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16. Abstract

This report presents a study of a movable bridge failure which involved a fractured trunnion in a vertical lift bridge erected in the early 1930's. The failure was induced by propagated fatigue cracking which became critical after about 600,000 cycles of complete stress reversal. The major contributing factor initiating the failure was the development of high concentrated stresses in a critical fillet area resulting from an abrupt change in section in the configuration of the trunnion. Repairs to the structure included a redesign of the trunnion utilizing a high-strength steel and providing a larger fillet in the area where a change in section occurs.

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INVESTIGATION OF TRUNNION FAILURES INVOLVING MOVABLE VERTICAL LIFT BRIDGES

By

Floyd K. Jacobsen

Special Report
IHR-104

A Research Study Conducted by
Illinois Department of Transportation

June 1980

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INVESTIGATION OF TRUNNION FAILURES INVOLVING MOVABLE VERTICAL LIFT BRIDGES

INTRODUCTION

In early July 1978, one of the four trunnions supporting the lift span of Shippingsport Bridge had suddenly failed with little or no warning. The bridge was under normal operation at the time of failure which occurred as the lift span was being lowered. Immediately prior to the failure, the lift span was raised to permit the passage of barge traffic using the waterway.

Even though the extent of the damage resulting from this failure did not result in a total collapse of the structure, bridge design and maintenance engineers directly involved with the repair of the Shippingsport structure were highly concerned about the potential of a catastrophic failure that could have occurred as a result of the broken trunnion. An investigation was made so that a report could be disseminated to other engineers responsible for the inspection and maintenance of movable bridges. The findings of this investigation are presented in this report.

Background

The use of vertical lift span bridges for crossing navigable rivers and canals was considered as early as 1850. However, little progress was made until 1872 when small lift spans were constructed to cross the canal system in New York. The South Halsted Street Bridge at Chicago was perhaps the first major structure constructed in this country. Although this bridge was designed in 1892 and constructed shortly thereafter, it was not until 1908 that the use of vertical lift bridges became more widely accepted for water crossings requiring a movable type bridge.

The Shippingsport Bridge is located near La Salle, Illinois, and carries Route 351 over the Illinois River (Figure 1). The bridge was designed and constructed during the mid 1920's and opened to land traffic in October 1928. Provisions were included in the original design for adapting a movable lift span to provide vertical clearance for future waterway traffic. The modifications were made in the early 1930's and completed before June 22, 1933, when the Illinois River was opened to inland waterway traffic. The movable lift span was in service for about 45 years before the failure had occurred.

The lift span is 260 ft (79.2 m) long and is supported by two towers each consisting of four columns braced in both directions. The two front columns of each tower are vertical and the two rear columns are inclined. The sheaves are directly supported by the vertical columns (Figure 2). The design provides 64 ft (19.5 m) of vertical clearance above high water when the span is raised to its highest position.

The weight of the bridge is slightly over 1,000 kips (4.4 MN). Counterweights of 500 kips (2.2 MN) are provided at each end of the lift span so that the lifted weights are balanced by the counterweights. The counterweights move up and down inside the towers (Figure 3).

Each end of the counterweight and each corner of the lift span is attached to 8 2½-inch (63.5 mm) diameter wire cables (Figure 4). The lift span and the counterweight are supported through the cables by a 12½-ft (3.8m) diameter sheave (Figure 5). The sheaves are grooved to fit the cables. Equalizers are also provided between the counterweights, the lift span connections, and the cables. The trunnion which serves as a shaft for the sheave has a main diameter of 11 3/4 inches (298.4 mm) which is reduced to 10 inches (254.0 mm) at the journal box (Figure 6). The trunnion is designed for bending,

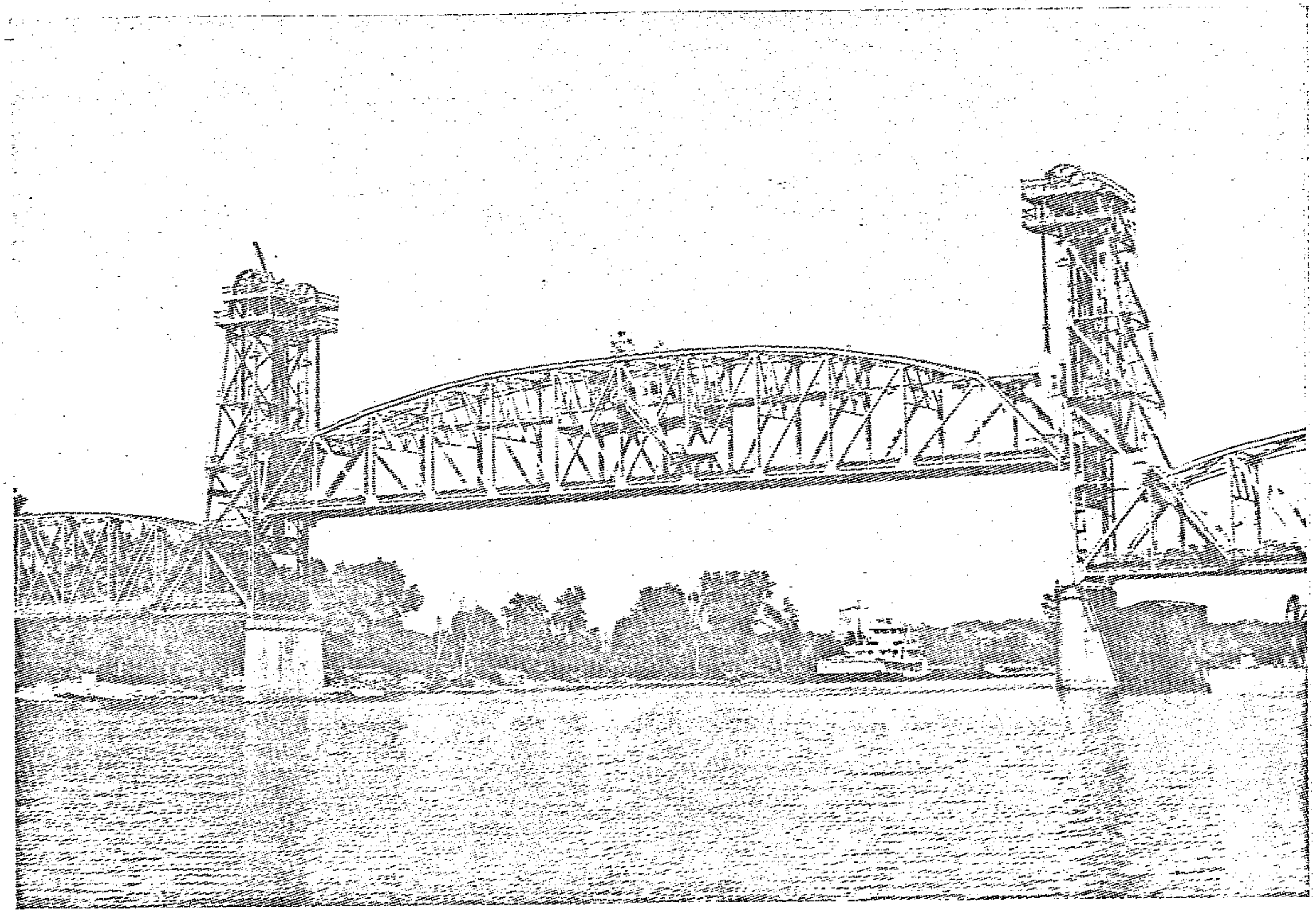


Figure 1. Shippingsport Bridge located near La Salle, Illinois.

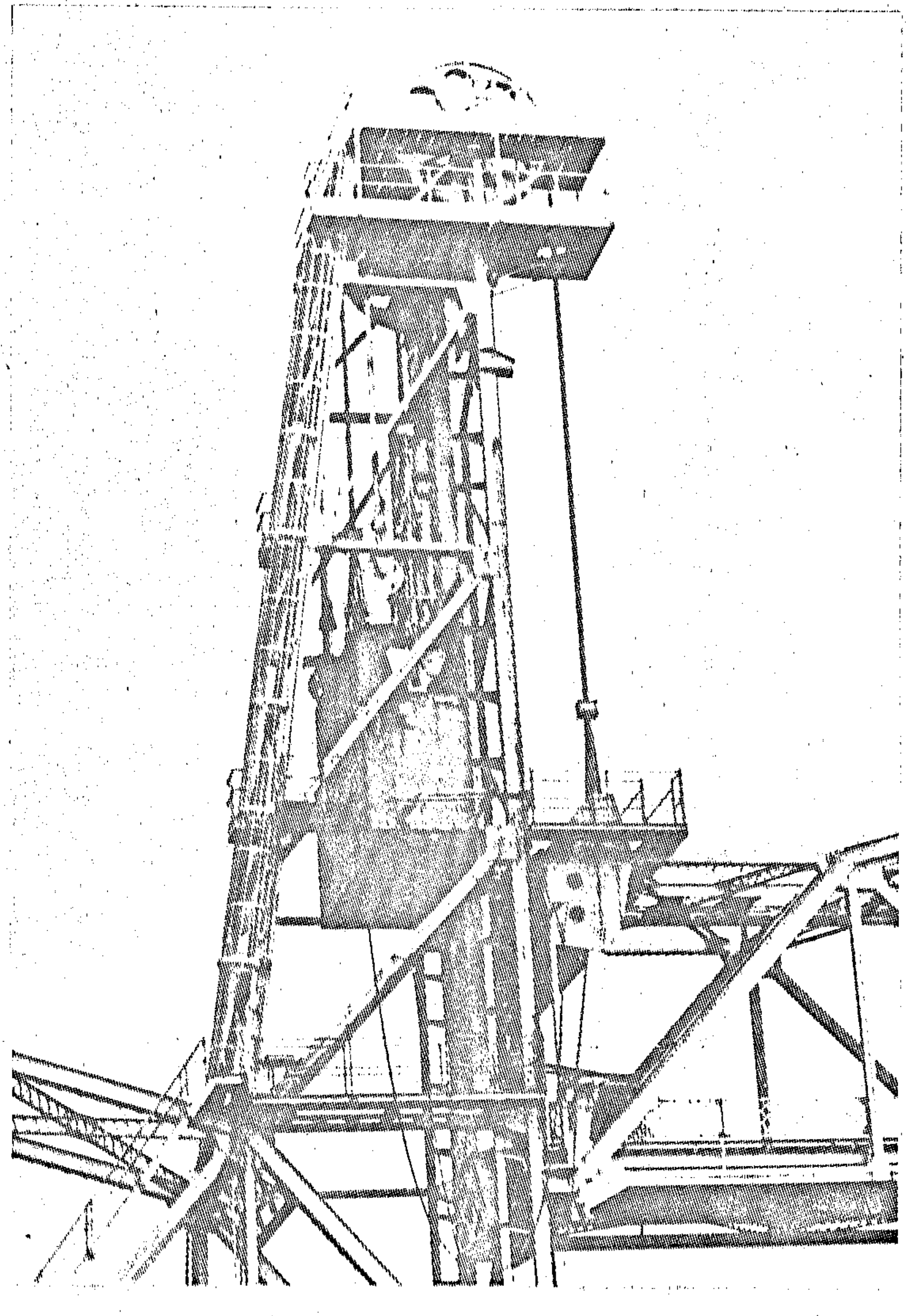


Figure 2. North supporting tower for lift span on Shippingsport Bridge.

