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WORKSHOP NOTES

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"Rehabilitation of Three 70 Yr Old
Bascule Bridges" Russel S. East,
Sverdrup Corp., Bellevue, Washington

ACKNOWLEDGEMENTS

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INTRODUCTION

In 1986, the City of Seattle Engineering Department began to evaluate the condition of the mechanical systems for three of the City-owned bascule bridges over the Lake Washington Ship Canal. University, Fremont, and Ballard Bridges, all built prior to 1920, were selected for the program. In the beginning, the program focused strictly on the condition of the existing mechanical systems of the bridges, and consisted of a condition inspection and evaluation of the bridges' drive machinery. As the inspection program progressed, and funding became available for engineering, the program expanded from an inspection/evaluation program of the drive machinery, to an expanded design program whereby the design of several bridge systems, including bridge deck grating, truss protection guardrail, centerlocks, buffer cylinders, live load shoe rehabilitation, span drive machinery replacement, control system replacement, new span balancing, and a temporary winch span drive system were completed.

Design challenges ranged from the simple to the complex. Simple problems, such as assembling live load shoes using the proper procedure to keep shims in place, and more complex design problems such as maintaining shipping traffic through the bridges while the span drive machinery and bridge control systems were replaced without a major disruption of vehicular traffic on the bridge deck. These problems and many more were overcome during the comprehensive bridge rehabilitation project.

Since the project funding came from limited capital funds from the City of Seattle Engineering Department, primary designs were completed for all the bridges, and the construction work was prioritized so that the most urgent work could be completed early in the project and the lower priority improvements could be postponed until funding was available. Upon completion of the first phase design, buffer cylinders were fabricated and installed on the University Bridge. Subsequent to installing the buffer cylinders on the University Bridge, the live load shoes were rehabilitated, and the span drive machinery and control systems were replaced. To date, the only work completed on the Fremont Bridge is the buffer cylinder installation. Ballard Bridge has not had any work performed as of this date. This paper will deal primarily with the rehabilitation design work for the University Bridge in Seattle, Washington (Figures 1, 2, and 3).

LIVE LOAD SHOE REHABILITATION

The City of Seattle Bridge Maintenance Department had a history of frequent shim adjustments of the live load shoes on the University Bridge. The maintenance crews for the City of Seattle followed a procedure for re-shimming the live load shoes that they had followed for years. The procedure that the City employed was a long-standing procedure with the City's Bridge Maintenance Department, and thus was followed for years. As noted in Figure 4, they typically wired shim plates to the top of the casting, and monitored the shims so that the shims could be re-secured and/or replaced if they were lost or became loose.

The City recognized the problem, and directed the consultant team to provide a solution. Upon review of the original design drawings, it was apparent that the live load shoes were originally constructed in a way to allow for a more permanent shimming procedure. The shimming procedure that made best use of the existing design required that the upper landing plate be removed, cleaned, epoxy-filled, and finally reinstalled. As noted in Figure 4, the space below the lower landing plate is fabricated in a way that shims could be placed in the area and left unattended for years at a time. Since the rehabilitation shimming was completed in 1989, there has been no need to adjust the shimming.

A second benefit of the live load shoe rehabilitation was the improved rigidity of the bascule structure. Throughout the years, the live load reactions at the back side of the counterweight box had worn and deteriorated to the point that they no longer made firm contact bearing with the anchor pier when the bridge was fully closed. Without firm contact at the anchor pier, the bascule span was being stabilized by the live load shoes and the trunnion, rather than the live load shoes and the live load reactions. The result of this narrow base (13 feet) was that the bascule span was quite live and bouncy under heavy bus and truck traffic. When measured at the live load reactions, the vertical deflection was observed to be approximately 1/8". This corresponds to 3/8 inches at the tip of the leaf.

More significant than the excessive span deflections was the fact that the trunnion was exposed to loads it was not designed for. This is potentially hazardous if span balance and geometry is such that the net load at the trunnion is uplift under live load, since most trunnions are not designed for this load condition. Typically, designs of bascule bridges designed in the early 1900's relied on the live load reactions at the back side of the counterweight box to resist overturning loads and assumed that the trunnion carried no live load.

After the live load reactions were re-built and shimmed for full bearing when the bascule span was closed, the stiffness of the span was quite remarkable. Instead of an overturning base of 13 feet measured between the live load shoe and trunnion, the overturning base measured 35 feet between the live load shoe and live load reactions, a significant improvement.

Two important design concepts practiced on the University Bridge were:

1) Proper shimming of the live load shoes requires disassembly of the casting, and placing shims between the formed plates to retain and protect the shim assembly.

2) In order to properly complete any live load shoe rehabilitation, the live load reactions at the rear of the counterweight box should be shimmed for full contact bearing when the bascule span is fully closed.

TRUSS PROTECTION GUARDRAIL

When the City of Seattle bascule bridges were originally designed prior to 1920, automobile traffic was in its infancy, travel was by streetcar trolleys, and high speed, heavy trucks and buses didn't even exist. Again in the 1930's when the University Bridge was rehabilitated the first time, and the City added outrigger lanes to the bascule span, traffic wasn't anything like it is today. When one considers the state of automobile traffic prior to World Was II, it is easy to understand the lack of, or simple, guardrail systems employed on the bascule span for the University Bridge. The old guardrail on the University Bridge was not an AASHTO type guardrail, but was a flexible energy absorbing proprietary guardrail that was attached directly to the vertical members of the "thru-deck" bascule truss. Although in principle the flexible rail was an excellent guardrail to absorb impact energy, the rail relied on the bascule truss for structural support, and if impacted, the rail would transfer the impact load directly to the truss structure the guardrail was intended to protect.

In order to overcome the concern of the truss protecting itself, an AASHTO type guardrail was developed that was not framed directly to the bascule truss, but was framed into a beam system that was in turn supported by the floorbeam system (See Figure 5). The primary advantage of this type of system lies in its ability to absorb significant impact energy without adversely affecting the vertical truss members. The final design of the guardrail system allowed for 5 inches of deformation of the guardrail without impacting the truss members.

During the design development of the truss protection guardrail, it was important to design to the above noted parameters while providing a cost effective structure and minimizing the weight of the guardrail. Weight on movable bridges affects both the mechanical and structural systems, therefore, it is a prime consideration to review the practicality of using light weight materials on the movable portions. When the option of lighter weight high strength steel was considered, the quantities of steel required for the guardrail were used in the tubular guardrail section but not for all materials, since the remaining materials were not readily available in the necessary quantities.

BRIDGE DECK GRATING

The University Bridge was originally constructed with a timber deck and rails for streetcars. In the mid-1930's (circa 1933), the City undertook a major rehabilitation program whereby the timber deck was replaced with 2 1/2" deep riveted steel deck grating supported on 6-inch deep channels (Figures 6 and 7). The original deck grating has been periodically repaired, but never replaced and has provided more than 50 years of satisfactory service to the City of Seattle. Throughout the life of the grating, and especially during the last 25 years, City maintenance crews have periodically made weld repairs to cracked bearing bars, welds, and support channels.

During the initial bridge condition inspection period, the City of Seattle secured funding for a thorough condition inspection of the bridge deck grating. The inspection was intended to catalog important load carrying data such as grating wear, weld cracks, grating cracks and fractures in support channels.

