HEAVY MOVABLE STRUCTURES, INC.

EIGHTH BIENNIAL SYMPOSIUM

NOVEMBER 8 – 10, 2000

Grosvenor Resort
Walt Disney World Village
Lake Buena Visa, Florida

"Rehabilitation of the State Route 103 Bridge/Movable Dam 5 over the Mowhawk River"

by Glenn Klein and Ronald Klinczar
TVGA Engineering

REHABILITATION OF AN INTEGRATED MOVABLE DAM AND HIGHWAY BRIDGE STRUCTURE

GLENN T. KLEIN, P.E., TVGA Engineering, Surveying, P.C.
RONALD J. KLINCZAR, P.E., TVGA Engineering, Surveying, P.C.
WALTER LYNICK, P.E., New York State Thruway Authority, Canal Design Bureau

ABSTRACT

This paper provides a case study of the rehabilitation of a 90 year-old movable dam structure, which serves a dual function as a highway bridge. The subject structure is Movable Dam 5 at Lock E-9 on New York State's Barge Canal and State Route 103 over the Mohawk River/Barge Canal. Using a system of movable gates, the dam functions to control the Mohawk River for seasonal navigation. During winter months, all dam components are raised out of the river to avoid flows of ice and debris. The overall objective of this project was to restore the structure to function as originally designed, while updating to meet current design and safety standards, thus extending the useful life of this unique and historic structure.

INTRODUCTION

The New York State Canal Corporation, a subsidiary of the New York State Thruway Authority (NYSTA), owns and operates the New York State Barge Canal System. The Erie Branch of the Barge Canal, which runs East-West across the State, includes a section in Central New York where the Mohawk River is utilized for canal navigation. A series of nine movable dam structures regulate the level of the Mohawk River to accommodate seasonal navigation. This paper focuses on Movable Dam 5 (MD 5), at Lock E-9, located in the Town of Rotterdam, Schenectady County. This particular movable dam structure also functions to support a highway bridge, which carries State Route 103 over the Mohawk River/Barge Canal. The New York State Department of Transportation (NYSDOT) is responsible for bridge-related elements of the structure, including the deck system, approach spans, sidewalk, and railings.

NYSTA, in partnership with NYSDOT, retained TVGA Engineering, Surveying, P.C. (TVGA) to perform design services for the rehabilitation of Movable Dam 5/State Route 103. The overall project objective was to rehabilitate the structure to function as originally designed, while updating to meet current design and safety standards.

BACKGROUND

The MD 5 superstructure consists of three camel back truss spans which support 17 sets of movable dam gates, in combination with one south approach span and two north approach spans. The substructure consists of five concrete piers, two abutments, and a concrete apron on the river bottom, all of which are founded on timber piles. When the dam gates are lowered against the concrete apron, a navigable pool is created upstream of the dam. The downstream pool levels are controlled by Movable Dam 4, which is located approximately 5 miles downstream from MD 5. The normal differential head between the upper and lower navigation pools is 15 feet. A canal lock, located immediately adjacent to the dam, allows canal traffic to pass between the upstream and downstream pools. During winter months, the dam components are raised up underneath the truss to avoid destructive flows of ice and debris. The dam gates and uprights are raised and lowered individually using chains drawn by 12-horsepower electric "mules". Power for operation of the dam and the adjacent lock is derived from gasoline generators located in an on-site powerhouse.

MD 5 was constructed in approximately 1909. In approximately 1914, the dam superstructure underwent extensive strengthening, including construction of a second downstream truss and strengthening of the original upstream truss, strengthening of floorbeams and bottom laterals, and installation of sway bracing. Stringers were also installed between floorbeams at this time. Shortly after, approach spans were constructed and a timber deck was installed.

In 1956, the timber deck was replaced with an open steel grate deck system.

Submerged elements of the structure including the dam gates, uprights, and chains, have been replaced in-kind several times during the life of the structure, with the most recent work being completed in the mid-1970's.

Bridge Data:

Bridge Length	750 ft.
Superstructure Types	Spans 1, 5, and 6 are through-girder spans Spans 2, 3, and 4 are camelback trusses
Bridge width	20 ft. curb-to-curb
Sidewalk Width	6 ft.
AADT	8350 vehicles per day

Dam Data:

Dam Length	530 ft.
Total Dam Height	20 ft.
Lower Gate Height	12 ft.
Upper Gate Height	8 ft.
Upright Length	45 ft.
Apron Width	110 ft. ±

Attached figures provide an illustration of the general configuration of the structure.

FIELD INVESTIGATIONS AND FINDINGS

TVGA completed a hands-on inspection of all superstructure and gate/mechanical elements. Substructure inspection included hammer-sounding all piers and abutments, and mapping of deterioration. River piers and the outer lock wall were inspected during the non-navigation season such that the full height of the pier was accessible for inspection.

TVGA personnel were able to access the downstream portion of the concrete apron for inspection while the dam gates were being lowered to ready the canal for navigation. This walk-over inspection confirmed the findings of previous diving inspections. TVGA also completed a hydrographic survey of the river channel immediately upstream and downstream of the structure to aid in determining the nature and limits of scour conditions.

The truss superstructure was found to be in fair condition with isolated impact damage and minor section loss due to corrosion on the lower chords. However, moisture and salt exposure from the open-grate deck had lead to severe deterioration of the deck framing, floorbeams, and bottom lateral members. The approach spans were in poor condition.

The dam gates, which consist of a steel skin plate braced by rolled beams and channels, exhibited moderate pitting throughout. The gate rollers and pins were severely corroded and, in some instances, inoperable. The dam uprights, which support the dam gates, were in good structural condition, with minor pitting on submerged portions.

The concrete surfaces of the substructure elements were in very poor condition, particularly on the river pier plinths, which are submerged for much of the year. The apron was in fair condition, except for minor isolated undermining and spalling below the dam gates, which resulted in leakage.

DESIGN

Based on a load rating analysis, numerous deficient members were identified which would require the structure to be posted for less-than-legal loads. A diagnostic load-testing program was implemented by TVGA to confirm and/or adjust the theoretical load rating analysis. As a result of the diagnostic testing, and by completing minor repairs at three locations, the load posting of the structure was increased from 4 tons to 16 tons. However, the requirement for a load posting on this heavily traveled route was a major impetus for rehabilitation of the bridge.