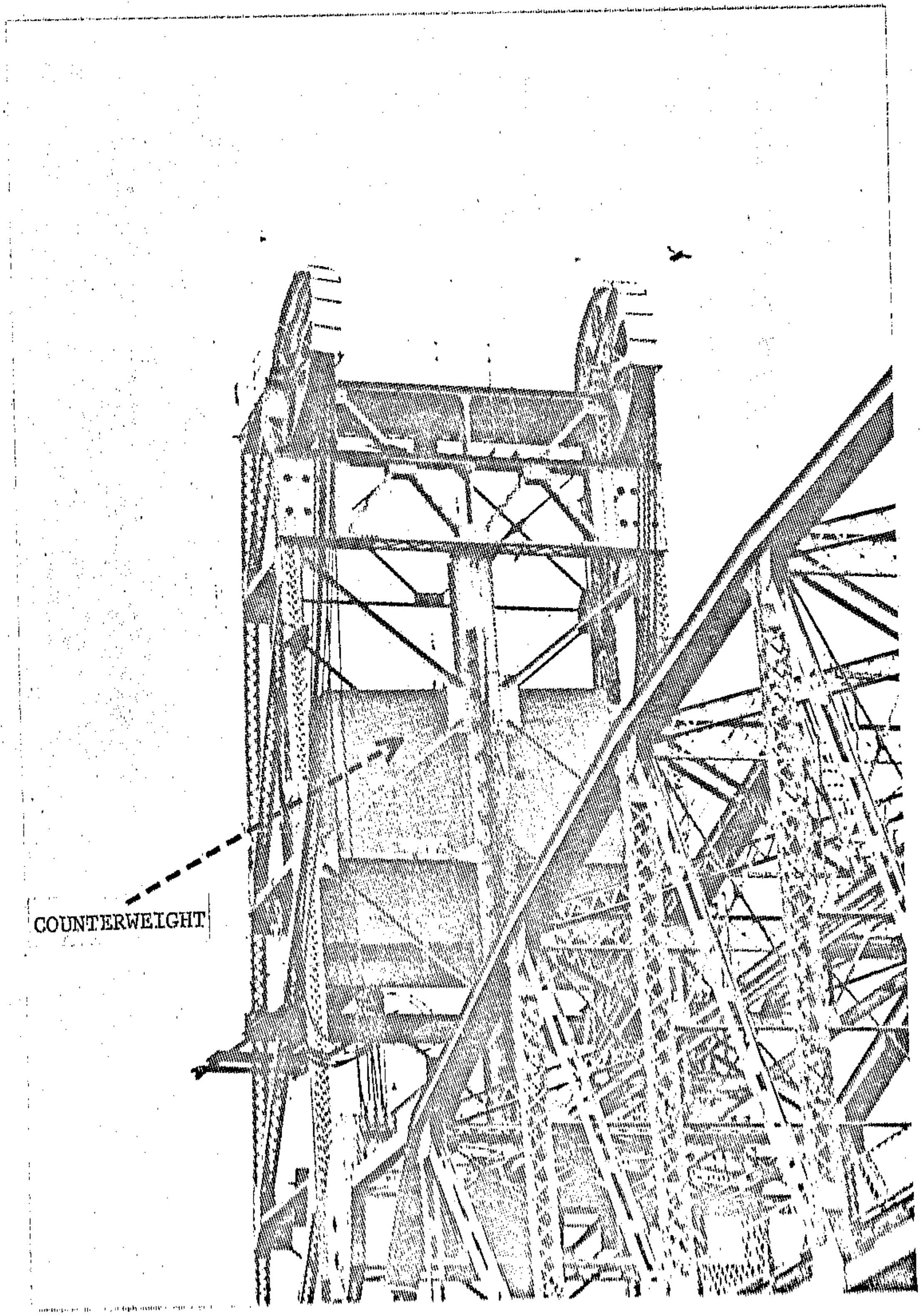


Figure 3. Supporting tower showing counterweight.

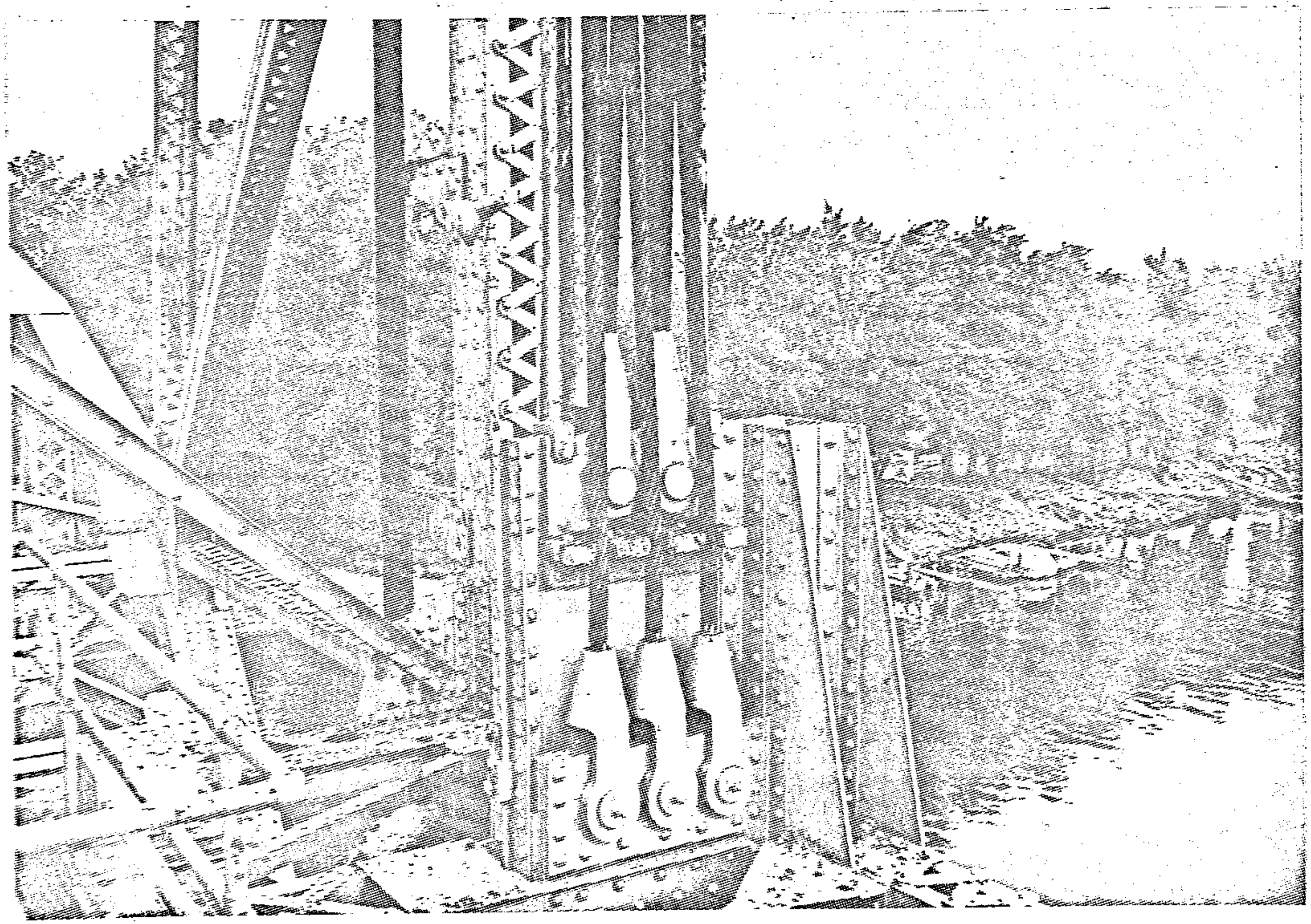


Figure 4. Wire cables supporting lift span and counterweight.

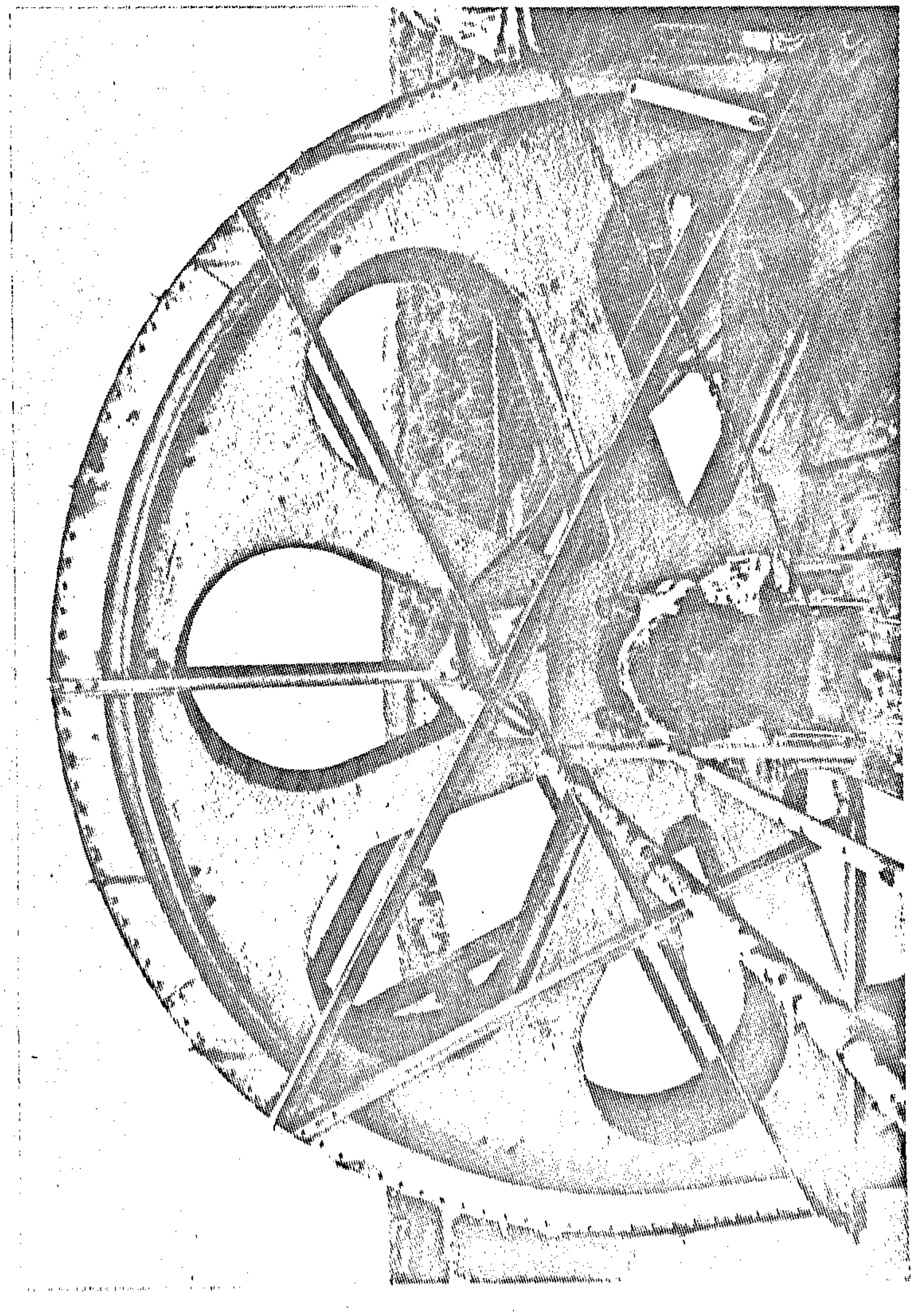


Figure 5. Side view of 12 1/2-ft diameter sheave.