The inspection was planned so that a detailed report of major defects, grating wear, and weld repairs could be cataloged for the bridge deck. The predominant wear and fractures were located at the joints between the fixed and movable spans, and at the center joint between the two movable spans. Even more so, the wear and fractures were concentrated in the tire paths of the right lanes of both the north and southbound lanes. As shown in Figure 9, a large portion of the weld repairs were concentrated in the right-hand lane of the first and last "bays" near the fixed roadway and center joints.

Similar to the fractures mapped in Figure 9, the deck grating wear patterns exhibited remarkable similarity. The predominant wear was near the roadway joints, and reached a maximum of 1/2 inch under tire paths at the floorbeams, and typically averaged 1/4 inches throughout the bridge deck.

Upon completion of the condition inspection of the bridge deck grating, the decision was made to replace the deck grating and support channels. The selection of a new grating had to meet four criteria: 1) The grating must have the capacity and fatigue strength to carry the design loads (HS20) for 1.5 million cycles without fracture or failure; 2) the grating must have skid resistance better than or equal to the riveted bar grating currently on the bridge; 3) the weight of the new assembly should not overload the existing bridge support structure, nor should it appreciably change the wind loads with the span open, or shift the center of gravity, or change the "Wr^2" term for the bascule span; and 4) the total depth of the new grating assembly should not differ from the original grating assembly by more than 1". The grating assembly that met all of these criteria was 5-inch 4-way HD grating supported on W4x13 cross beams (Figure 8).

Typically, conventional riveted bar grating has better fatigue properties than 5-inch 4-way grating. The primary concern with most welded grating, such as 5-inch 4-way HD grating, is that the welds do not stand up to wear resulting in lost section modulus and significant fatigue loads. There are however, manufacturers that provide a "groove"-type puddle weld at the bar intersection point. By notching the cross and diagonal bars, they are able to achieve a penetrating weld that is more fatigue resistant than that provided by other manufacturers that choose not to notch the intersecting bars. By selecting heavy-duty 5-inch 4-way grating, and limiting the span to 48 inches maximum, the user can achieve acceptable performance. The inclusion of the notch prior to welding in conjunction with selection of heavy-duty grating gives the depth and weld size required to resist fatigue fractures.

In addition to the "notching" in preparation for groove welding, another procedure used to improve the fatigue resistance of grating was to hot-dip galvanize the grating after fabrication. It is interesting to note that the 850°F temperature attained during hot-dip galvanizing is an informal means to stress relieve the welds. Studies performed by grating manufacturers and others have determined that although galvanizing should not be considered as a formal means for stress-relieving welds, it is effective in improving the fatigue resistance of 5-inch 4-way HD grating, and also providing corrosion resistance.

Skid resistance of various grating products have been tested per ASTM E-274. Although ASTM E-274 is intended for use on pavements, it gives a relative comparison for comparable gratings. Recent skid resistance data of new 5-inch 4-way HD grating indicates that the skid resistance of 5-inch 4-way grating may be better than that of riveted bar grating products. The most likely reason for this is that

5-inch 4-way HD grating has more contact surface and more corners and edges per unit area than other grating, and thus, the surface and edges provide better grip to the tires than conventional riveted bar grating. Typically bar grating products have tested with skid resistance from 17 to 39, and a skid resistance number of 35 is considered the lower bound for safety. However, 5-inch 4-way typically tests above 35.

Other systems were considered for providing superior skid resistance to the bridge deck grating, but each had a drawback that made it unacceptable. Abrasive granules such as aluminum oxide can be bonded to the surface of grating but they wear off quickly and become ineffective. Weld studs provide a reasonable increase in skid resistance, but they too wear down and become ineffective. Concrete fill is particularly effective but heavy, and the concrete fill wears down in time and the grating skid resistance decreases. Solid decks such as an Exodermic Bridge deck are effective, but the solid deck increases wind loads on machinery and the deck is also extremely heavy at 53 psf. Based on these reasons, plus the fact that the bridge structure and machinery could tolerate the added weight, 5-inch 4-way HD grating, fabricated from A36 steel and galvanized, was selected as the best replacement grating for the University Bridge. Due to funding shortfall, the grating has only been purchased for the bridge, but has not yet been installed.

STRUCTURAL CAPACITY

Throughout the University Bridge Rehabilitation Project, the City of Seattle Engineering Department requested a number of improvements be incorporated into the bridge structural and mechanical systems. The improvements included new bridge deck grating, new truss protection guardrail, and a new spanlock system. Each item imposed additional dead loads on the bascule structure that were not included in the original design. Consequently, the dead load of the bridge increased substantially. The new 5-inch 4-way HD bridge deck grating added approximately 7 psf to the span. The original 8-3/4 inch timber deck structure weighed approximately 39 psf, the existing 2-1/2 inch riveted bridge deck grating weighs 21 psf, and the new 5-inch 4-way HD bridge deck grating system weighs 28 psf. Each leaf measures 4,000 square feet, and the weight on each span ranged from 84,000 pounds/leaf for 2 1/2 inch bar grating to 156,000 pounds/leaf for the original timber deck.

In addition to the bridge deck grating, the truss protection guardrail was added to the span. The original truss protection guardrail consisted of lightweight sheet metal products, and added very little weight to the span.

However, the new truss protection guardrail added significant weight, approximately 130 pounds per linear foot to each bascule span.

The most significant weight added to the span was due to the addition of new span lock machinery. The new "clamping" spanlock machinery was a different configuration than the original spanlock system and weighed substantially more than the existing "pin and socket" span-locks. The net effect of the added spanlock machinery was an additional 15,000 pounds added to the tip of the north leaf to balance machinery and inspection platform, and 1,500 pounds added to the tip of the south leaf to balance the weight of the protuberance and associated structural supports.

Upon investigation of the structural capacity of the University Bridge, the investigation revealed that the capacity of the structure was substantially more than that required to carry the original dead load plus current AASHTO HS20 loading. The structural capacity of the bascule truss was so much more than that required, that all three of the improvements could be made without structurally upgrading the main bascule trusses. The only structure improvements required due to the new weight were rebalancing the span. Rebalancing was accomplished by supplementing the counterweights with added mass. A total of 148,000 additional pounds were required for the north leaf, and 66,000 additional pounds were required for the south leaf.

The primary lesson learned in this phase of the project is that on older bascule bridges originally designed with heavy timber decks and for heavier streetcar loading, can be upgraded to carry additional dead weight without upgrading the main structural systems. In the case of the Fremont Bridge, the only structural work that needed to be completed was reinforcement of the counterweight box.

TEMPORARY WINCH SPAN DRIVE SYSTEM

The University Bridge is a double leaf bascule bridge and relies on independent drive machinery systems to operate each leaf. Each leaf is independently operated and both leaves must be opened to allow passage of ship traffic. This type of operation presents a considerable challenge during any type of major machinery rehabilitation work. During a major rehabilitation of an existing movable bridge over a busy waterway, the primary concern from the waterway users and public is to keep the bridge open to waterway and vehicular/pedestrian traffic. Rehabilitation of the University Bridge was no different. Since the span drive machinery contained severe cracks in the original gears, the City was forced to act and replace the machinery before the gears failed and caused a major disruption in the span operation, or precipitated a major failure where the bascule span slammed close without any braking force, and caused a major closure of the bridge span.