Analysis of the dam/bridge superstructure required consideration of unique load combinations. The bottom laterals, in combination with the floorbeams and the lower chords of the trusses, form a horizontal truss, which functions to support the hydrostatic loads from the dam gates and uprights. In addition to stressing the bottom lateral members, the hydrostatic loads induce axial forces into the floorbeams and the lower chords of the trusses. The hydrostatic loads are transmitted from the truss end floorbeams to the piers via massive lateral thrust anchorages. In addition to this unique lateral load, the dam operation introduces numerous load cases associated with various positions of the gates, uprights, and mules.

An HS-20 minimum design live load capacity was established for the rehabilitated truss structure. All members of the double downstream truss had sufficient capacity to support this load. However, a total of 26 vertical and diagonal members of the upstream truss required strengthening to accommodate the HS-20 criteria. Truss member strengthening details consisted of installing added angles while the members were subject to a substantially reduced loading condition during construction.

Due to extensive deterioration, repair of the existing deck system would not have been cost-effective. Several alternative deck systems were evaluated considering deck weight and associated truss capacity, surface characteristics, maintenance considerations, and cost. The alternatives considered included open steel grate, conventional concrete deck, exodermic deck, concrete-filled steel grate, and timber with asphalt wearing surface. The exodermic deck alternative system was selected due to its light weight and safe, durable wearing surface. Both NYSTA and NYSDOT indicated that a new open steel grate deck was not preferred due to exposure of the deck framing members and poor skid resistance. The weight of the conventional concrete deck would have required extensive truss member strengthening, even with lightweight aggregate. Though the timber/asphalt system would have provided a relatively lightweight closed deck, there were concerns regarding difficult quality assurance, durability, and potential future maintenance costs. When compared to the exodermic system, the concrete-filled steel grate was not preferred due to decreased durability and skid resistance.

A cast-in-place exodermic deck design was advanced. New stringers were designed composite with the deck to minimize framing weight. Four new 26 lb/ft stringers replaced the nine original 30 lb/ft stringers, while increasing capacity. The intermediate floorbeams were strengthened by replacing deteriorated partial-length cover plates with new full-length cover plates, and replacing deteriorated portions of flange angles in-kind. Due to the extent of deterioration, replacement of the end floorbeams was deemed more cost-effective than rehabilitation.

The deteriorated condition of the original bottom lateral members was a major concern, since these members function to transmit considerable hydrostatic forces. The Canal Corporation had previously identified this deficiency, and had installed a retrofit to strengthen the bottom lateral system shortly after TVGA's initial inspection. The bottom laterals were inaccessible for repair or replacement due to the presence of the deck and deck framing, so the Canal Corporation's retrofit consisted of installing a secondary set of bottom laterals below the original members. However, since the deck and deck framing would now be removed, the original bottom laterals would be accessible for a permanent repair.

Cost analyses indicated that the initial cost of approach span rehabilitation would have been approximately 70% of the cost for completely new approach spans. Given the relatively minor differential cost and the opportunity to update the structure to meet current standards, NYSDOT elected to completely replace the approach spans. A multi-girder configuration with a conventional concrete deck, was proposed for the new approach spans. New abutments were proposed immediately behind the existing abutments, and a new pier was proposed on the north approach.

A concrete coring and testing program was implemented to ascertain the depths of concrete deterioration and the soundness of the substructure interior. The coring and testing program revealed sound interior concrete with ultimate strengths on the order of 4000 to 6000 psi. Based on a cost analysis comparing patching, refacing, or replacing the piers, the refacing alternative was determined to be the most effective treatment for the piers. A minimum removal depth of ten inches was selected to ensure that a sound concrete substrate was exposed, and to provide adequate cover for the facing reinforcement. A single mat of reinforcement, consisting of #5 bars at 12 inch centers in both directions, was placed near the center of the concrete facing, such that 4 inch minimum cover was provided. Drilled and grouted reinforcing bars anchored the concrete facing to the original concrete.

CONSTRUCTION

The Canal Corporation mandated that construction work be performed without hindering normal canal operations. The canal navigation season lasts from April to November, and the conditions during the winter months are hardly suitable for construction work. Utilities on the structure also had to be maintained throughout construction, including a high-pressure gas main, electric transmission lines, telephone lines, and the electrical systems for dam operation and lighting. However, roadway closure was permitted during construction, with traffic being maintained via an off-site detour.

Dam rehabilitation work was completed in stages, one span at a time, using a cellular cofferdam system to de-water the work area while maintaining the navigation pool. Cofferdams could be installed in only one span at a time to maintain adequate control of the river flow and pool levels. Also, the cofferdams could not be left in place during the non-navigation season due to the potential for flooding as a result of the constricted flow. Due to the time required to construct and remove the cofferdam system, as well as completing the necessary repairs, only one span of the dam could be rehabilitated per year. Thus, the required duration of construction was three years.

The majority of the required work could be completed during the navigation season with the cofferdams in place. However, certain elements of work such as rehabilitation of the dam electrical system could only be completed during the non-navigation season to avoid interruptions in dam operation.

With the cofferdams in place, the dam gates and uprights were removed. The uprights were taken off-site for rehabilitation and painting in the shop. New lower dam gates were fabricated. The upper dam gates were not rehabilitated as part of this work, as the Canal Corporation's maintenance forces routinely replace these elements on an as-needed basis without hindering the normal operation of the dam. Concrete repairs were completed on one side of the piers at a time.

Construction progress to date is summarized below:

1999 Construction Season

- Bridge closed April 1999
- Span 4 rehabilitation (cofferdam, truss, deck, gate/mechanical, substructure, paint)
- Temporary relocation of gas main
- Removal of existing south approach span (Span 1)
- Construction of new South Abutment
- Construction of new Span 1 superstructure

1999-2000 Winter (non-navigation season):

- Span 2 deck replacement
- Removal of the north approach spans (Spans 5 and 6)
- Construction of new North Abutment and Pier 5

2000 Construction Season:

- Span 3 rehabilitation (cofferdam, truss, deck, gate/mechanical, substructure, paint)
- Construction of new Span 5 and 6 superstructure
- Permanent relocation of gas main
- Approach roadway reconstruction

The bridge deck and approach span elements are nearly complete, and the roadway is scheduled to reopen in December 2000. Rehabilitation of the Span 2 substructure concrete and gate/mechanical elements is scheduled for 2001 construction, which will complete the project.