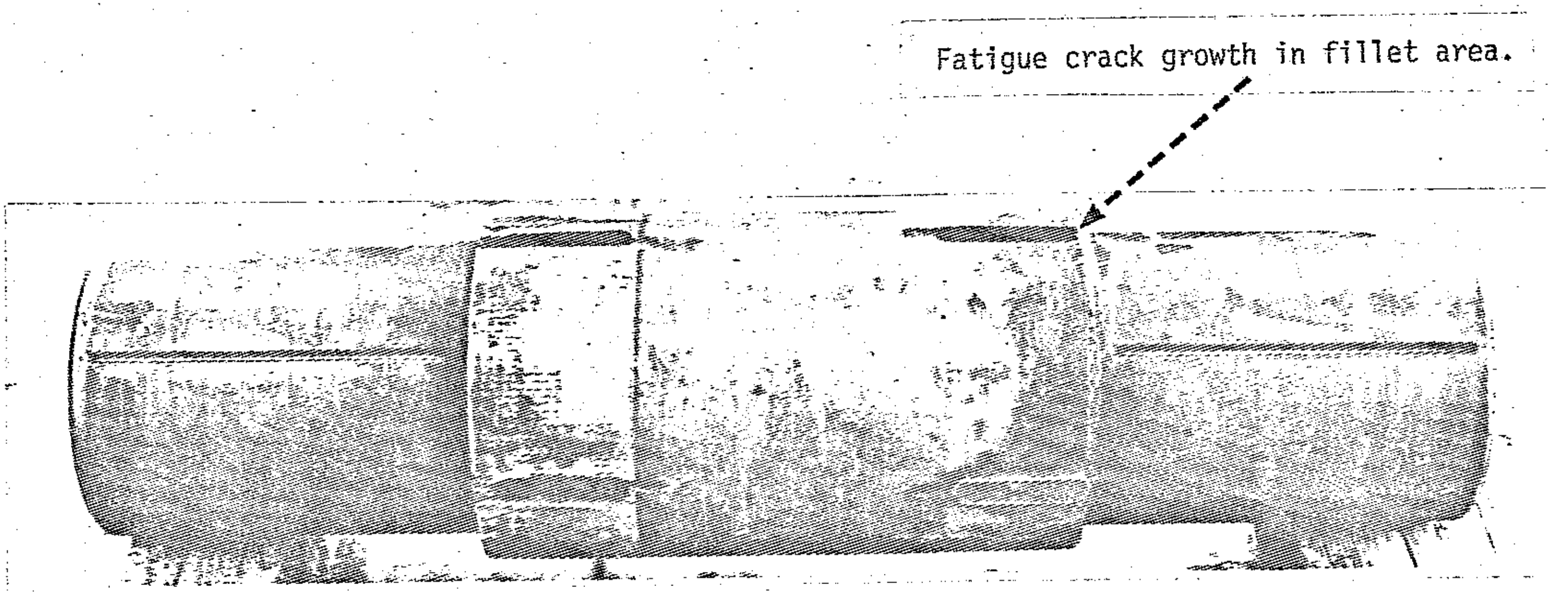


Figure 6. View of 10 1/2- to 11 3/4-inch trunnion.

bearing, and shear stresses based on the nominal section at the bearing journal. Except for a 1/2-inch (12.7 mm) fillet, little or no consideration is given to the abrupt change in section of the shaft from the sheave to the journal box. This feature was also commonly used in the design of trunnions for bascule bridges.

The failure was induced by a fractured trunnion that broke on the inboard side of the subposts that fasten the sheave to the tower (Figure 7). Immediately after failure, the inboard side of the sheave dropped about 5 inches (127.0 mm) (Figure 8) and came to rest with the inner rim of the sheave being supported by the crossframe at the top of the tower. The outboard subpost was partially collapsed (Figure 9) and the inner portion of the broken trunnion came in contact with the bottom saddle of the bearing block. The main restraining elements preventing a total collapse appeared to be the anchor bolts that fasten the inboard bearing journal to the inner subpost (Figure 10) and the severely damaged channels at the base of the outer subpost.

Even though in this particular case the damage was relatively minor, a catastrophic failure could have occurred if the conditions of structural stability and the mechanism of failure had varied. One of the main supporting elements which may have helped avert a total collapse of the tower is the top crossframe which provides lateral bracing or support at the top of the inboard subpost. If the failure had occurred on the outboard side of the trunnion, it is questionable as to whether or not the unsupported outboard subpost could withstand the lateral thrust of the sheave as in the case of the inboard fracture. Should the support system for the sheave become unstable or unable to support the sheave in an upright position, it is doubtful that the

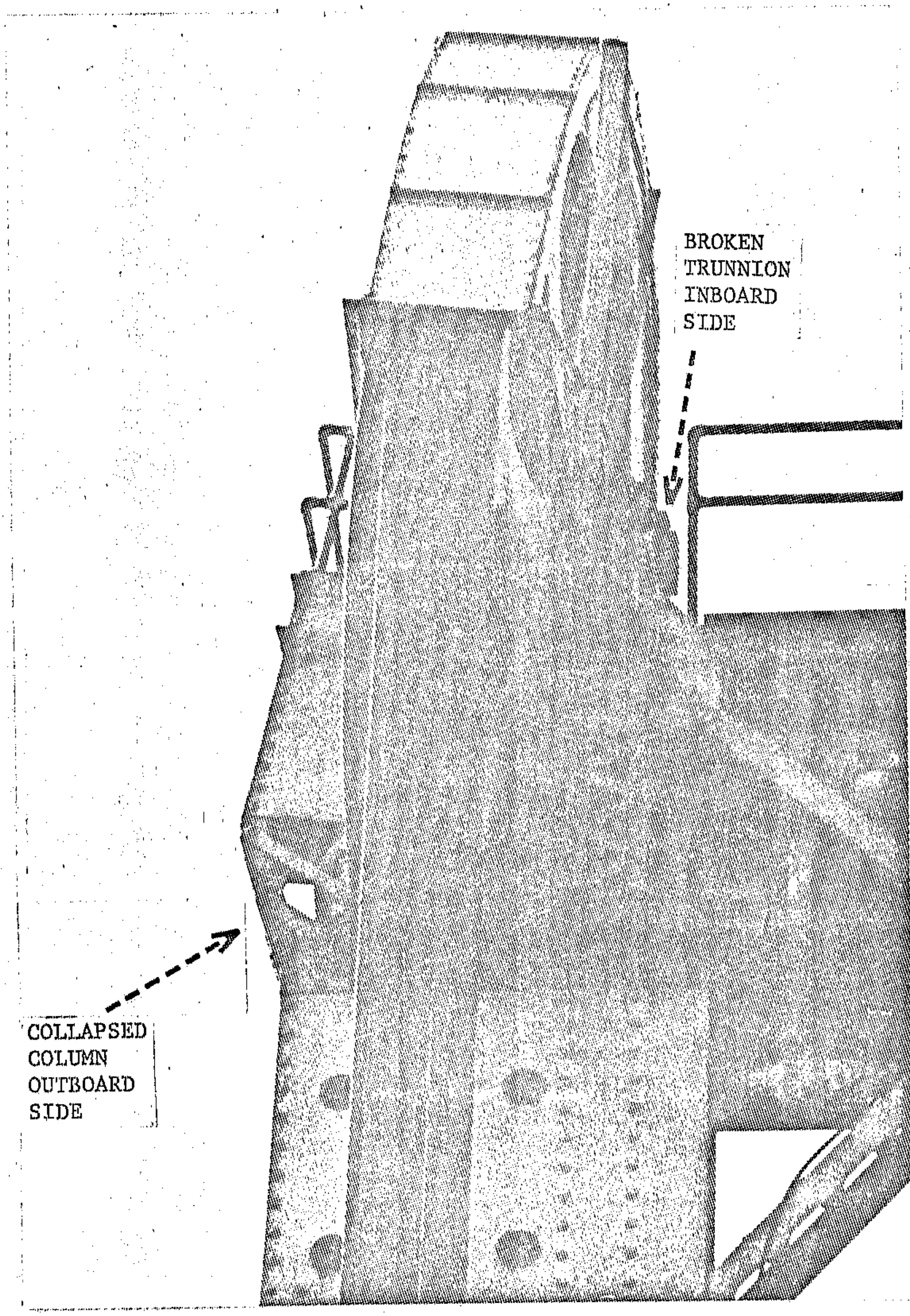


Figure 7. Overall view of trunnion failure.

Figure 8. View of fractured trunnion on inboard side of sheave.