Designing the new span drive machinery systems for replacement of the old, presented several challenges; however, maintaining ship canal traffic and vehicular traffic during the replacement period was the most significant. Three options were considered for replacement of the drive machinery. The first option required that the bascule spans be locked in either the up position or down position for the duration of the replacement period. The second option was to drive the spans using "single rack operation", and the third option was to design an alternate temporary span drive system. Since the University Bridge has over 6,000 openings per year, the Coast Guard and waterway users were opposed to leaving the bridge down for the duration of the project; and since the University Bridge is part of a major north-south route through the city, leaving the spans up, closing the road to vehicular/pedestrian traffic, and causing gridlock in the University district of Seattle was also an unacceptable solution. Obviously, the only acceptable solution to the problem was to maintain the operability of the spans throughout the machinery replacement period.

In order to maintain operability of the spans throughout the machinery replacement period, the second and third options were evaluated. However, the option to use the existing span drive machinery in a single rack mode increased the operational loads on an otherwise defective drive system. That left the only viable alternate to maintaining an operable bridge as providing a temporary span drive system independent of the existing span drive machinery. From these developments, a temporary winch span drive system was developed.

The operational parameters of the temporary winch system were significantly different than those required for the permanent system. Whereas normal procedure requires that both leafs be opened for passage of any waterborne vessel, revised procedures were implemented that waived the rules. The Coast Guard and the City reached an agreement that the bridge opening procedures could be modified to allow single leaf openings to permit passage of smaller vessels while work proceeded on the other bascule leaf. To accommodate larger vessels, the Coast Guard and the City agreed that double leaf openings would only be made at predesignated times regularly scheduled by cooperative agreement between the City and Contractor. The Contractor was required to make 2 opening windows available to waterway users each day. The waterway users were then required to notify the City of their intention to use an opening window at least four hours in advance of the prescheduled time.

Once scheduled, the Contractor would open both leaves of the bridge on demand for the waterway user. The primary advantage of setting up a procedure such as that which was employed by the City of Seattle, is that waterway users will respond with unusual measures to avoid being affected by waterway restrictions. During the University Bridge rehabilitation, the City was only making two or three double leaf openings per week, as opposed to an average of 120 openings per week under normal operating conditions.

The solution the City chose to use to accomplish double leaf openings was a temporary winch span drive system. (See Figure 10 and 11.) The temporary winch system allowed the Contractor to simultaneously remove the permanent span drive machinery from the bridge and still maintain operability of the system. The winch system used on the University Bridge was a specially designed winch system consisting of two "opening" winches and two "closing" winches. By providing separate opening and closing winches, the winch system was able to provide full control to the bascule leaf throughout the operation period. The system was designed with two-speed constant torque motors to allow low speed seating operations, and higher speed opening and closing operations. Each winch was equipped with a foot-controlled hydraulically operated thruster brake such as that provided on crane systems and a separate spring set thruster brake attached to the motor. The gear reducers were parallel shaft reducers with helical gearing. Each winch was equipped with a slack cable switch that sounded an alarm if the cables became slack during operation. The operating station for the winch system was located next to the winches behind a safety shield to provide full view of the entire opening and closing operations.

In order to minimize the power requirements for the winch, bridge unbalance and maximum wind speeds were closely evaluated. Upon review of wind speed records from the local weather reporting station near the bridge, the decision was made to limit the winch operation to winds less than 26 miles per hour under ideal balance conditions, and less as the balance condition varied. As a result, it was determined that a 15 H.P. motor in combination with a 160:1 gear reducer and a two part 7/8" wire rope, would provide sufficient force to safely operate the span. In order to ensure the wind speed requirements were followed, an anemometer was placed on the bridge for guidance in the temporary span operation procedure.

The general operation principle of the winch system was similar to that of a traction winch. The geometry of the bridge could not accommodate a true traction winch so an alternate system was developed that overcame the geometry requirements for a traction winch. The winch system consisted of a pair of opening winches and a pair of closing winches. During bridge opening, the opening winches haul cable in and pull the counterweight down opening the span, while the closing winches pay out cable. As the closing winch pays out cable, the operator maintains control of the span by maintaining pressure on the line that tends to pull the counterweight up and close the span. Similarly, the bridge is closed using the same, but opposite operation procedure. A typical opening took approximately 10-15 minutes using the winches in the slow speed, and 4-6 minutes using the high speed.

The procedure used to operate the span was as follows:

1. The control tower bridge operator (permanent system) established radio contact with the temporary winch system operator.

2. After all vehicular traffic was stopped, the bridge operator disengaged the centerlocks. This put the control of the winch operated span in the hands of the winch operator.

3. The winch operator applied pressure to the hydraulic brake system, and simultaneously applied power to the opening winches. When power was applied to the winch system, the motor thruster brakes released and the hydraulic brake became active. By riding the hydraulic brake with light pressure, the operator was able to keep tension in the closing winch system wire ropes, and maintain full control of the bascule span as the bridge opened.

4. The opening continued until the operator released the raise button, and the motors stopped and the motor thruster brakes were applied. This held the span in the open position.

5. The operator in the control tower for the permanent span drive system opened the operable span and accomplished a full bridge opening.

6. Closing was similar to opening, and as the leaf approached the final seated position, the operator shifted the motor speeds to slow and seat the span.

7. After the permanently powered span was lowered into place, the operator engaged the centerlocks and opened the bridge to traffic.

SUMMARY

During the rehabilitation of an old bascule bridge, such as the University Bridge in Seattle, Washington, technical, as well as non-technical, problems or constraints are of equal importance. On the University Bridge, the most significant non-technical problem was determining how to keep the Lake Washington Ship Canal Channel open while, at the same time, maintaining vehicular/pedestrian traffic on the bridge deck. The solution to the problem which allowed a quick, efficient machinery changeover was a temporary span drive system that used wire rope winches to control the bridge.

Skid resistance was also a prime concern when selecting a grating for replacement on the bascule bridges. Design considerations included cost, weight, fatigue resistance, and skid resistance. The consideration of all of these factors led the design team to adopt 5-inch 4-way HD grating as the preferred grating for the rehabilitation work. The 5-inch 4-way HD grating provides the best skid resistance of all the bridge deck grating profiles available, and also has acceptable weight and fatigue properties.

The primary criteria for selection of a truss protection guardrail was that the railing must meet current AASHTO rail loading requirements, and also provide reasonable clearance to the main truss members to allow deflection under impact without loading the truss members. The railing selected for the project met the criteria. The railing was attached to the floorbeam system, and provides for 5-inches of deflection before contacting the main bascule bridge trusses.

The live load shoe rehabilitation revealed an important flaw in the maintenance procedure, practiced for years by the City of Seattle. Whereas the City maintenance crews were building up live load shoe shims with shims wired to the top surface of the shoes, the shimming should have been placed below the top bearing plate. Once the proper procedure was developed to shim the live load shoes, the new procedure provided a simple solution to an ongoing maintenance problem. Rebuilding the live load shoes also required reshimming the live load reactions at the back side of the counterweight. Once accomplished, the vibration and movement of the bascule span was considerably lessened, and a corresponding decrease of the uplift loading on the trunnion was accomplished.

To date, the following work has been completed on the University Bridge:

- New span drive machinery has been installed on the bridge.

- New computerized control equipment has been installed to replace the older "relay" type of equipment.
- The bridge has been fitted with buffer cylinders.
- The control room has been relocated for better viewing of the channel.
- New bridge deck grating was purchased and is in storage awaiting funding for installation.