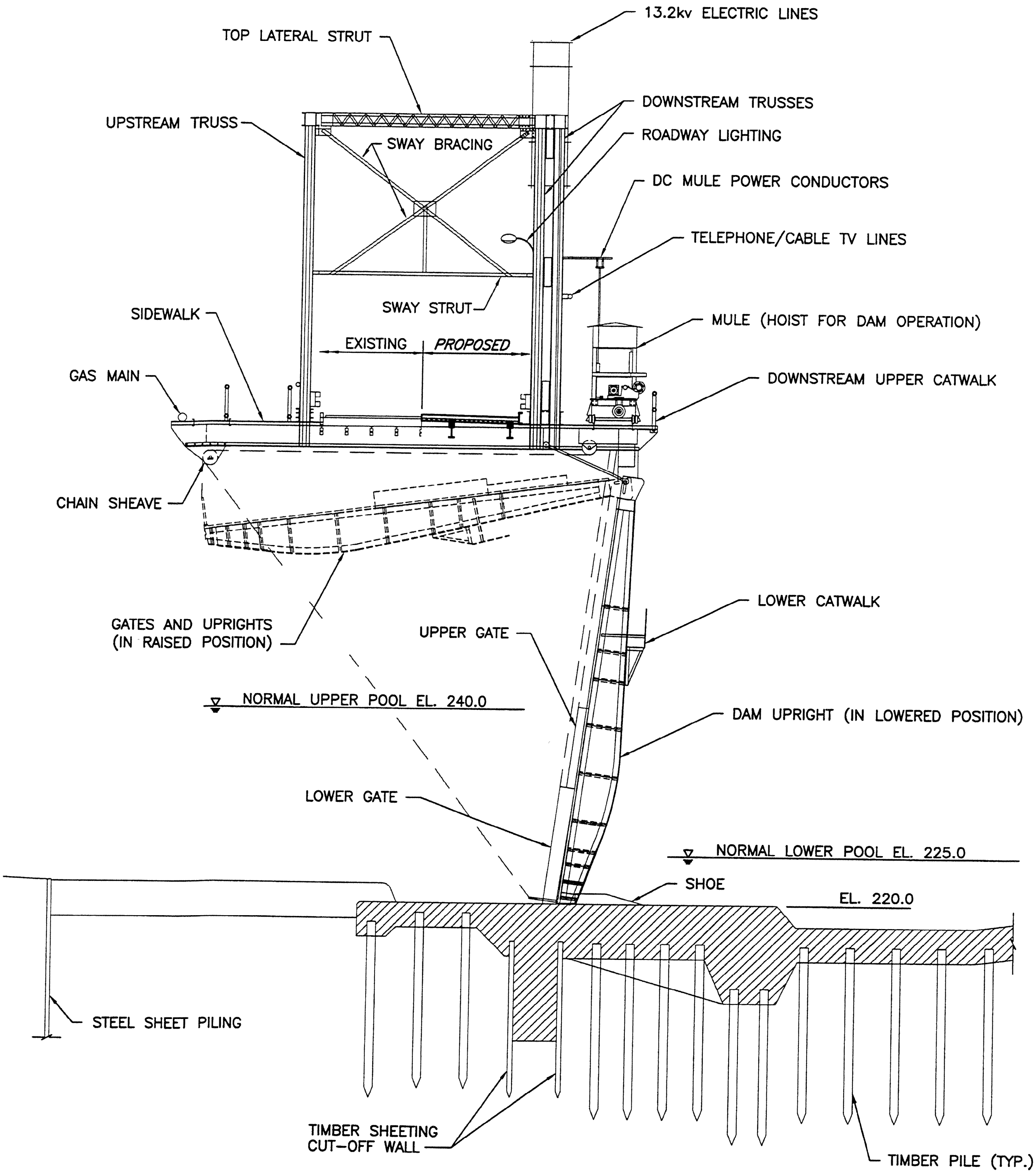
The total project cost was approximately $15 million, which was split roughly 50-50 between NYSTA and NYSDOT.

ACKNOWLEDGMENTS

The authors wish to acknowledge the contributions of the following agencies and companies who played an integral part in the successful completion of this project:

New York State Thruway Authority/New York State Canal Corporation
New York State Department of Transportation, Region 1
CD Perry and Sons, Inc.
Exodermic Bridge Deck Institute, Inc.
Lu Engineers, Civil and Environmental
Dewkett Engineering, P.C.
Robson and Woese, Inc., Consulting Engineers