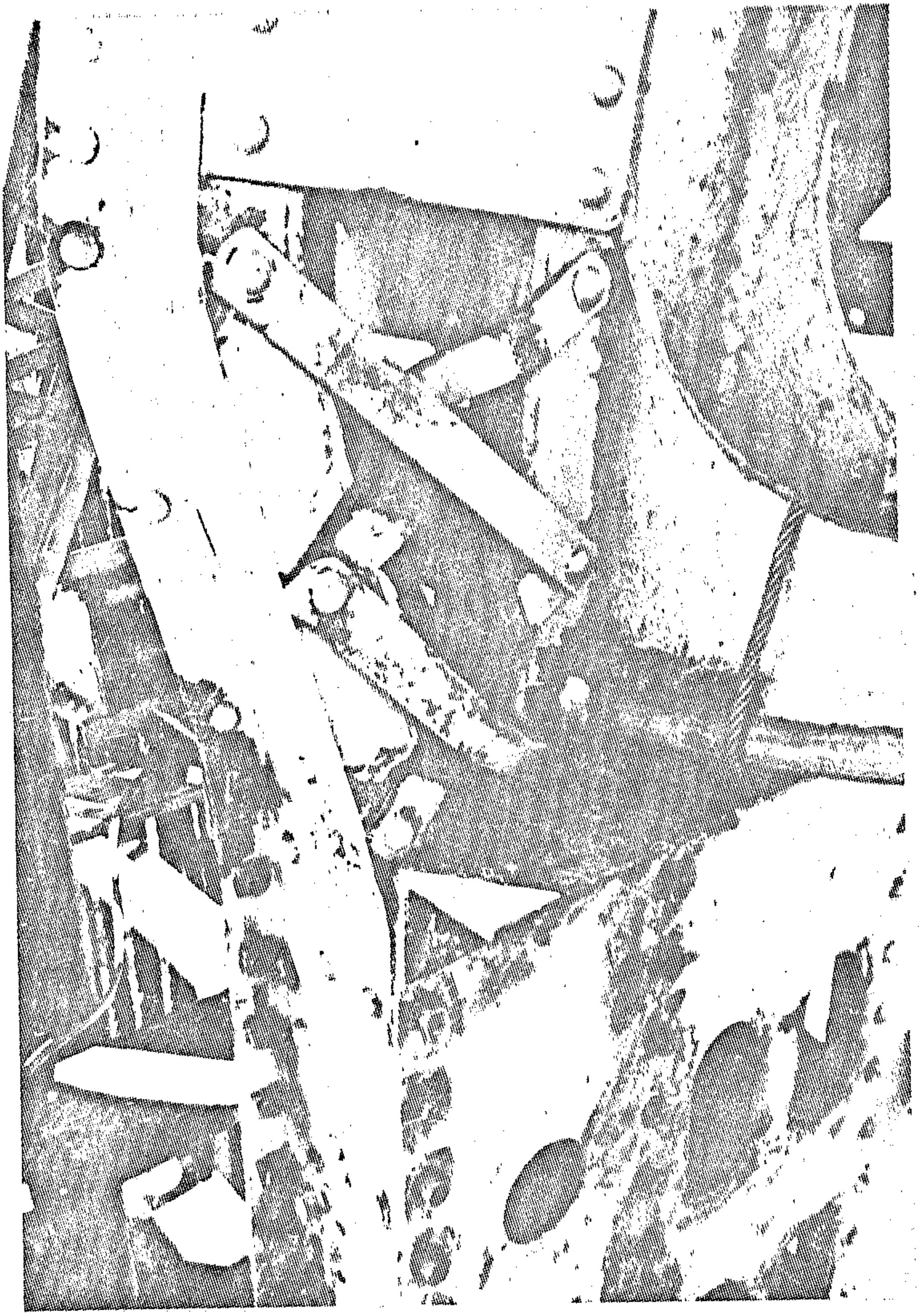


Figure 9. View of collapsed subpost on outboard side of sheave.

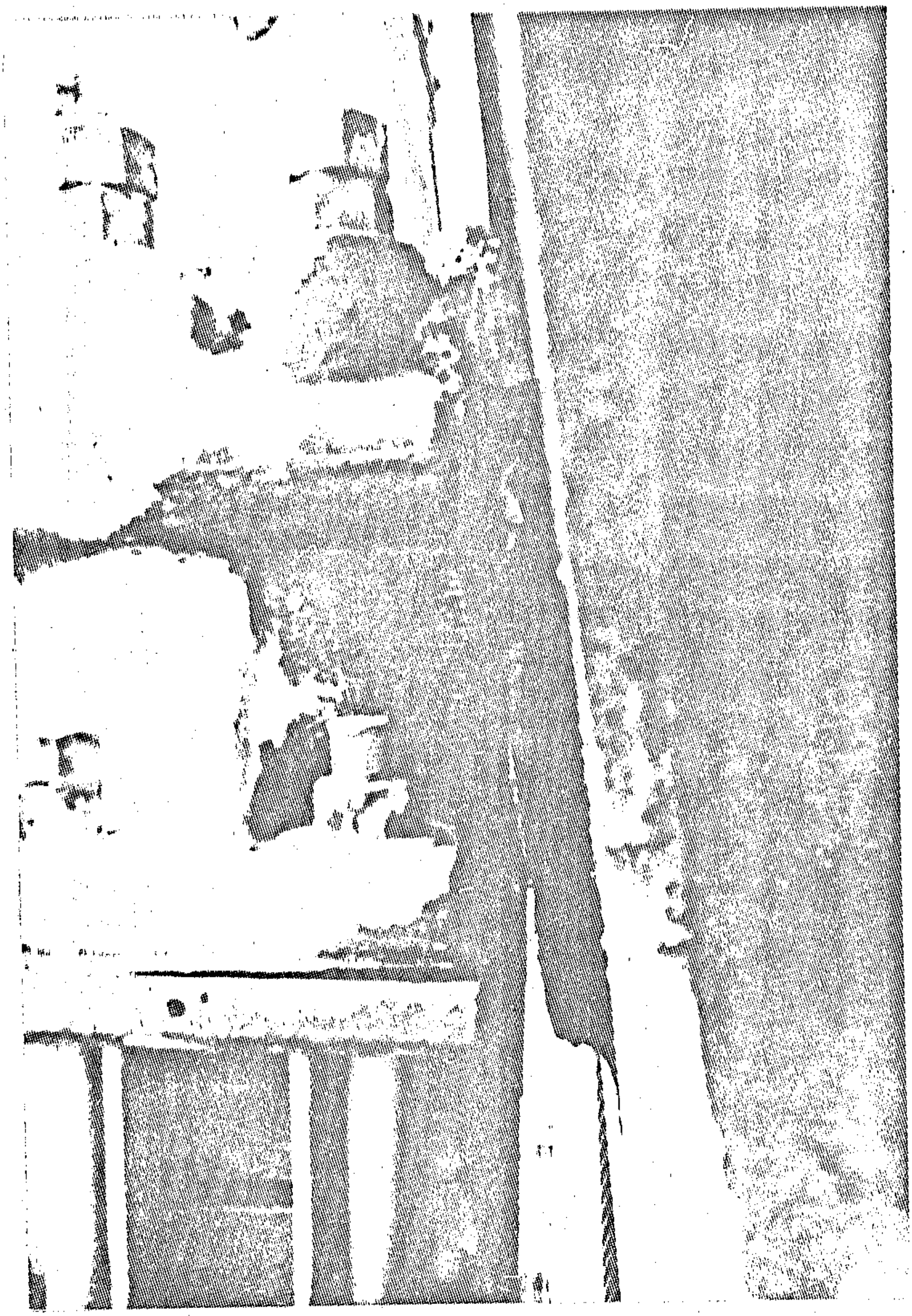


Figure 10. Side view of fractured trunnion resting on saddle.

vertical columns could withstand the side thrust of the 500 kip (2.2 MN) counterweight bearing laterally against the columns.

Mode of Failure

The type of failure encountered with the broken trunnion of the Shippingsport Bridge is a typical fatigue failure resulting from the member being subjected to repeated cycles of complete stress reversal (Figure 11). It is classical failure associated with progressive fracturing as induced by repeated loads. The failure starts with a minute crack and continually progresses until a final break suddenly occurs. The crack, which may occur at one or more points in a member, grows with repeated cycles of high stress at the edges of the crack (Figures 12 and 13). The crack is often induced at a point of high localized or concentrated stress such as in this case at an abrupt change in section (Figure 14).

A typical plot of stress concentration factors based on flat sections with uniform stress distribution is shown in Figure 15. The data are based on photoelastic studies reported by Frocht. Although the data used for plotting the curves may not be entirely representative of the true stresses developed across the fillet, they are somewhat indicative of the high localized stress conditions that are probably occurring within trunnions that are designed with an abrupt change in section. The curves show that the stress concentration factor also becomes highly critical when the p/d ratio is less than 0.1. For a fillet radius p equal to 1/2 inch (12.7 mm) the p/d ratio for the trunnion that failed is 0.05, which could result in a stress greater than twice the nominal stress across the reduced section. The plot also shows the effect of increasing the fillet radius to 7/8 inch (22.2 mm) where t = p and p/d = .0875. By increasing the fillet radius to the

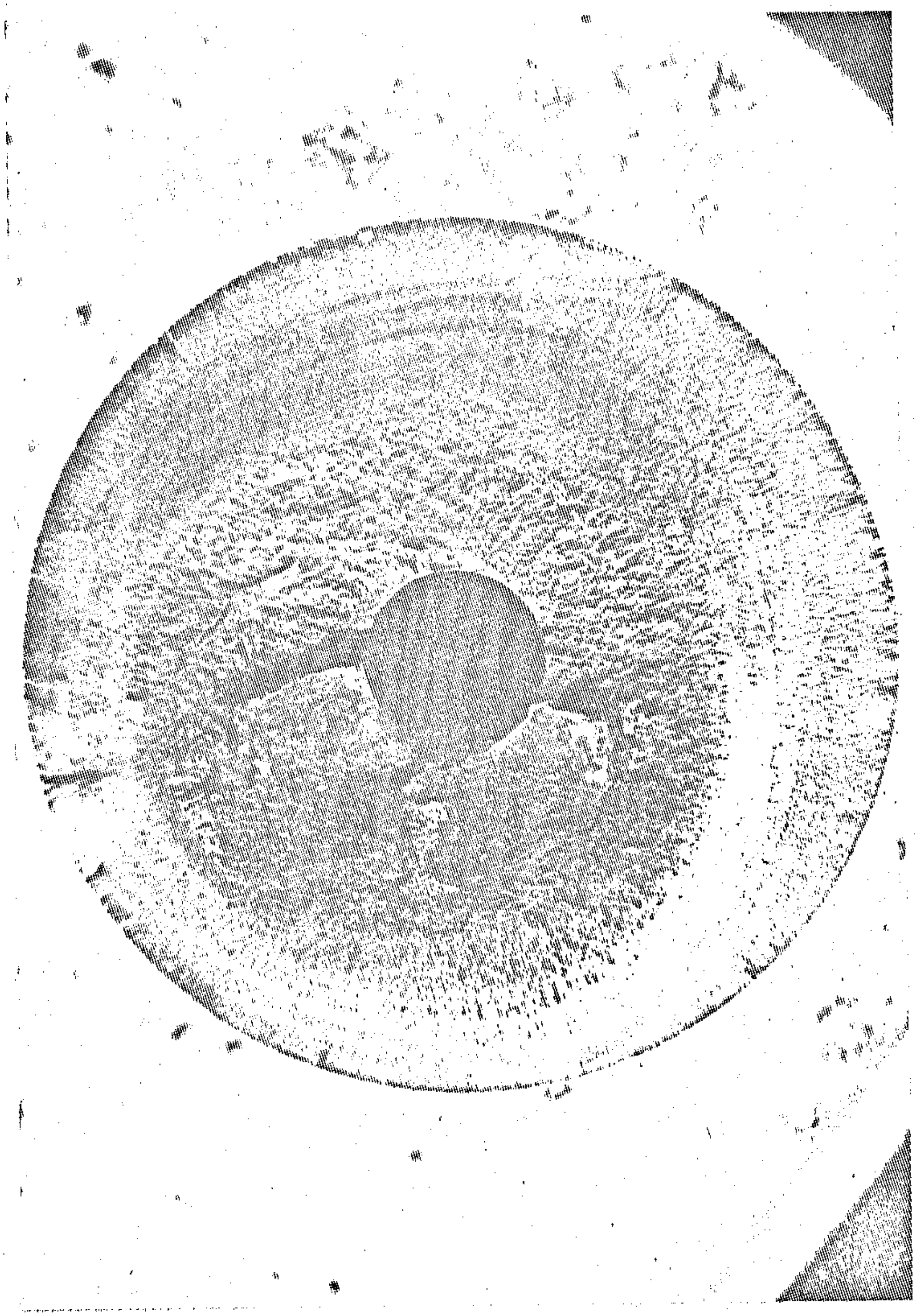


Figure 11. View of fractured trunnion.