Upon receipt of additional funding, the City of Seattle Engineering Department will proceed with installation of the new bridge deck grating, truss protection guardrails, new spanlock systems, and subsequent span balancing for the added weights. No remedial structural reinforcements will be required for the added weight on the bridge. However, the counterweights will be modified by the addition of weight to provide the proper span balance.

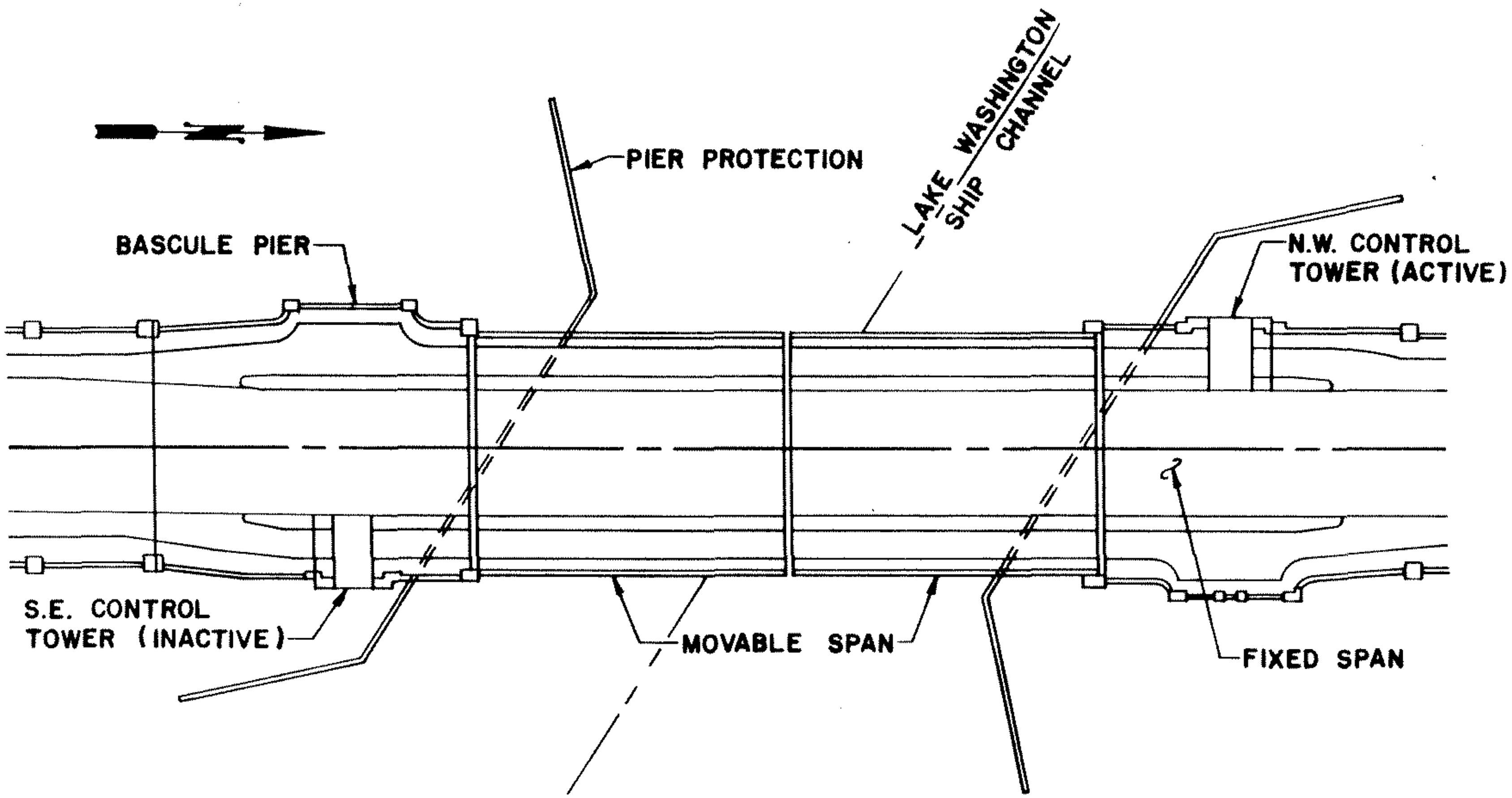


FIGURE 1 - PLAN UNIVERSITY BRIDGE

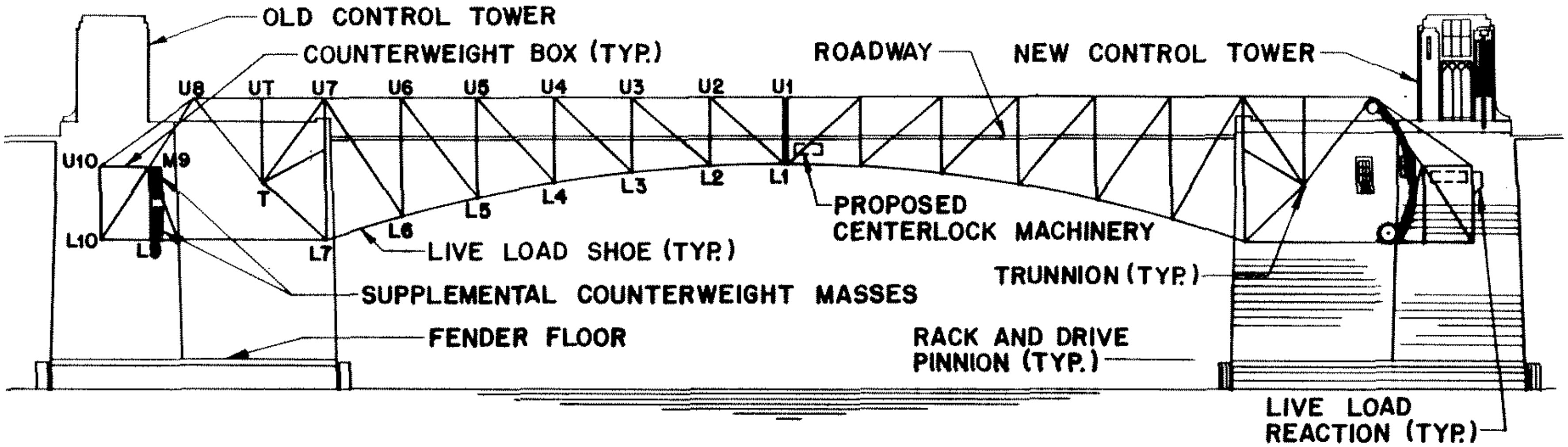


FIGURE 2 — ELEVATION OF UNIVERSITY BRIDGE
(LOOKING WEST)