HMS Report.doc



TYPICAL TRUSS SECTION

N.T.S

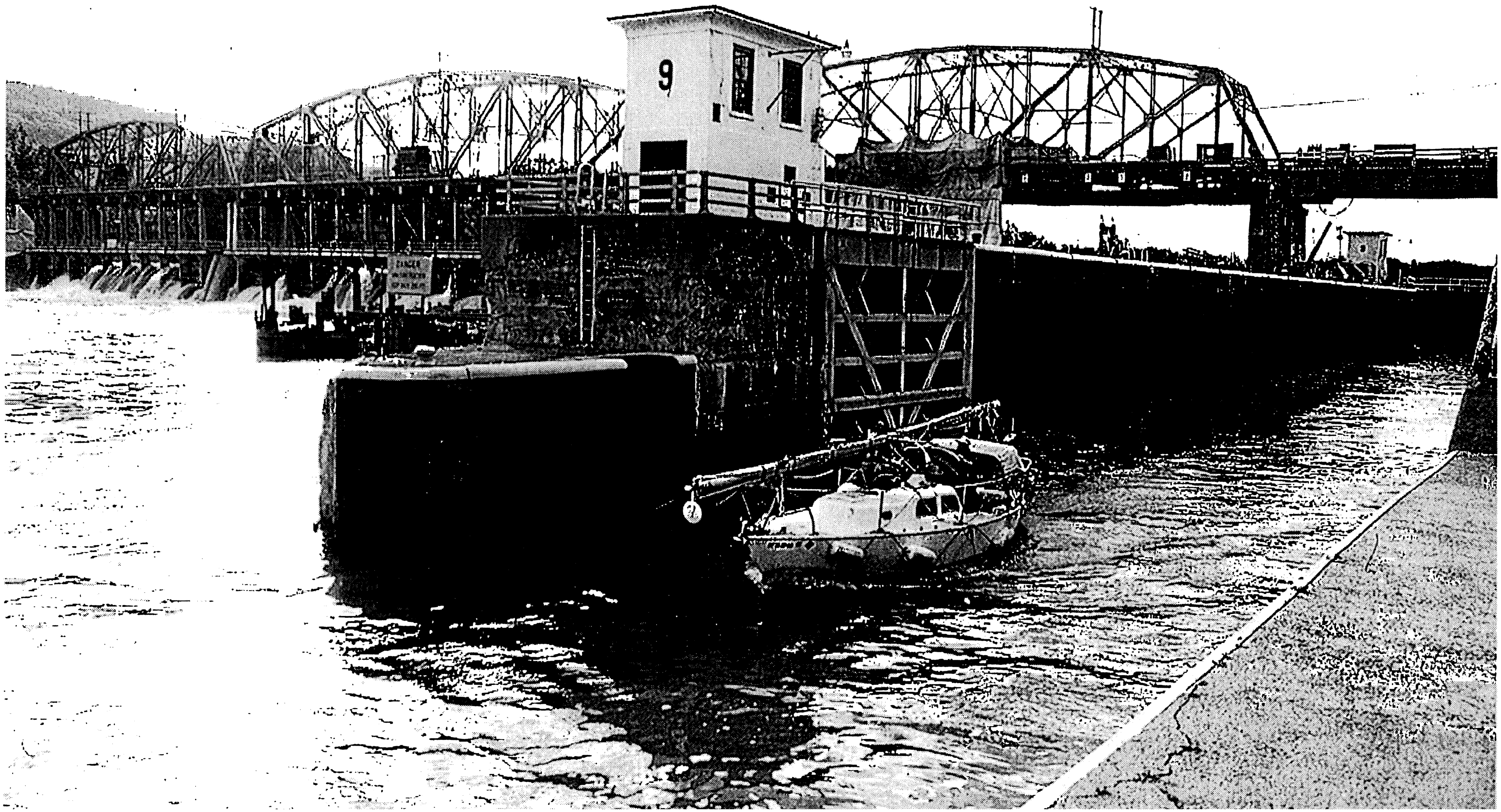
TVGA

TVGA ENGINEERING, SURVEYING, P.C.

ENGINEERS • SURVEYORS • PHOTOGRAMMETRISTS

1000 MAPLE ROAD, P.O. BOX "H"
ELMA, NY 14059-0264

(716) 655-8842
FAX: (716) 655-0937


9
DANGER

HEAVY MOVABLE STRUCTURES, INC.

EIGHTH BIENNIAL SYMPOSIUM

NOVEMBER 8 – 10, 2000

Grosvenor Resort
Walt Disney World Village
Lake Buena Visa, Florida

"CUSHIONLOKS - an Overview after Five Years Successful Service"

by James F. Alison, P.E.
Steward Machine Co., Inc.
And
Robert L. Cragg, P.E.
Consulting Engineer

CUSHIONLOKS® – An Overview After Five Years Of Successful Service

At the Heavy Movable Structures meeting in Fort Lauderdale, Florida, November 1992, Steward Machine Co. introduced the concept of improved, zero clearance, energy-absorbing span locks for bascule bridges. The heart of that system included assurance of firm lock bar contact with the wear shoes in the guides and receivers at all times, provision for automatically compensating for wear of the shoes and lock bars, and means for cushioning the shock loads during passage of heavy traffic.

These three goals were achieved by supporting the wear shoes on stiff disc springs contained in a heavy-duty steel housing secured to either the end floor beam or the bascule girder at the tip of the leaf. Steward manufactured and tested prototype units, made and was granted a patent application and Cushionloks® are now manufactured and marketed under U.S. Patent No. 5327605. Figure 1 shows a section through typical Cushionloks® guides and receivers and illustrates the manner in which the wear shoes are supported by stacks of Belleville Washers, or disc springs.

Originally we had several concerns about the serviceability of Cushionloks®, including:

- Accelerated wear of the bronze shoes against the steel lock bar due to the built-in preload
- Excessive wear of the Acme drive nut (on mechanically actuated installations) due to the increased force required to push the bar through the preloaded shoes
- Stresses and deflections in the housings during the passage of heavy traffic
- The ability of the system to perform as envisioned and actually absorb the shock loads, maintain a uniform leaf tip deflection, and reduce the routine maintenance and frequent adjustments required on existing bar lock installations

In-shop testing of the system throughout more than 36,000 continuous, full insert/withdraw cycles revealed that the high-strength bronze material selected for the wear shoes and Acme nut in the actuator was ideal for the application. No measurable amount of wear was observed on the lock bar and wear shoes or the Acme screw and nut at the completion of the tests, which represented about 10 years service for a bridge opening 300 times a month.

The guide and socket housings were investigated by an independent consultant using a finite element computer software package developed by Algor, Inc., Pittsburgh, Pennsylvania. This analysis disclosed that no excessive stresses, from either a loading or fatigue standpoint, were present under even the most adverse conditions and confirmed the serviceability of the housings.

Because it was not practical to duplicate field conditions, confirmation of the ability of the system to perform as intended had to wait until Cushionloks® were actually installed and had been in service for at least a year. The first installation was completed in February 1995, on the Bellaire Bridge, Pinellas County Florida. The application has been distinctly successful. Performance of the Cushionloks® exceeded even our most optimistic expectations--no maintenance, adjustments, or servicing has been required or conducted in more than five years, and they continue daily, trouble-free operation.

At this time we have furnished Cushionloks® for more than 20 double leaf bascule bridges in bar sizes from 4 x 6 to 6 x 9 inches with shear load transfer requirements from 30,000 to 120,000 pounds. Additionally they have been successfully installed on trunnion and rolling lift bascules as shear connectors at the leaf tips as well as tail locks located in the vicinity of the counterweights.

The purpose of this presentation, then, is to update movable bridge designers and users with our experiences in the design, installation, maintenance, successes, and limitations of these span lock systems since that time.

SYSTEM DESIGN

Design of the system is relatively straightforward. Steward must know the required shear load transfer together with details of the leaf tips to determine if sufficient room is available to install a workable system.

From this data Steward will select the bar size and material, determine the number and arrangement of the disc springs, propose the guide and receiver housing configurations, fix the required actuator stroke, and prepare a layout defining the locations of the actuator--mechanical or hydraulic--as well as the guides and receivers on the leaf tips.

Determination of a suitable bar size is dependent upon the shear load transfer, the relationship of the locations of the bar guides and receiver, the resulting stresses due to bending and shear, and the allowable stresses of the material selected. Figure 2 illustrates a typical calculation.