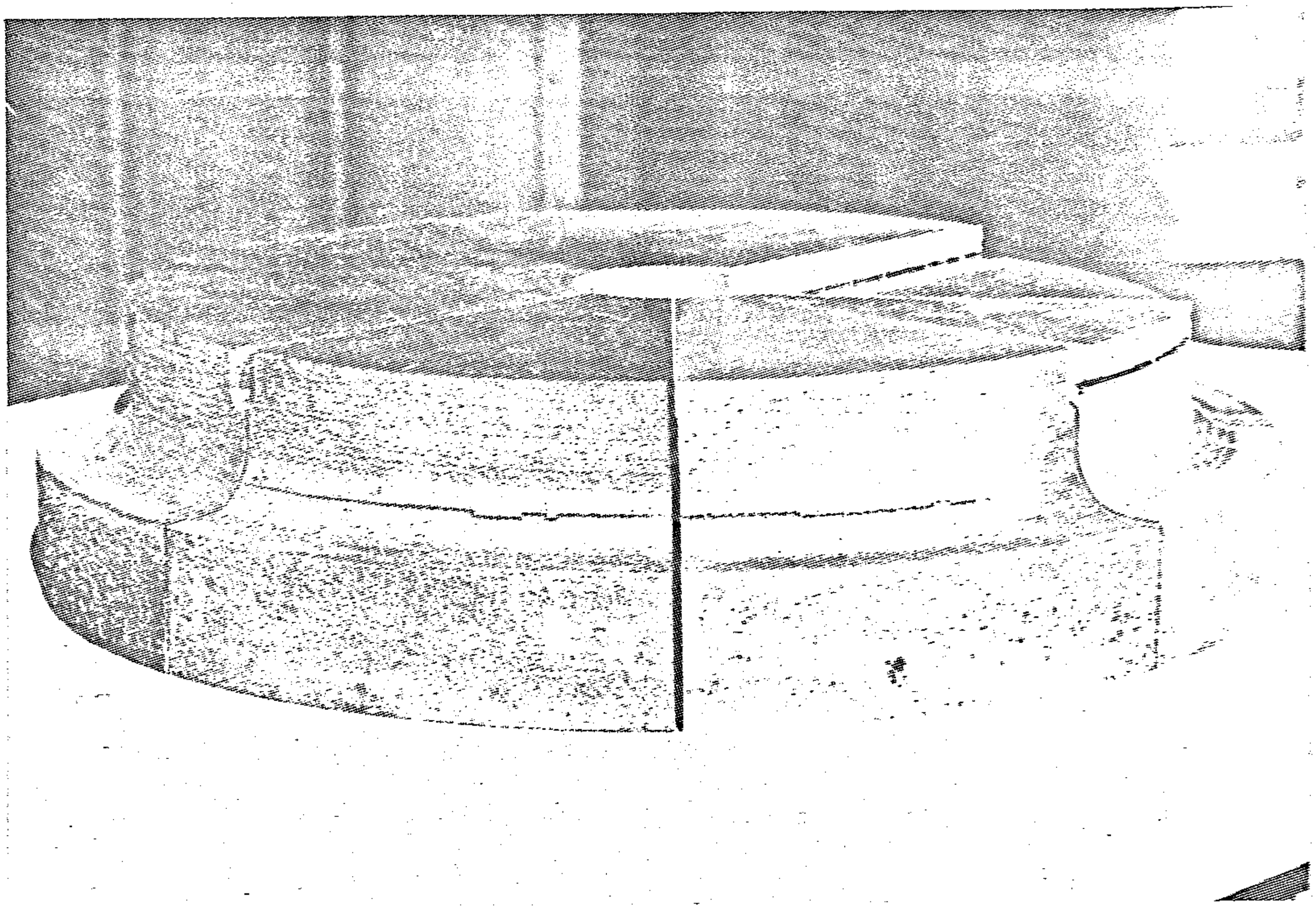


Figure 12. Cut sections of trunnion showing fatigue crack at opposite end of trunnion.

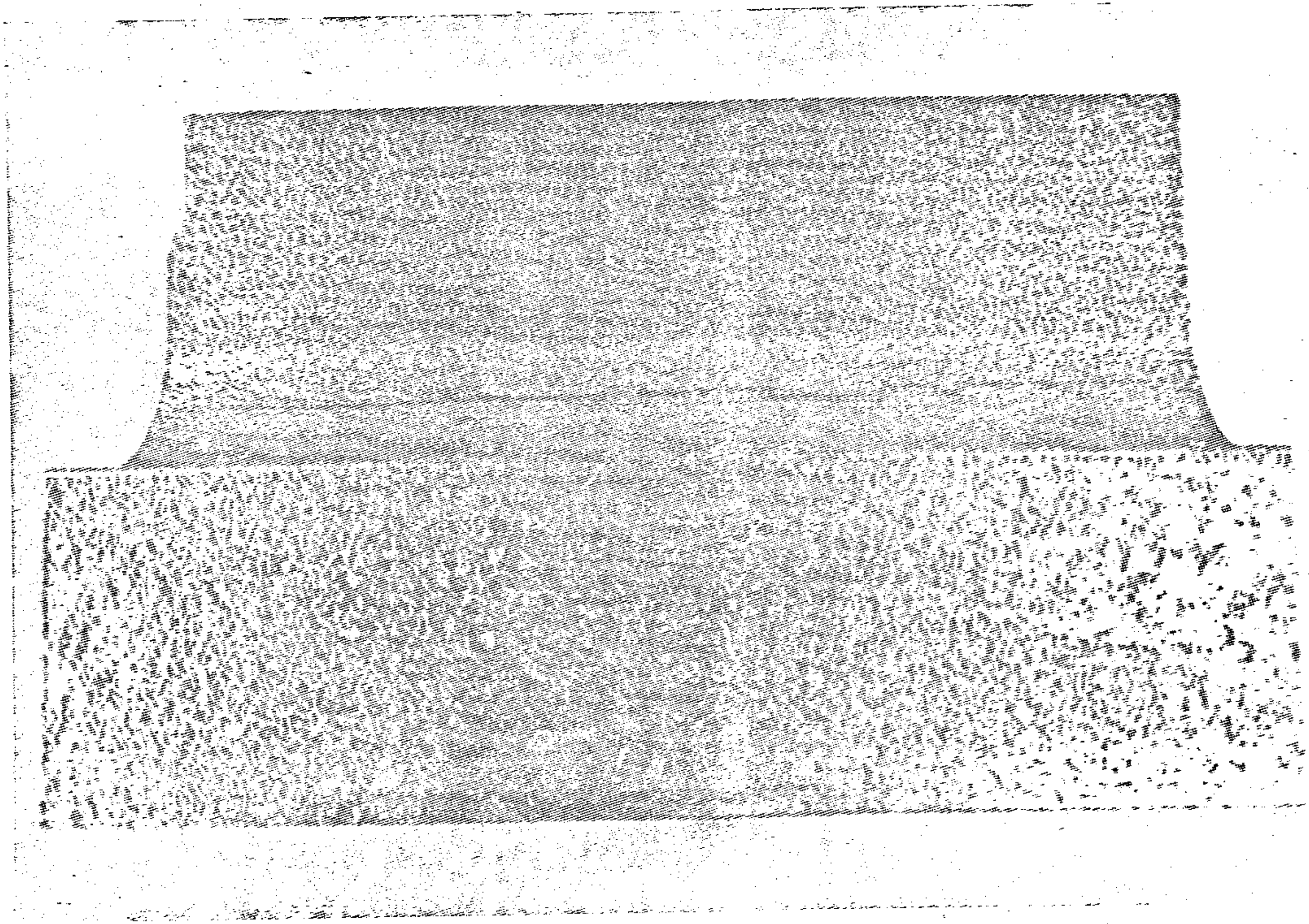


Figure 13. Close-up view of a cut section of trunnion showing fatigue crack at opposite end of trunnion.

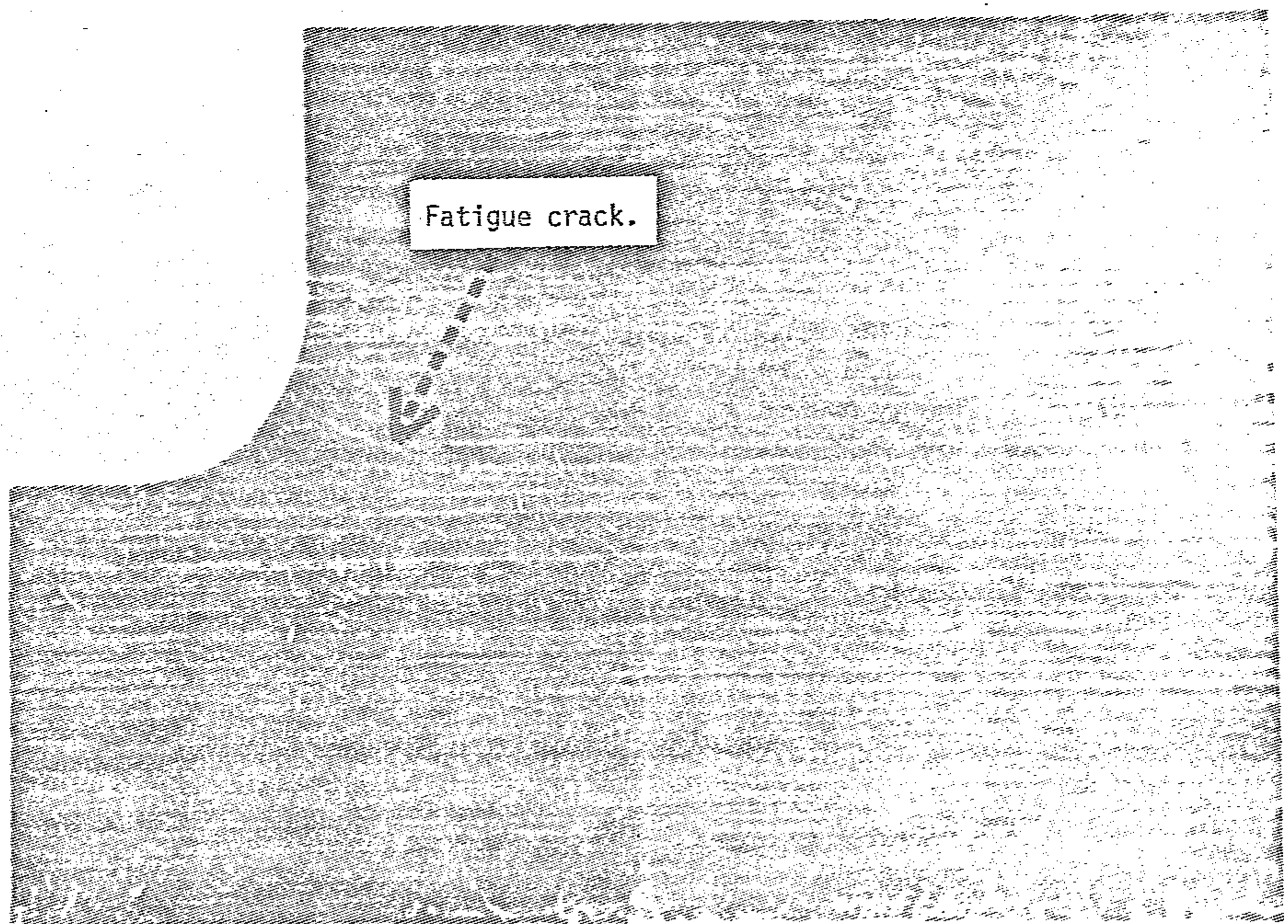


Figure 14. Side view of cut section showing penetration of fatigue crack.