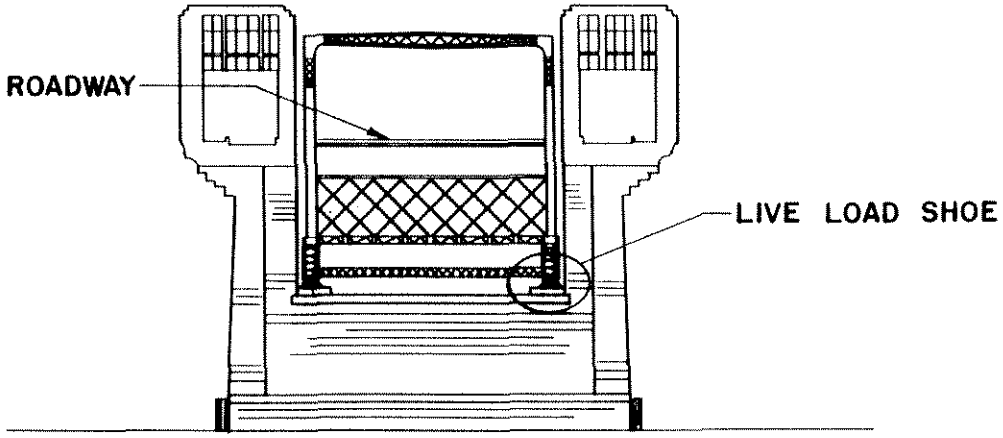


FIGURE 3 — SECTIONAL ELEVATION SHOWING BOTH THE NORTHWEST & SOUTHEAST CONTROL TOWERS

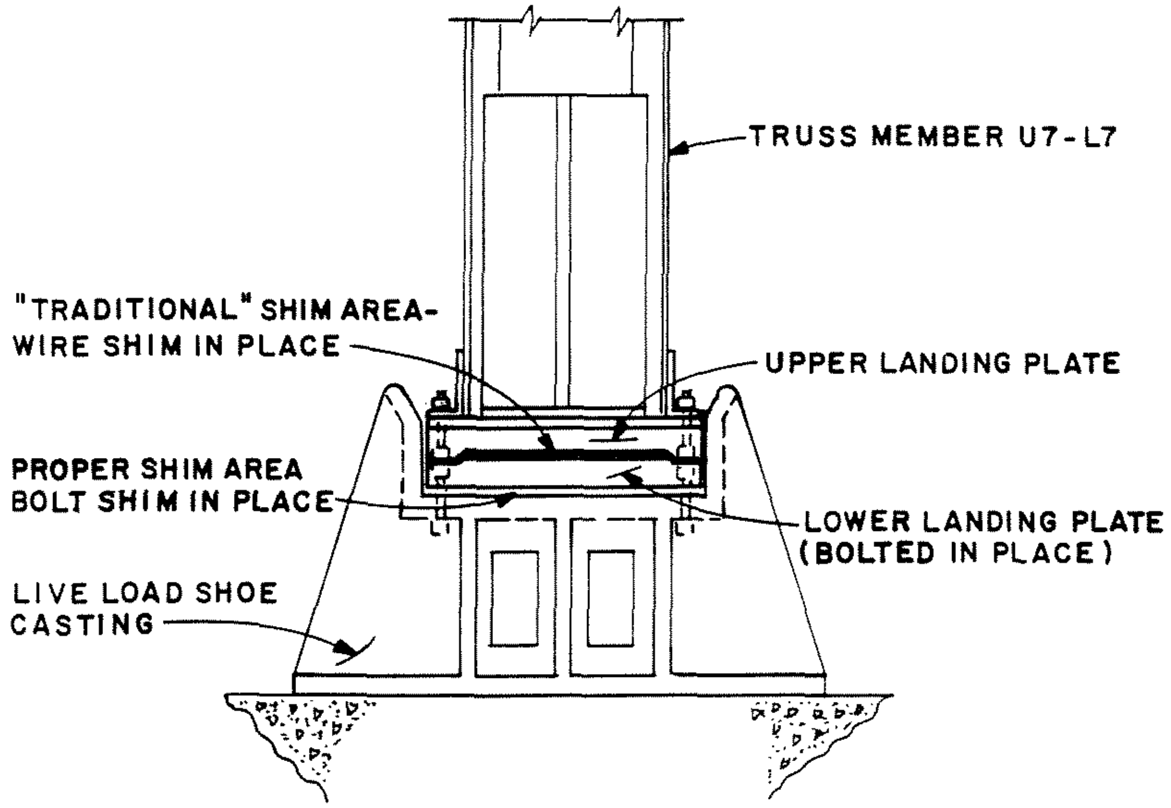


FIGURE 4 - LIVE LOAD SHOE

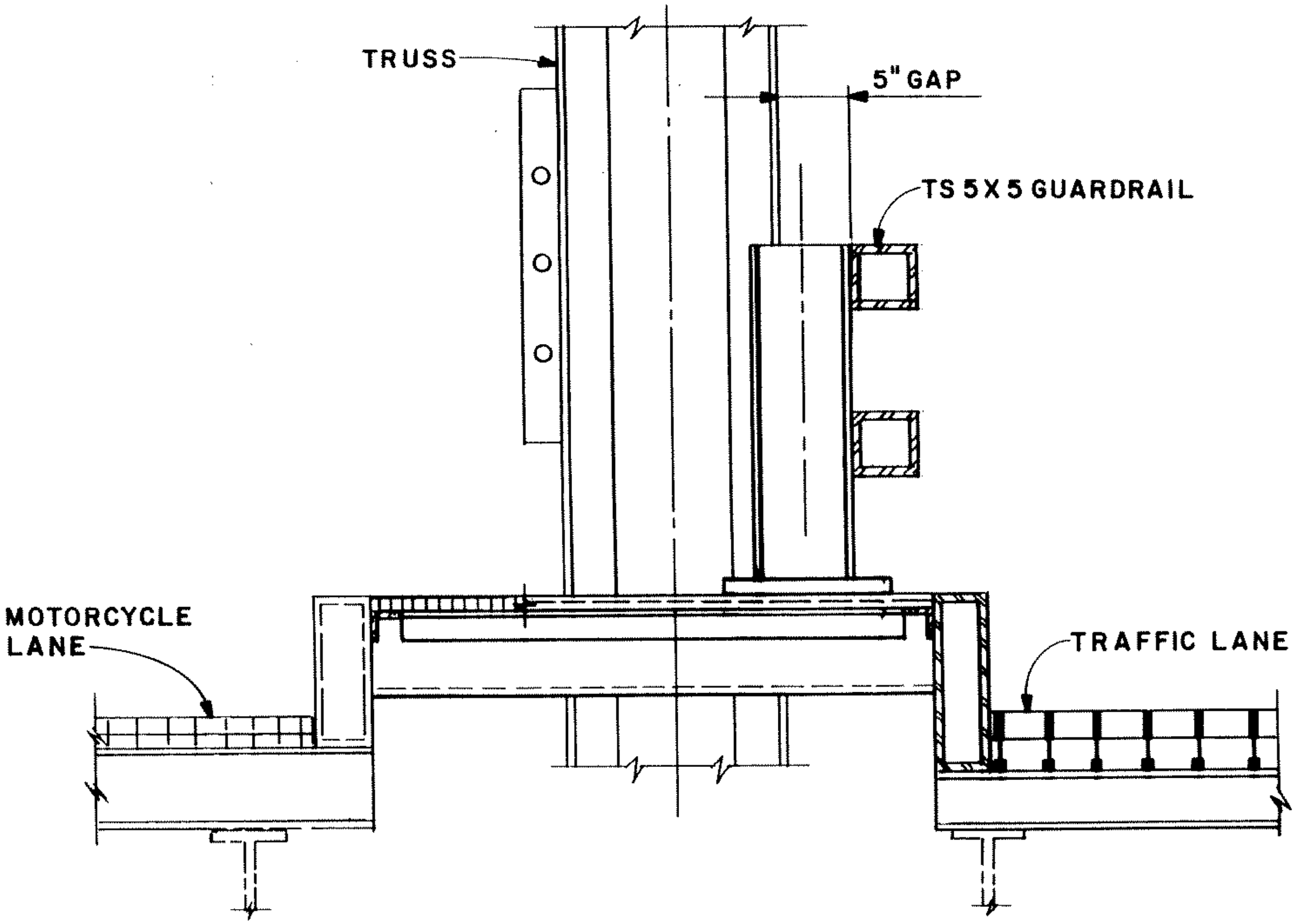