Disc spring sizes and stack arrangements are selected to limit the total vertical misalignment of the leaf tips as well as provide for infinite fatigue life of the springs. These calculations are in accord with the methods developed by Almen and Laszlo [1]. Figure 3 shows a sample calculation relating the applied load to the spring deflection and resulting stresses at the corners.

The configurations of the housing are flexible, and they may be either foot or flange mounted, and in some installations they are securely mounted between two structural supports. Obviously housing design is extremely important and we work closely with the engineer and owner to achieve the best arrangement for the job at hand.

After locating the guide and receiver, it is simple to determine the required stroke in order to fully insert and withdraw the bar to permit operation of the leafs.

[1] Almen, I.O. and Laszlo, A. "The Uniform-Section Disc Spring," ASME 58(1936), p 305-314

Functionally it makes no difference whether a mechanical or hydraulic actuator is used--we have many installations with both types in successful service. However, when a hydraulic actuator is used, an additional guide is required to resist the moment introduced by the shear load. With a unitized operator such as an Earle EG-3, the reaction is taken by the guide at the forward end of the operator assembly.

INSTALLATIONS

Figure 4 lists current Cushionloks® installations. The majority of installations are retrofits on existing bridges. These usually involve removing the existing lock system and installing Cushionloks® and new operators, either mechanical or hydraulic. The major differences from an installation standpoint between Cushionloks® and conventional locks are their size and weight. Due to the spring stacks over the shoes, Cushionloks® guides and receivers are taller and longer than conventional lock systems. In only one instance was this size difference problematic. In cases where the locks are being replaced with the same size bar, we have been 100% successful at working the Cushionloks® into the existing structures. On one bridge where the bar size was increased from 4 x 6 to 5 x 8 inches, the owner was not satisfied with proposed installation and the Cushionloks® were not installed. The concern was not that the guides and receivers would not fit, but that access for maintenance was extremely limited.

The good news is that our original installation at Bellaire Causeway in Pinellas County was recently inspected and found to be still operating satisfactorily without any adjustments or failures. Pinellas County maintenance personnel lubricates the guides and receivers via the lube station on the roadway every three weeks. No other maintenance has been performed or required in over five years. The Bellaire Causeway Bridge is scheduled to be replaced in the next three to five years. Steward plans to disassemble and inspect the guides and receivers for a complete evaluation and perhaps install them on another bridge to continue their useful life.

Minimal maintenance and no adjustments have been Steward's goal from the beginning, and we have been successful with all installations to date. Cushionloks® have reduced noise associated with traffic crossing the bridge and reduced impact damage to its structure, lock operator, and drive machinery.

In the process of working through the installations, Steward has developed a general sequence of steps to ensure a successful application. The first step is the design stage, when bar sizes, materials, and approximate locations for the guides and receivers are determined. Although the spring housings and bronze shoes are fairly standard for a particular bar size, the mounting arrangement for the guides and receivers can be quite different. To date Steward has designed foot mounts, flange mounts, combination wing mounts, floor beam mounts, mounts over existing rivets, mounts in gusset plates on truss girders, mounts with sidewalk supports built in, even circular flange mounts (Figures 5-11). This work is done in coordination with the owner's design engineering consultants or directly with the owner. If needed, proposal drawings are produced by

Steward for incorporation into design plans. Once a contract has been awarded, the next step is detail shop drawings. If it is a retrofit into an existing bridge, Steward visits the site to verify existing dimensions. Calculations are rechecked with the final guide and receiver locations. The completed assembly drawings are submitted to the customer for approval of the mounting arrangement. Production begins once the drawings are approved. Steward batch produces and stocks some standard parts, such as bronze shoes. However, the guide and receiver weldments are produced from scratch for each job. This allows flexibility with mounting arrangements.

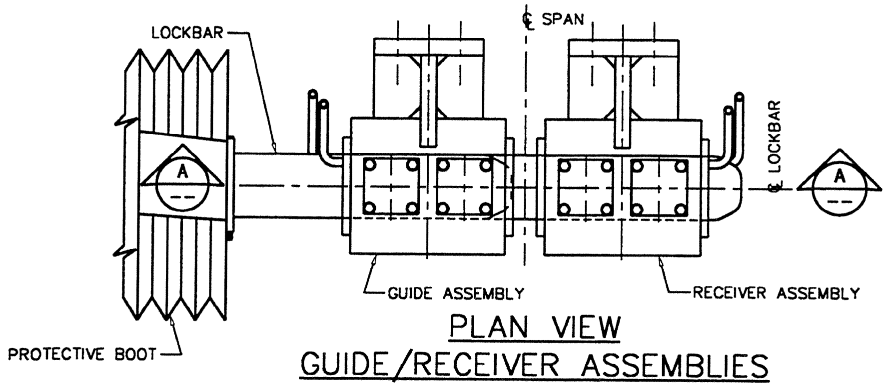
The completed guides and receivers are shop assembled with the lock bars and tested before shipment. In most cases the guides and receivers are shipped with the lock bars inserted into them. A pair of wedge tools for separating the shoes is shipped with each job to ease removal and installation of the lock bar at field assembly. As with any bar lock system, alignment of the lock bar, guides, and receivers is important. With Cushionloks®, the guide and operator can be mounted and then the receiver clamped onto the extended lock bar to allow shimming of the receiver before mounting. Steward will provide field assistance with the mounting and alignment of all Cushionloks® installations. For a typical installation Steward recommends at least one day of on-site support at the start of mounting the first unit.

Once installed, Cushionloks® do not require any maintenance or adjustments. Lubrication of the shoes and lock bar is through remote deck level lube fittings, similar to conventional lock systems. The lube lines will have to incorporate flexible hoses at the shoe connection due to the movement of the shoes in operation. All of the moving parts inside the Cushionloks® guides and receivers are packed with grease and sealed at shop assembly and do not require further lubrication.

