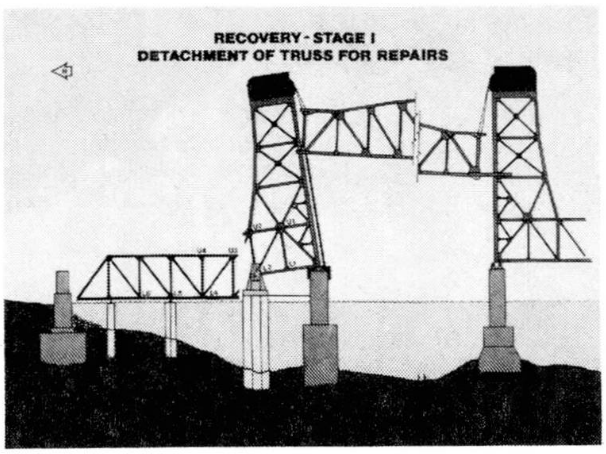

Figure 8

Prior to separating the tower from the horizontal truss, the tie-downs were installed to the box beam and the 318t jacks at L2 were loaded to provide a positive reaction of 45t per side. This ensured no sudden movement of the tower when the chords were cut. The pivot bearings on bent L5 were brought into contact by flat jacks and shimmed in place and thus prevented sudden movement during chord cutting.

The lift span was supported by a pin-connected telescopic jacking bent supported on the main pier in front of the north tower. This bent did not carry load initially but was brought into light contact only with the lift span by two 363t jacks in each leg of the bent. Support under the lift span was through a teflon-coated rocker bearing on a traversing girder. This bearing allowed easterly traversing of the lift span as the tower was plumbed.

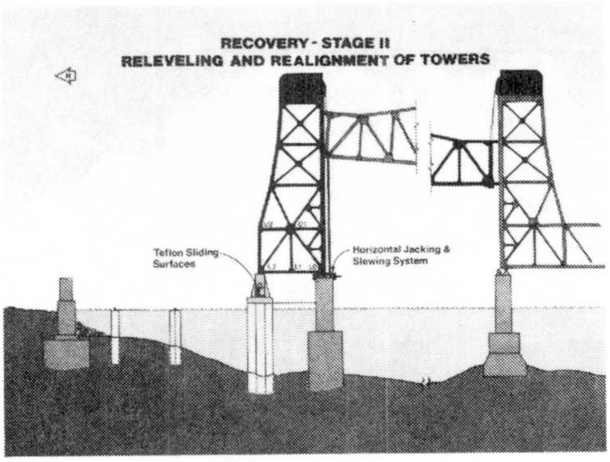

Figure 9

Cutting frames were installed on the east and west top chords and the U3W-U2E diagonal. The chord cutting frames contained four 180t jacks and the diagonal frame two 90t jacks.

The temporary bottom chords and the crippled top lateral bracing from U2 to U3 were subsequently parted. The U3W-U2E diagonal was cut using oxy-acetylene torches and the load released by retracting the jacks. The top chords were then cut simultaneously by use of oxygen lances. The east chord was the first to be cut completely and showed to be in a state of zero stress. The load in the west chord was released after cutting by retracting the jacks. The maximum measured compression load was 220t as the tower reaction was transferred to the jacking frame.

The jacking frame supported the tower through a teflon-coated rocker contained between two 55t jacks.

As the bottom chord was raised the jacks were activated to ensure that the frame remained plumb as the locus of the point of contact moved first to the south and secondly to the north.

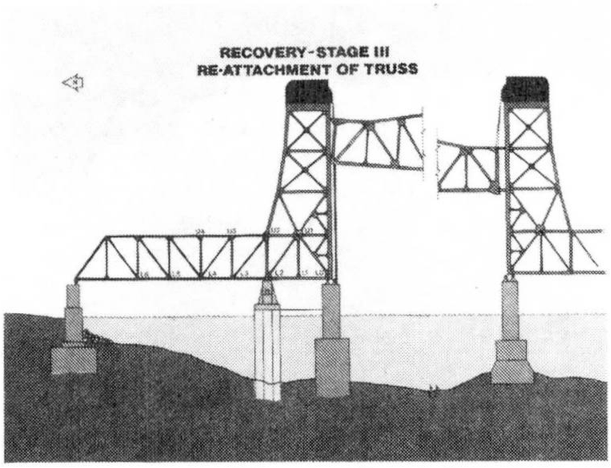

Figure 10

Total lift required to bring the chord to geometric elevation was 3 m. The box beam was raised in 400 mm increments using a climbing system employing 190 mm diameter retaining pins. Controls during jacking monitored both individual jack loads and, by an ingenious system of lights, the level of the four corners of the box beam. Maximum movement of the head of the jacking frame relative to the node point was 150 mm to the south, thence 25 mm to the north.

As the tower was jacked to the vertical the effect of the rack in the span, which was now contained by the vertical bracing, caused the jacks to be more heavily loaded on the east side than those on the west. At one time this disparity in loads increased uncomfortably close to the design loading of the easterly pile group.

The locus of the tower top during jacking was south east while the lift span was supported N-S. Therefore, during the jacking of the tower it was necessary to traverse the lift span to the east as the weight of the lift span was transferred to the bent.

The pier reaction of 2636t from the tower was subsequently transferred to two skidding sleighs each containing three 509t jacks. The jacking frame at L2 was replaced by a skidding frame which enabled the tower to be slewed into alignment and on to chainage. The tower was pivoted through a structural system on the main pier. The skidding sleighs, which were controlled by opposing jacks on a fixed radius from the pivot, were carried on a teflon bearing surface against a polished steel sheet.

Similarly, the skidding frame at L2 was supported on teflon bearings moving on a polished steel surface. Total movement at L2 was 3100 mm.

Concurrently with the jacking-slewing operations, the submerged portion of the span was raised level, stripped of damaged members and dismantled as two trusses. These trusses were set up vertically on two scows and towed to a yard where replacement members were erected. The two 155t trusses were then returned to site and erected to the tower span using a 364t capacity marine crane. The floor system and chord bracing were installed concurrently with the removal of the bent at L2.

With the completion of the north tower the false bent under the lift span was lowered in decrements of 400 mm as for the jacking at L2, transferring the reaction of the lift span back to the lift ropes.

On the morning of March 3 the lift span was lowered, using a combination of tackle and the winding machinery. On March 4 the span was opened to rail traffic.

Although the ropes had suffered only minimal damage it was deemed to be prudent to replace them while it was opportune to do so without any great inconvenience to marine or rail traffic. Therefore, while the bridge was open to rail traffic and with the harbour closed for the next 16 days, crews worked around the clock to replace 80 - 60 mm diameter lift ropes.

Figure 11

Only twenty weeks elapsed from the date of the accident until restoration of rail service. It was a feat made possible by the enthusiastic support of all who participated in the work; the engineering community, draughtsmen, fabricators, sub-contractors and the transportation groups; the marine contractors who, in recognition of the exigency of the work, made available their heavy lift water borne equipment, and above all the ironworkers, whose performance under severe conditions of weather and risk carried the work through to a successful conclusion.