FIGURE 5 - NEW GUARDRAIL

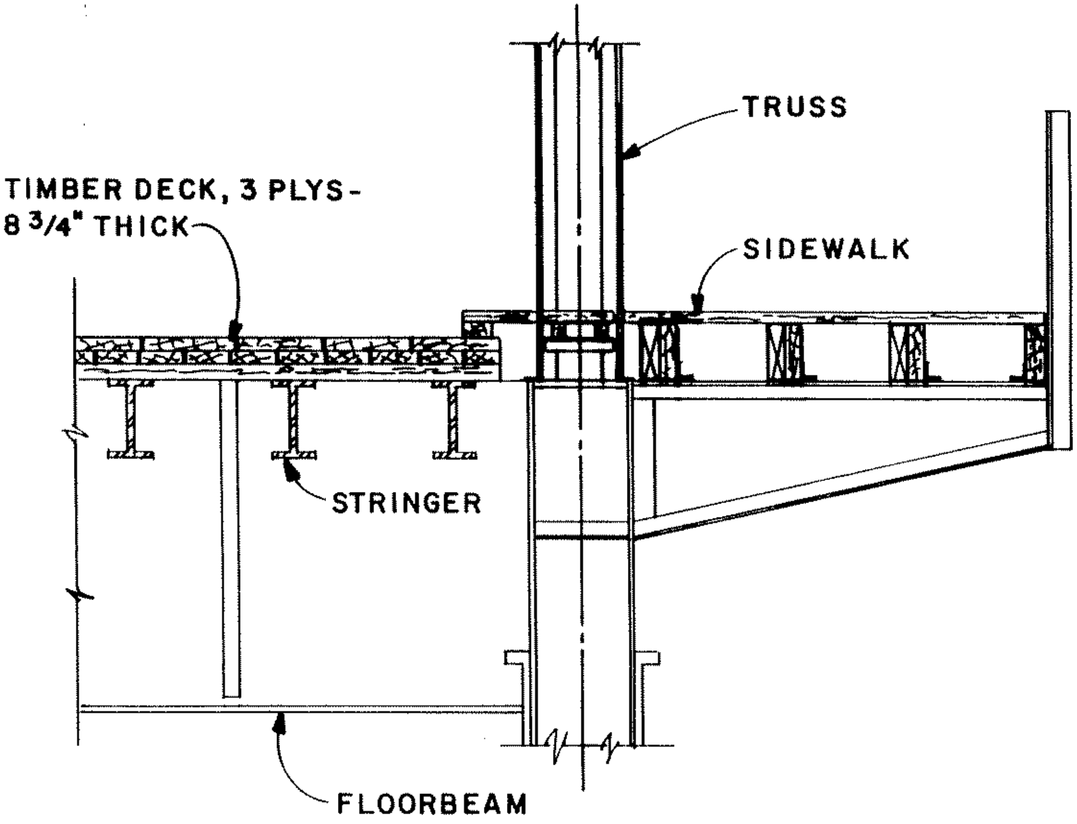


FIGURE 6 - ORIGINAL TIMBER DECK

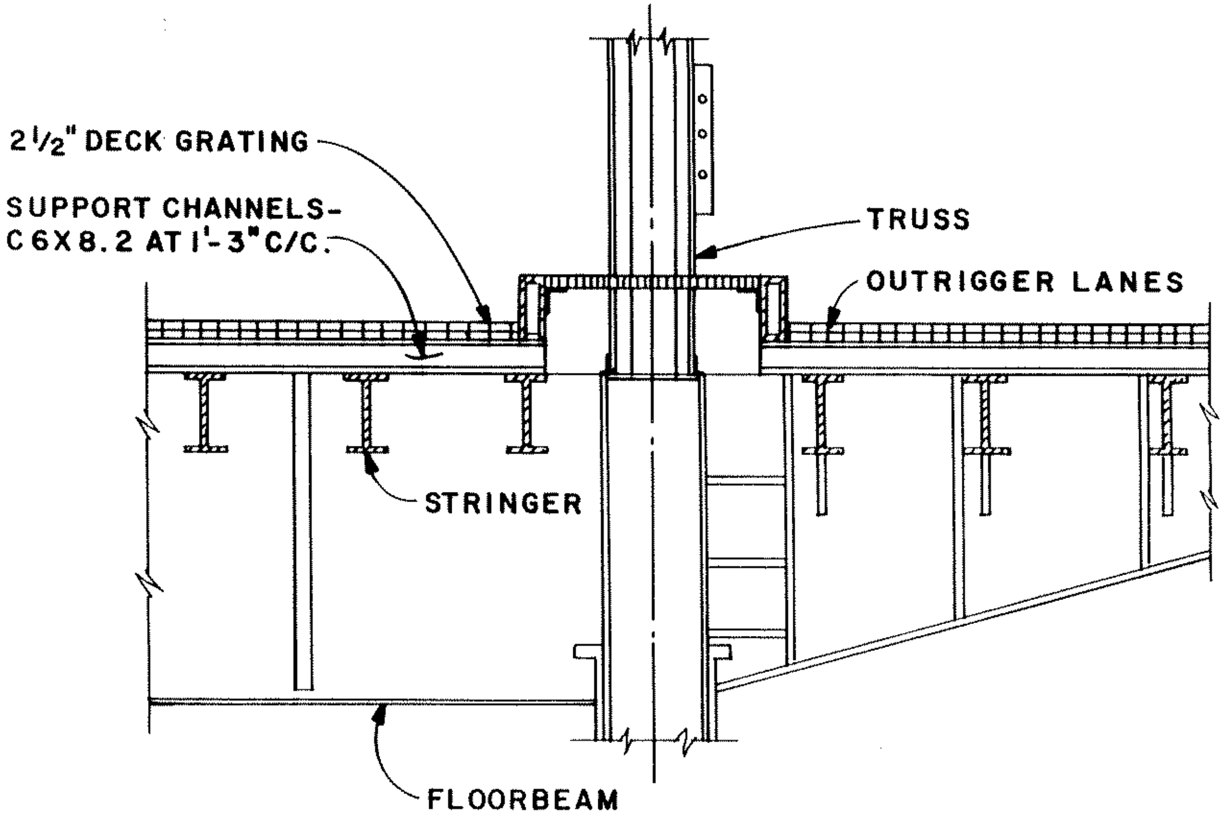


FIGURE 7- FIRST 2 1/2" DECK GRATING AND "OUTRIGGER" LANES.