Metric conversion: 1 inch = 25.4 mm

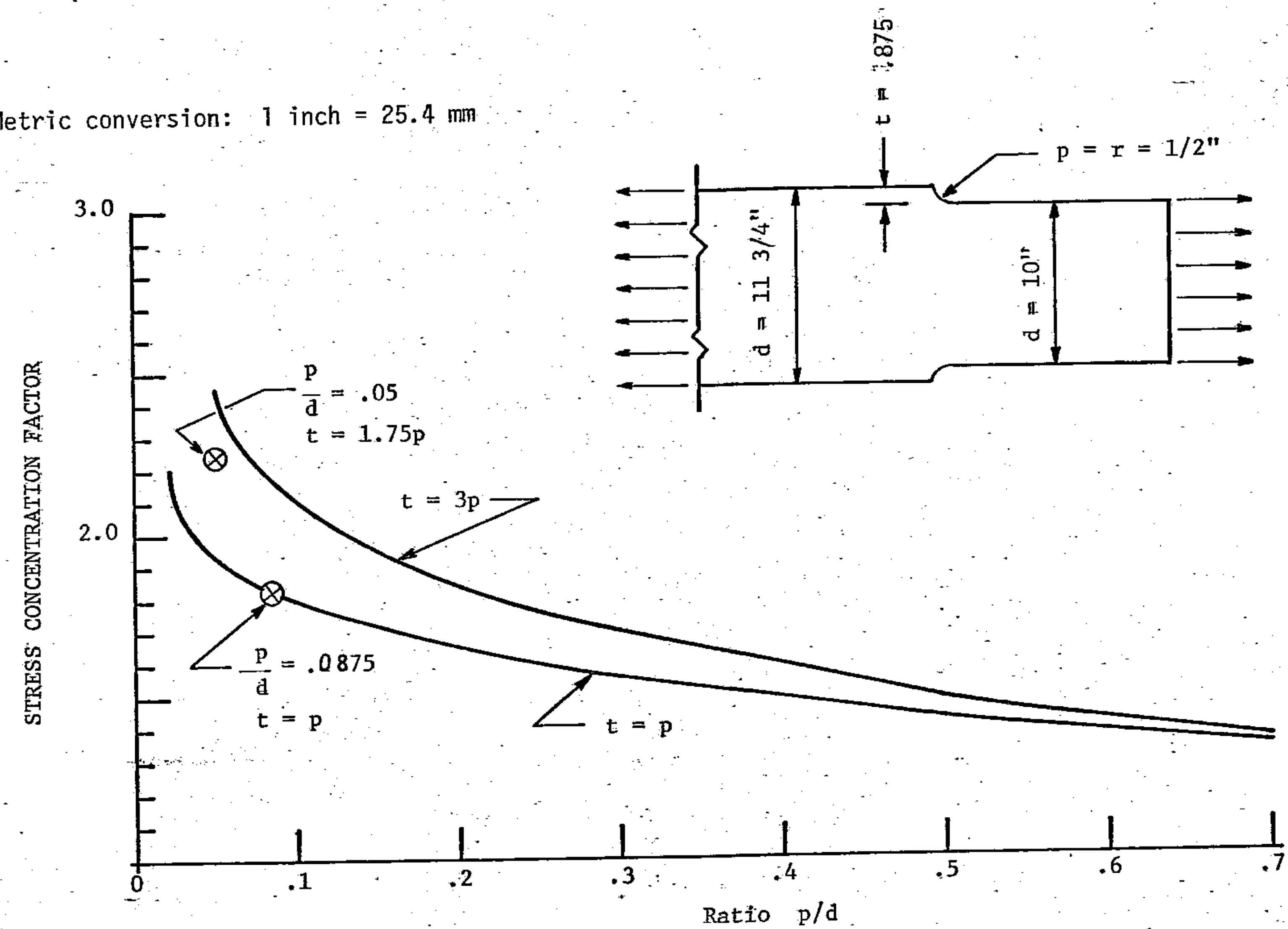


Figure 15. Theoretical stress concentration factors for fillets.

maximum that could be accommodated by the original configuration, the localized stresses within the fillet could be reduced by as much as 30 percent.

The effect of high stress reversals in relation to the fatigue life of mild steel is shown in Figure 16. Considering that the pulley is 12 1/2 ft (3.8 m) in diameter and that 55 ft (16.8 m) of vertical travel is encountered during a lift, the pulley would rotate 1 1/2 revolutions each time the span is either raised or lowered. Assuming that the lift span has been raised and lowered on an average of 15 times per day with 3 complete cycles of rotation occurring for each lift, over 600,000 cycles of complete stress reversal could have been accumulated over the past service life of 45 years.

The curve relating to the mild steel indicates that cumulative fatigue damage would occur with stress levels near 20 ksi (137.9 MN/m^2). With the stress concentration induced by the abrupt change in section, stresses well above the 20 ksi (137.9 MN/m^2) could be expected. The predicted fatigue life of high-strength steel is shown also in Figure 16. The curve for the higher strength steel shows a fatigue life of 2,000,000 cycles at stress levels between 20 ksi (137.9 MN/m^2) and 30 ksi (206.8 MN/m^2), whereas for the mild steel the fatigue life is about 600,000 cycles at the same level of stress. This relationship indicates that substantial increase in fatigue life at a ratio of about 3 to 1 could be achieved by utilizing a high-strength steel in the design of the trunnion.

Other Factors

The effect of certain loading factors not considered in the original design were also investigated to determine if these factors contributed to the failure. These factors included the effect of possibly unbalanced loads

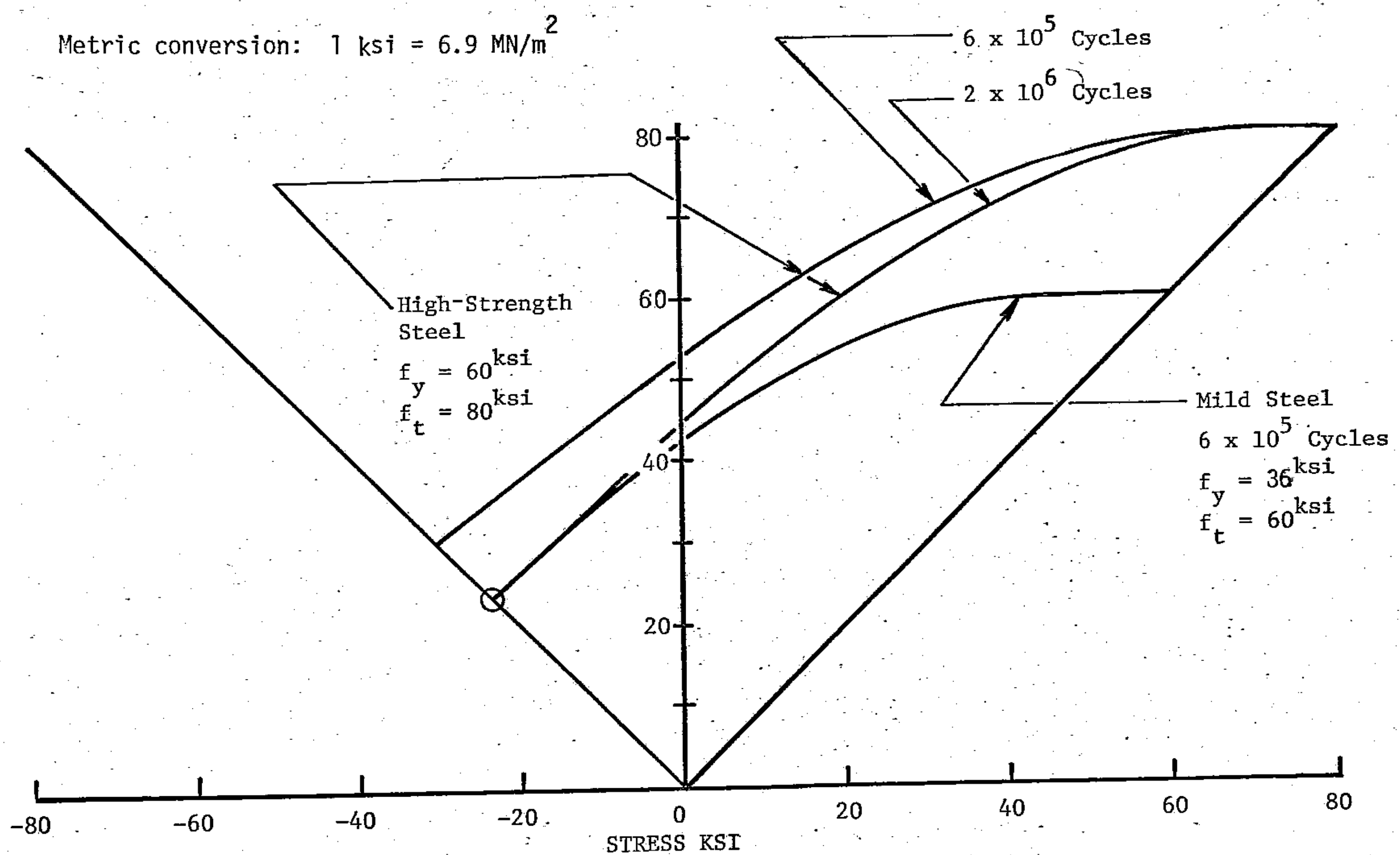


Figure 16. Fatigue strength of mild and high-strength steel.

in the cables supporting the lift span and the counterweights, the effect of increased loads induced while raising and lowering the lift span, and the effect of live loads or truck traffic crossing the bridge.