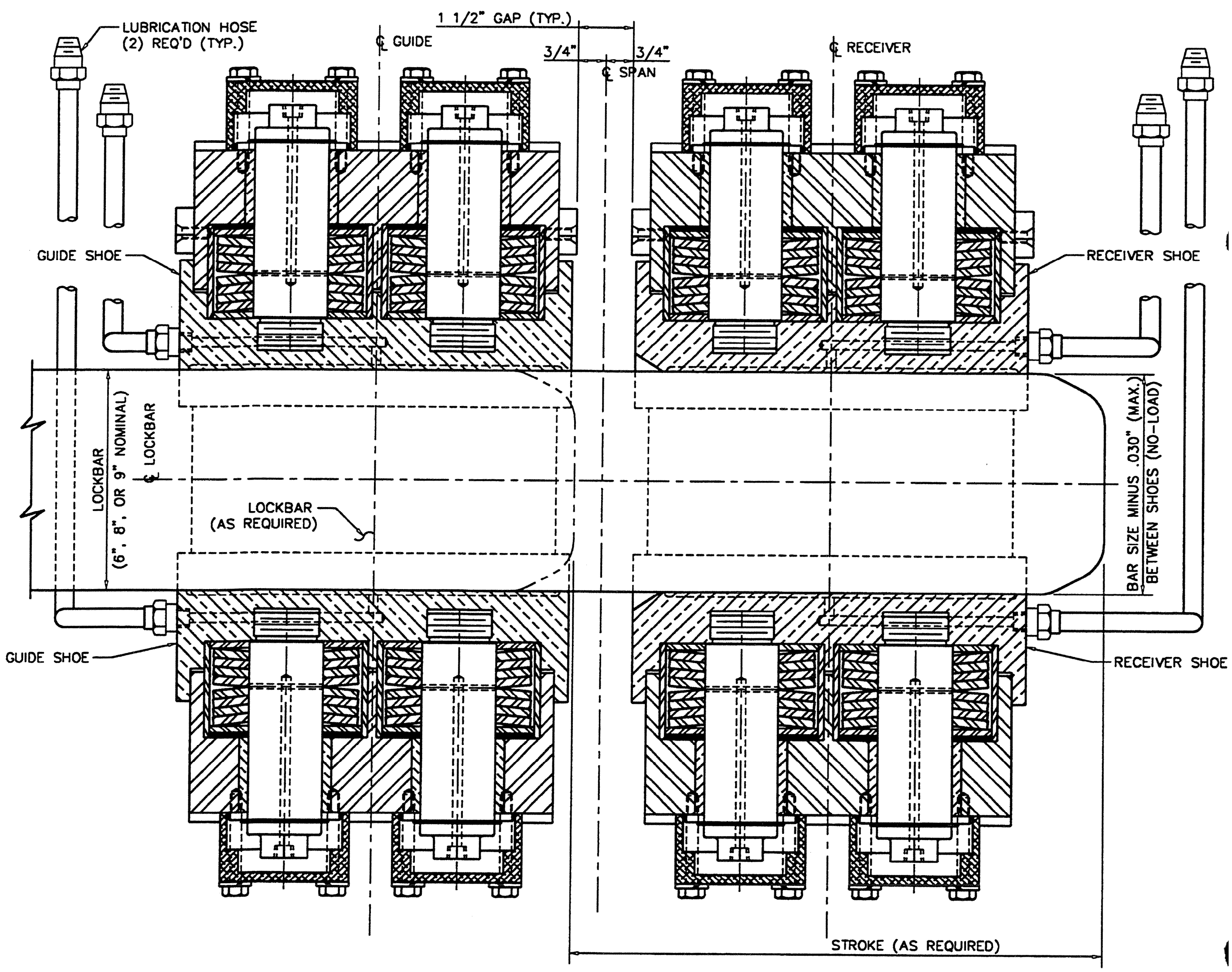
The benefits of Cushionloks® have been demonstrated with each installation. The three original goals of keeping the bar in firm contact with the shoes, self-adjusting for wear, and reducing shock loading have been achieved with all installations to date. In addition we've seen a couple of unexpected benefits. The most noticeable benefit has been a significant reduction in noise. With the passing of traffic, the leafs deflect and move normally but do not slam down on the lock bars. The other benefit has been that the components do not seem to wear nearly as fast as conventional systems. By eliminating the shock loading (pounding) on the shoes and bars, these components don't seem to wear as rapidly as with standard systems. A recent field measure of lock bars on the Bellaire Causeway Bridge found no measurable wear after five years service. The other major benefit is the complete elimination of periodic maintenance for shimming the guides and receiver shoes. Not a single installation has required any adjustment since put into service.

The proposed advantages of the Earle Cushionloks® span lock system have proved to be real in the five years that they have been in service. The measured lack of wear on the bars, the total elimination of periodic adjustments, the quieter, softer, more controlled leaf tip movement are all noticeable, documented advantages. The concerns of operator wear or installation difficulties have not been significant. The increased weight has been accommodated in all installations on existing bridges without overloading the drive systems.

With numerous projects in the works, Steward continues to refine the systems. Goals for the future include reducing weight and cost and further documenting the progress of the existing systems as time and traffic passes.

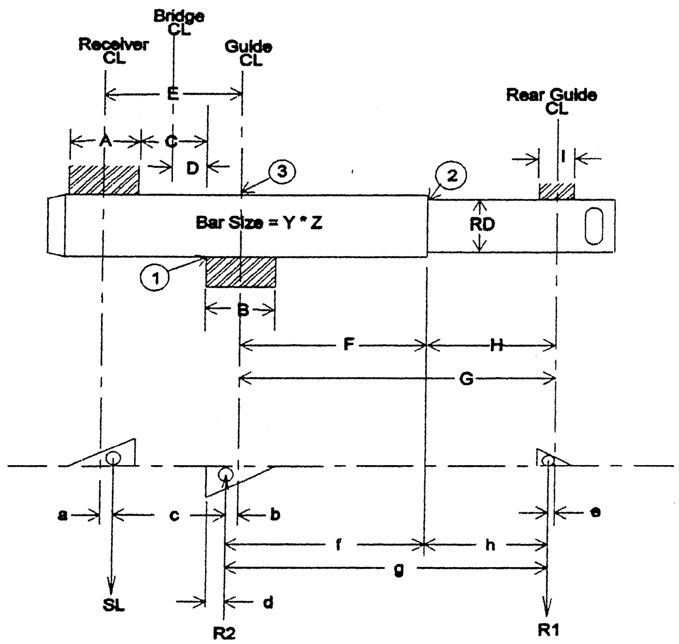


PLAN VIEW
GUIDE/RECEIVER ASSEMBLIES



SECTION A

FIGURE (1)



Job Name: HMS - 11/2000

INPUT: Lock Bar: Size - Width, Y 5 Material: A 668 Cl G
Height, Z 8 Allowable Stress.PSI 16000

Max. Shear Load Transfer, SL, lbs : 64295

Note: For bars with no round section insert "NA" for dimensions F and H and "0" (zero) for RD.