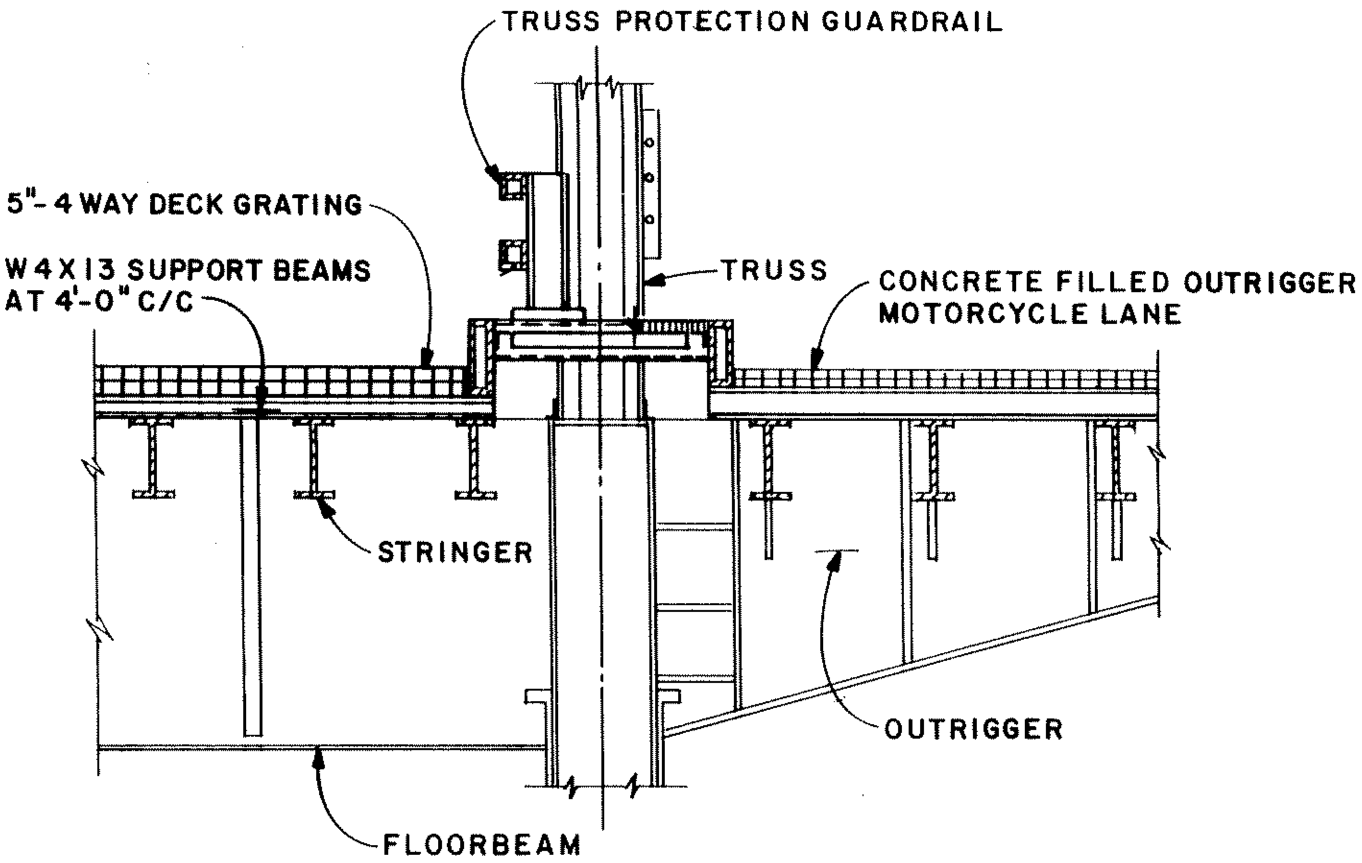
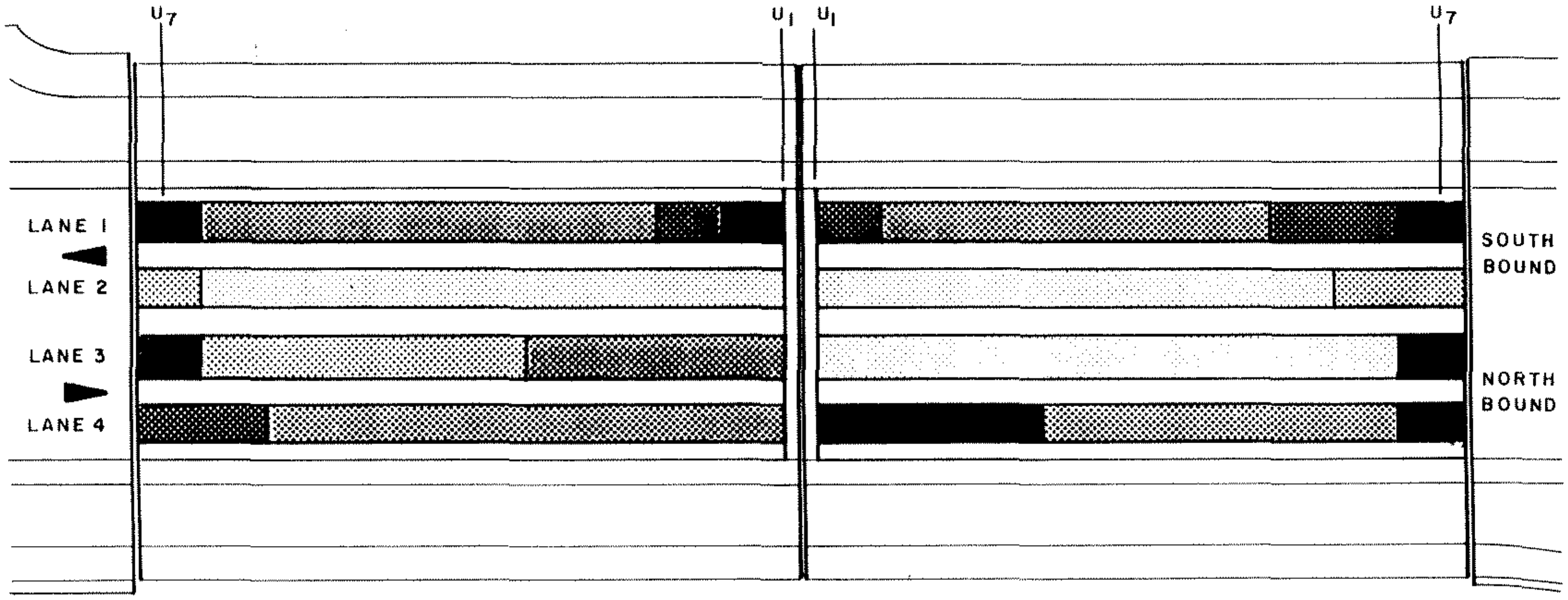


FIGURE 8 - NEW 5"-4 WAY HD GRATING



KEY- PERCENTAGE OF ADDITIONAL ATTACHMENT FILLET WELDS FROM BEARING BARS TO SUPPORT CHANNELS.