Verification of the loads in the cable suspenders was determined by the theory of harmonic motion. With the length, geometry, and unit weight identical for each cable, the relative load level per cable is directly related to the square of the natural frequency of each cable. An accelerometer (Figure 17) with auxiliary electronic signal-conditioning equipment was used to determine the natural frequency of each cable at one of the four support locations. Strip chart recordings of the accelerometer readings and the corresponding natural frequencies are shown in Figure 18. A summary of cable load factors based on differences in natural frequency is shown in Table 1. The maximum variation in load for a given cable was 18 percent. The data also indicate a slight unbalance in load across the pulley, which may have a slight effect in the bending stresses induced in the trunnion. An analysis of the load factors indicates about ± 6 percent variation in stress due to the unbalanced loading.

Strain gages were applied on two cables (Figure 19) at the same support location where the accelerometer measurements were taken. These gages were used to study the effect of live loads and loads induced while raising and lowering the lift span. A loaded truck with a gross weight of 72 kips (320.3 kN) was used as a test vehicle to evaluate the effect of live loads due to truck traffic. The test vehicle was operated at various speeds from a crawl to 30 mph. A typical strip chart recording of strain measurements at a test speed of 30 mph is shown in Figure 20. Analysis of the recording indicates about a 3 percent increase in load through the suspender cables

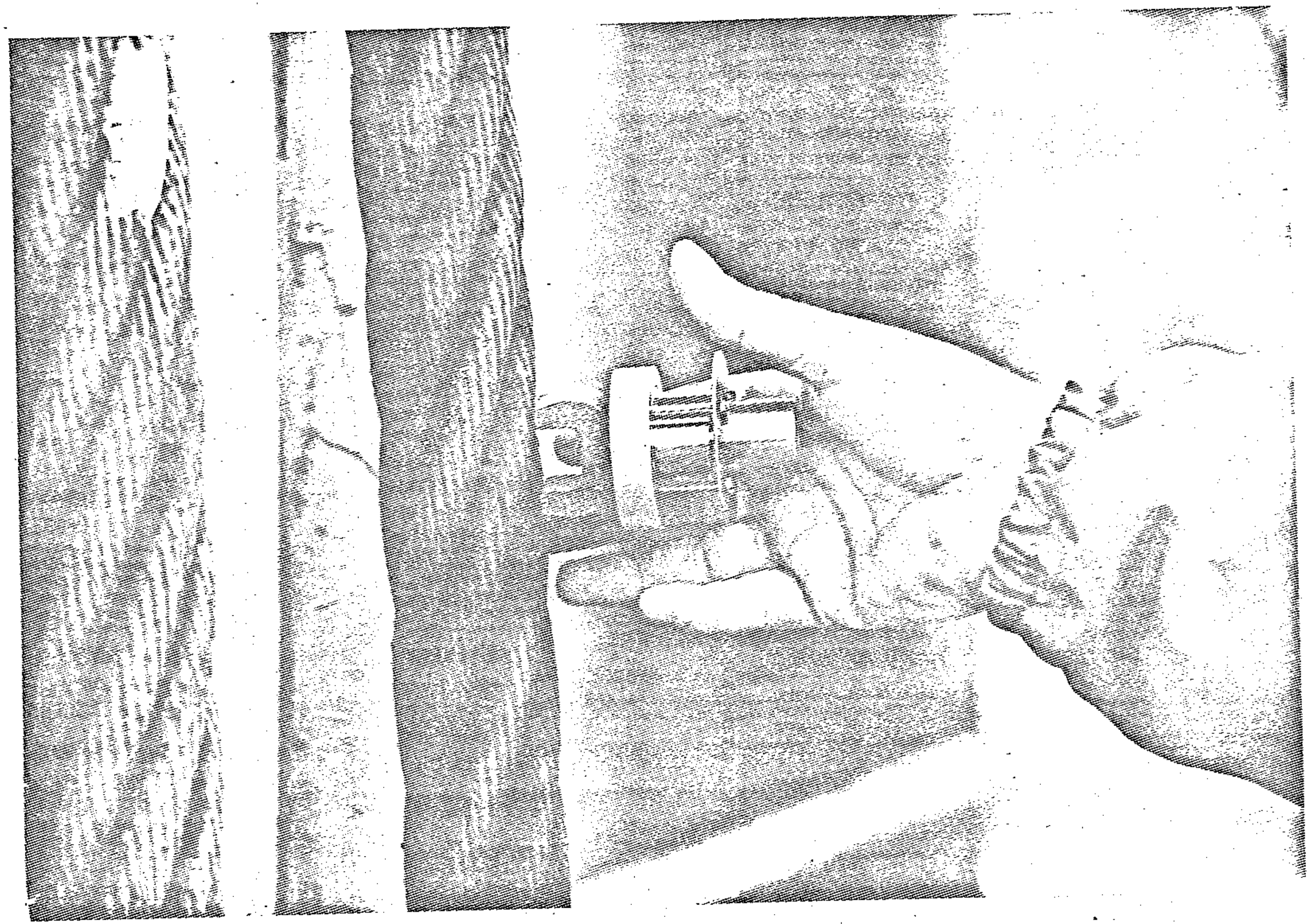
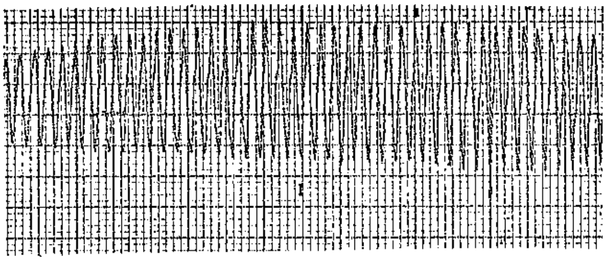
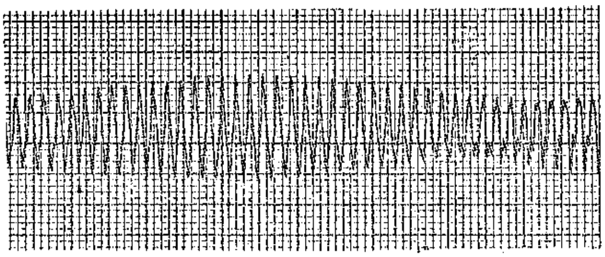


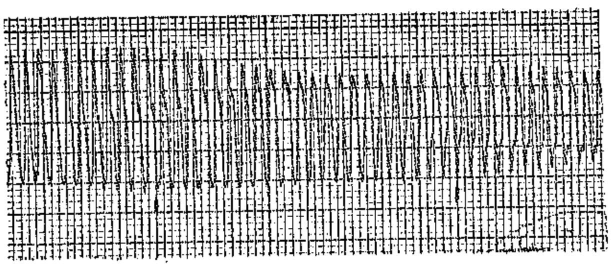
Figure 17. Accelerometer used for determining natural frequency of suspender cables.



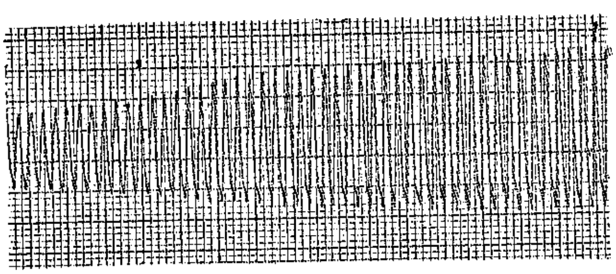
CABLE 1
f = 2.76 HZ



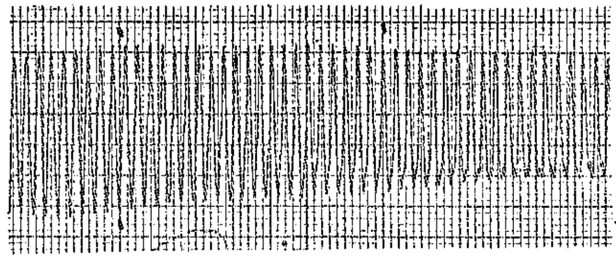
CABLE 2
f = 2.76 HZ



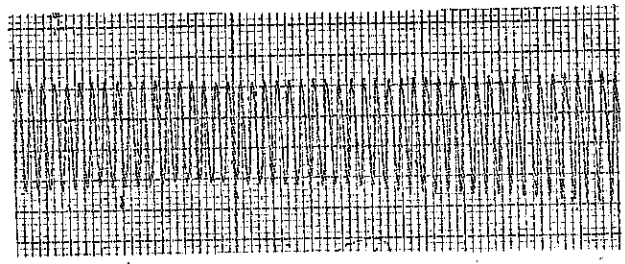
CABLE 3
f = 2.77 HZ



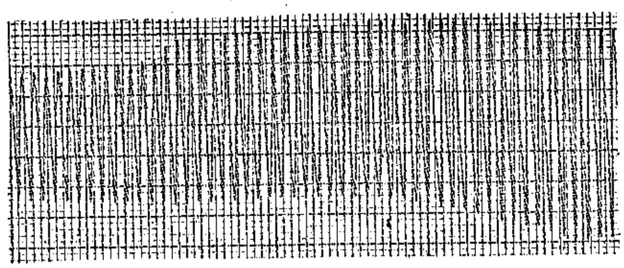
CABLE 4
f = 3.08 HZ



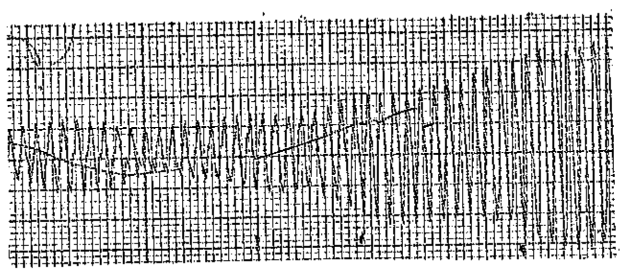
CABLE 5
f = 3.07 HZ



CABLE 6
f = 3.06 HZ



CABLE 7
f = 3.20 HZ



CABLE 8
f = 2.87 HZ

Figure 18. Strip chart record showing accelerometer measurements of cables at northwest tower.