Dimensions:

A	9.875	G	32.5
B	10.625	H	18.5
E	13	I	3
F	14	RD, dia.	6.5

CALCULATED RELATIONSHIPS:

Dimensions, in:

a	1.645833	e	0.5
b	1.770833	f	12.22917
c	9.583333	g	33.77083
d	3.541667	h	18

Reactions. lb :

R1	18245
R2	82540

Bending Moment at "1", lb.-in: 388449
Section Mod, s = y*z^2/6 in^3 : 53.33
Stress at "1", PSI - M/Sec Mod 7283

Bending Moment at "2", lb.-in : 328416
RD. Sec. Mod, = Pi*d^3/32 in^3: 26.96
Stress at "2", PSI - M/Sec Mod 12181

Check shear stress at "1". Allowable stress, PSI = 8000
Shear stress -3*SL/(2*Area) PS 2411

Check bending stress on bar at centerline of guide, "3" :
Moment at "3": 547310
Stress @ "3", PSI -M/Sec Mod: 10262

Check bending stress along line of action of R2:
Moment along "R2: 616160
Stress @ "R2", PSI-M/Sec Mod: 11553

Figure 2

Cushionlok Belleville Spring Analysis

This program will calculate the stresses at the critical points on Belleville Springs.
Use correct values for Modulus of Elasticity and Poisson's Ratio for the material selected.

	Steel	17-7 PH	Phos. Br.
Modulus of Elasticity	3.00E+07	2.90E+07	1.86E+07
Poisson's Ratio	0.30	0.34	0.33

Spring Data :		*Material Data :*	
Outside dia, in. :	7.87	Material :	Steel
Inside dia., in. :	3.62	Modulus of Elas. :	3.00E+07
Thickness, in. :	0.5520	Poisson's Ratio :	0.30
Dish height, in. :	0.1614		
Load Data:		Deflection, in. :	0.050
		Load, lb. :	25951

The stresses in PSI are :

Q0 = -77,465 Q1 = -142,341 Q2 = 91,881

Q3 = 68,885 Q4 = -38,851

Figure 3

Cushionloks Installations

Name	Location	BridgeType	Bar Size, (in)	Shear Load, lb. per bar	Operator
Bellaire Causeway	Pinellas County, FL	Trunnion	4 x 6	28,000	EG-2B
Woolbright Avenue	Palm Beach Co., FL	Trunnion	4 x 6	41,600	EG-3
Ocean Avenue	Palm Beach Co., FL	Trunnion	4 x 6	41,600	EG-3
Palmetto Park Blvd.	Palm Beach Co., FL	Trunnion	4 x 6	41,600	EG-3
*Donald Ross Road	Palm Beach Co., FL	Rolling Lift	5 x 8	75,000	EG-3
Co. Rd. 707	Palm Beach Co., FL	Trunnion	4 x 6	41,600	EG-3
*Kinnickinnic Avenue	Milwaukee, WI	Trunnion	5 x 8	75,000	EG-3
Kilbourn Avenue	Milwaukee, WI	Trunnion	5 x 8	48,000	EG-3
*N. Emmber Lane	Milwaukee, WI	Trunnion	5 x 8	75,000	EG-3
*Roosevelt Bridge	Stuart, FL	Trunnion	4 x 6	50,000	Hyd.
Las Olas Blvd	Ft. Lauderdale, FL	Trunnion	4 x 6	50,000	Hyd.
Hillsboro Blvd	Ft. Lauderdale, FL	Trunnion	5 x 8	59,000	Hyd.
*Hallendale BeachBlvd.+	Ft. Lauderdale, FL	Trunnion	5 x 10	85,000	Hyd.
*Boynton Beach Blvd.+	Ft. Lauderdale, FL	Trunnion	5 x 8	75,000	Hyd.
B.B. McCormick Bridge	Jacksonville, FL	Trunnion	4 x 6	56,400	EG-3
Cermack Rd.	Chicago, IL	Rolling Lift	6 x 9	120,000	EG-3