- < 30% ░
- +40–60% ░
- +70–90% ▒
- +100–130% ▓
- >130% █

$$\% \text{ REPAIR} = \frac{\text{NO. OF REPAIRS} - \text{ORIGINAL NO. OF WELDS}}{\text{ORIGINAL NO. OF WELDS}} \times 100$$

FIGURE 9- BRIDGE DECK GRATING WELD REPAIRS

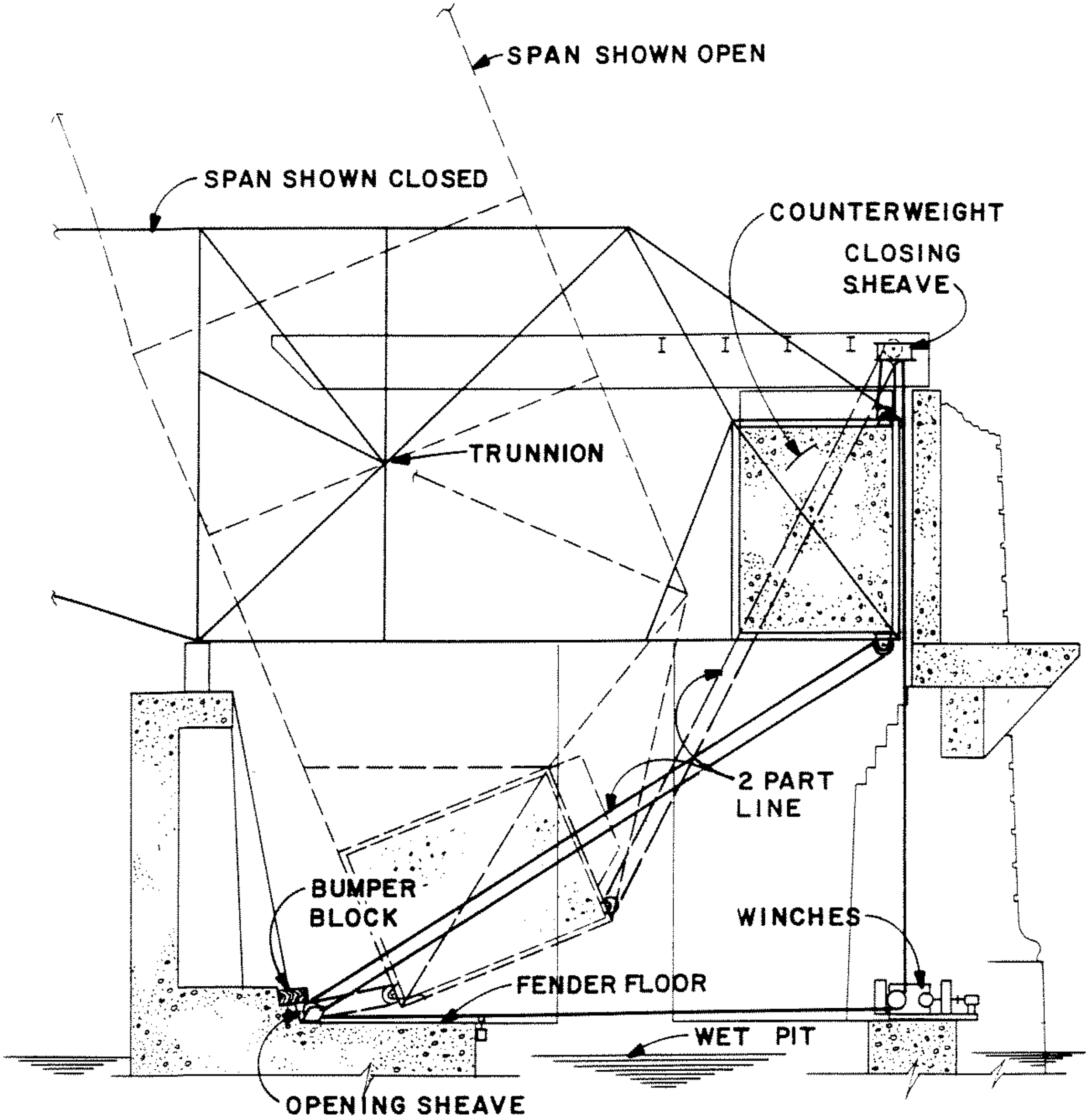


FIGURE 10 - ELEVATION OF WINCH SYSTEM

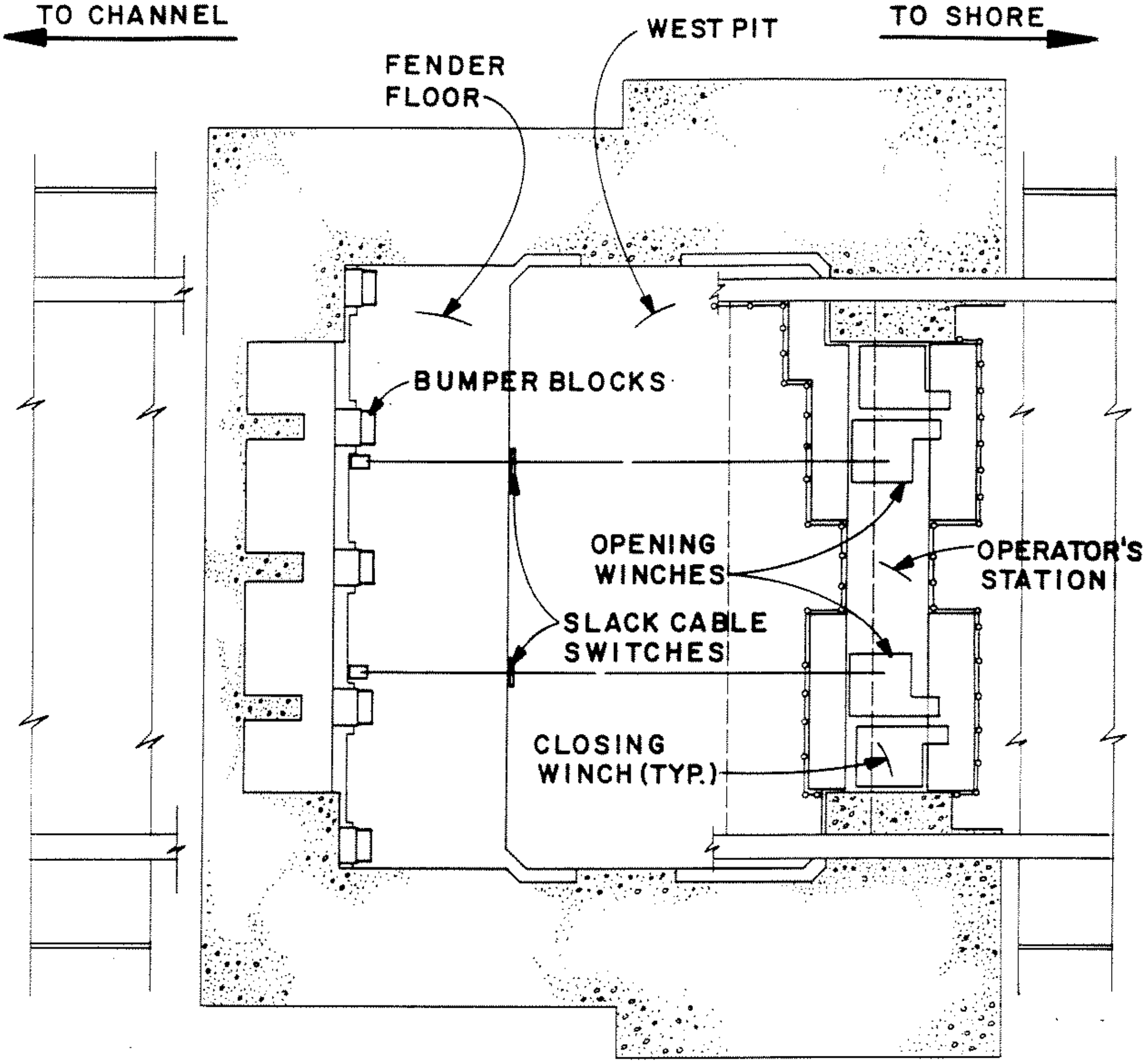


FIGURE 11- PLAN OF WINCH SYSTEM