TABLE 1

SUMMARY OF CABLE LOAD FACTORS

ILLINOIS ROUTE 351 OVER ILLINOIS RIVER

NORTHWEST TOWER

Cable No.	f HZ	f^2	Load Factor	Percent Deviation
1	2.76	7.62	0.87	-13%
2	2.76	7.62	0.87	-13%
3	2.77	7.67	0.88	-12%
4	3.08	9.49	1.09	+ 9%
5	3.07	9.42	1.08	+ 8%
6	3.06	9.36	1.07	+ 7%
7	3.20	10.24	1.18	+18%
8	2.87	8.24	.95	- 5%

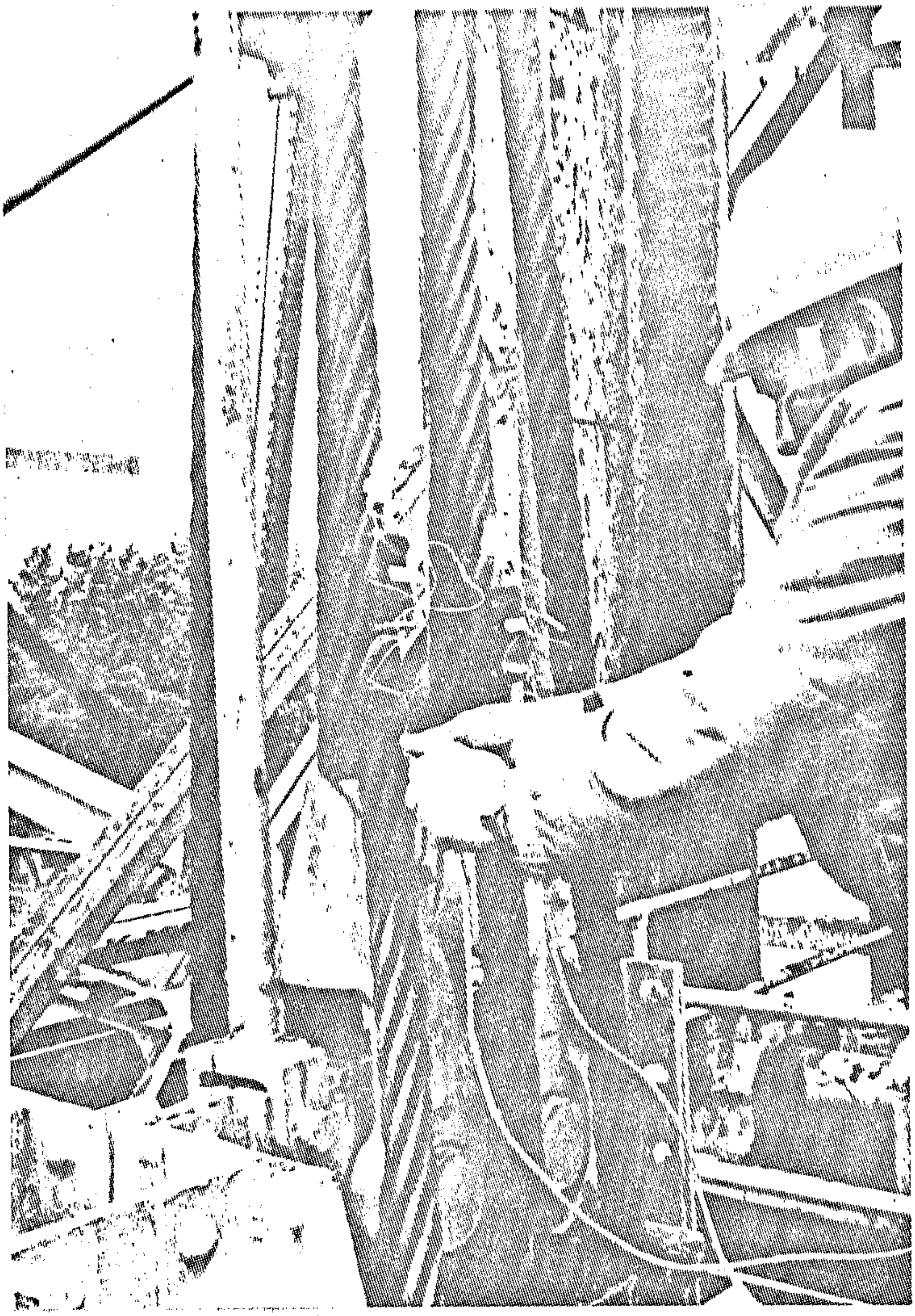
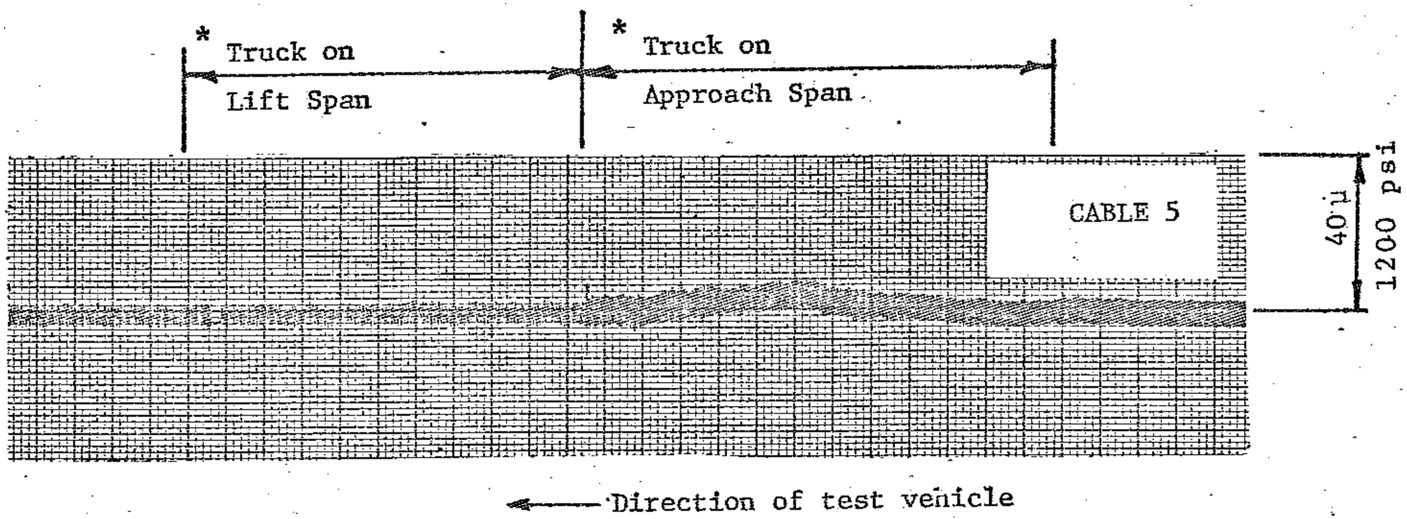


Figure 19. Strain gages attached to cable to determine the effect of live loads.

Metric conversion: 1 psi = 6.89 kPa
1 kip = 4.45 kN



* 3S-2 Truck (Gross Weight = 72 K)

Figure 20. Strip chart record showing strain measurements during live load test.

while the test vehicle crossed the approach span which supports the lifting towers. The recording also shows little or no effect with the test vehicle on the lift span.

Strain measurements were taken during the normal operation of raising and lowering the lift span. Figure 21 shows a typical strip chart recording of measured strains induced while raising the lift span. The data indicate about a 4 percent increase in load under normal operation of raising or lowering the lift span.

Metric conversion: 1 psi = 6.89 kPa

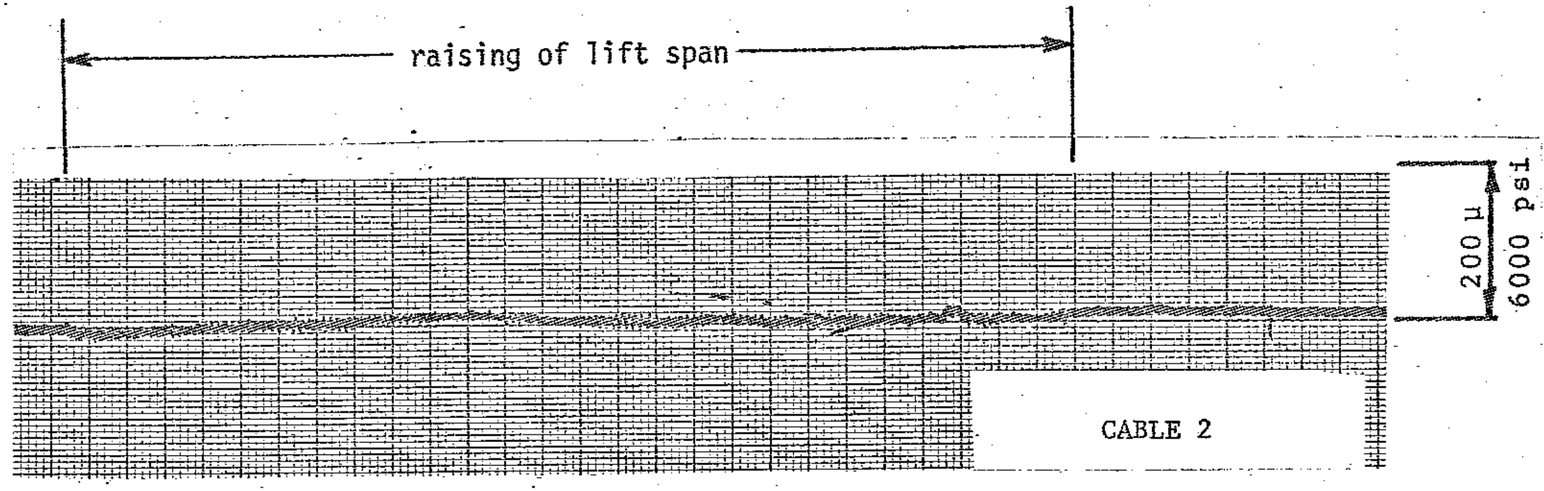


Figure 21. Strip chart record showing strain measurements when raising the lift span.

SUMMARY AND CONCLUSIONS

The mechanics of failure associated with the fractured trunnion on the Shippingsport Bridge were induced by propagated fatigue cracking which became critical after about 600,000 cycles of complete stress reversal. The major contributing factor initiating the failure was the development of high concentration of stresses in the critical fillet areas resulting from an abrupt change in section in the configuration of the trunnion.

The effects of other factors not normally considered in the design of trunnions were considered in the investigation. A study of these factors indicated possible increases in stress of 6 percent due to unbalanced loads in the cable suspenders, 4 percent due to the normal operation of raising or lowering the lift span, and 3 percent due to live load. The accumulated effect appeared to be from 9 to 13 percent, depending on whether a loaded truck was on the approach span at the time of operating the lift span. A variation in stress of this magnitude is not unreasonable and would be normally considered acceptable when comparing actual stresses to theoretical. Also, under normal circumstances, an adequate factor of safety is provided to accommodate additional stresses of this magnitude without reducing the service life of the structure.

Two approaches were considered in the redesign of the trunnions. One approach involved refitting the trunnion supports with new bearing blocks and saddles that would accommodate a straight shaft with no change in section. However, to economize the repair cost and to minimize the time required for the remedial work, a decision was made to use the present configuration with modifications that would extend the service life of the trunnion. Past studies of fracture mechanics indicate that a substantial improvement in fatigue life could be achieved by increasing the size of the fillet and by incorporating a higher strength steel.

Cracks of an identical nature were also found in other trunnions within the same structure and also three other structures under maintenance by the Illinois Department of Transportation. The frequency by which fatigue cracks have developed in the trunnions of the Shippingsport Bridge and other lift bridges within the state is alarming. An in-depth inspection of the four structures revealed evidence of severe fatigue damage in over 50 percent of the trunnions within the structures. Considering the number of movable bridges in service throughout the country, the potential for a catastrophic failure due to a fractured trunnion should be recognized by engineers and technicians responsible for maintaining and inspecting these bridges. In regard to movable bridges, a thorough inspection should be made of trunnions that have incorporated an abrupt change in section, especially in cases where the trunnion is subjected to more than 90° of rotation under normal operating conditions.