* New Bridges + Construction in process Aug. '00

Figure 4

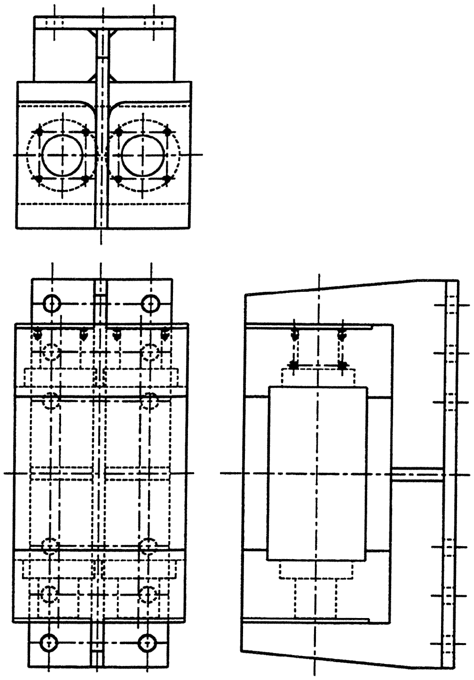


FIGURE (5)
STANDARD FOOT MOUNT

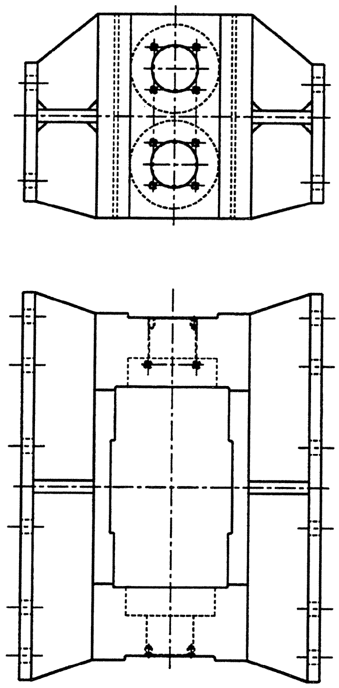


FIGURE (6)
WING MOUNT

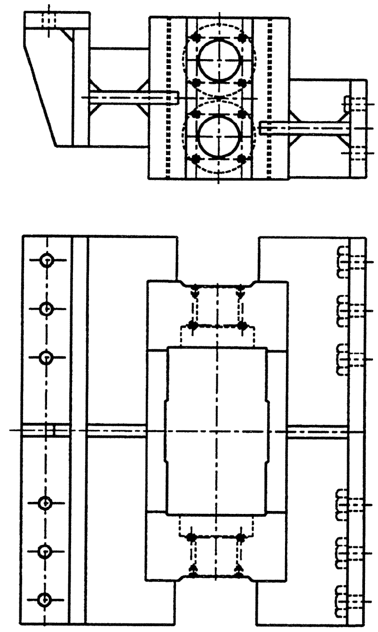


FIGURE (7)
COMBINATION FOOT/SIDE MOUNT

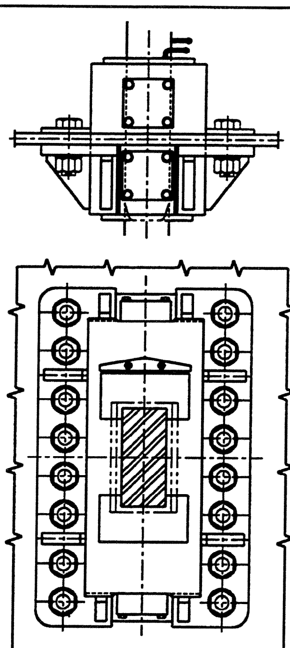


FIGURE (8)
FLANGE MOUNT

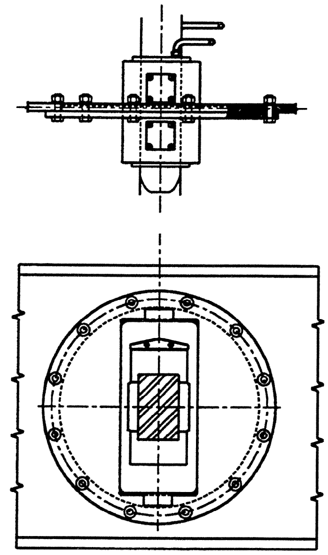


FIGURE (9)
ROUND FLANGE MOUNT

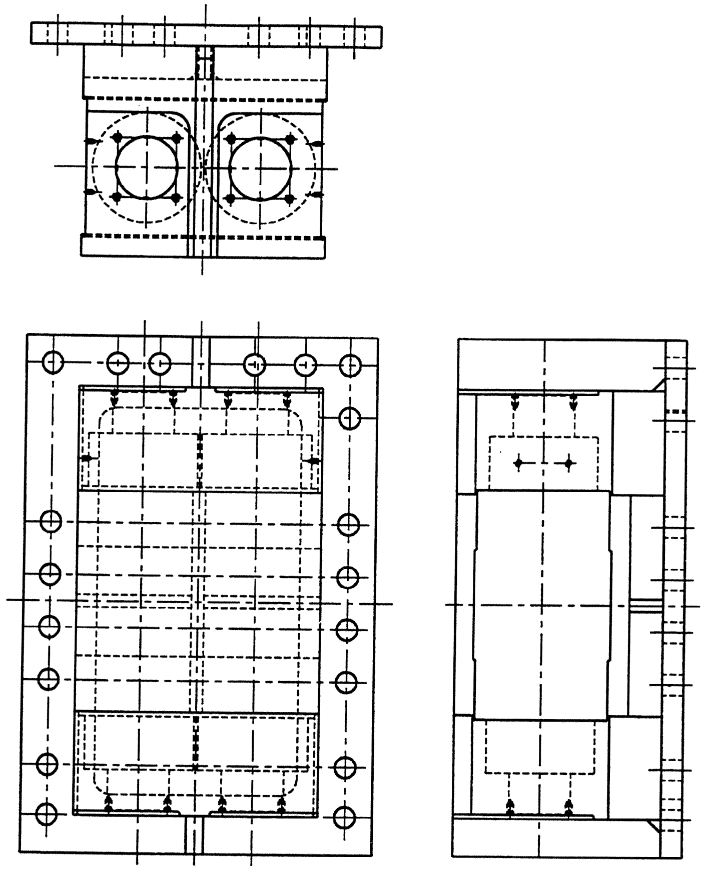


FIGURE (10)
MODIFIED FOOT MOUNT FOR 6"X9" BAR

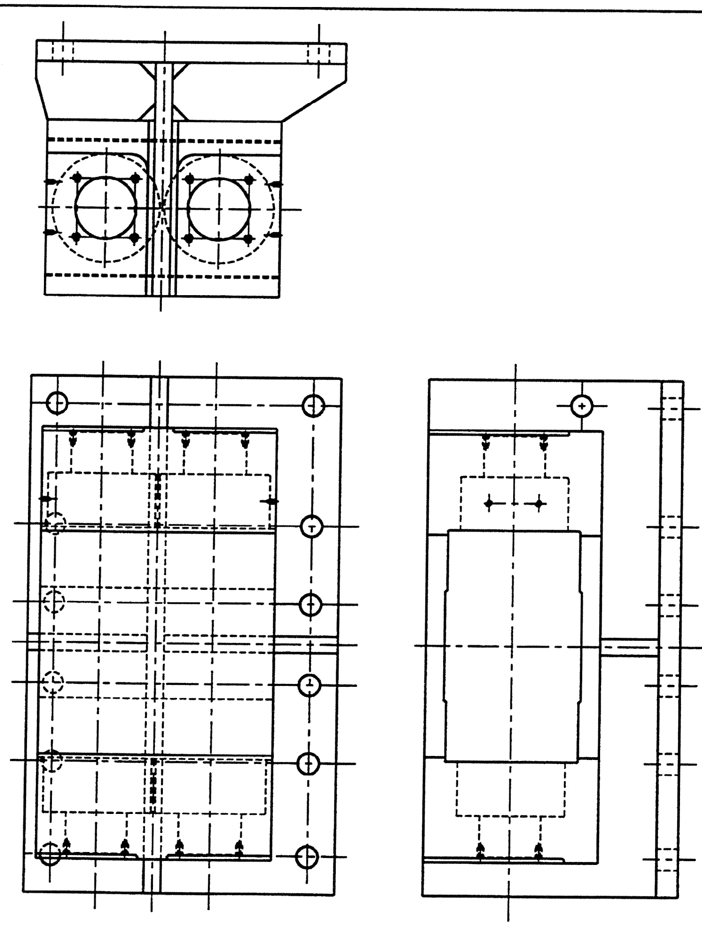


FIGURE (11)
MODIFIED FOOT MOUNT (OFFSET)