Workshop 2: Movable bridges

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WORKSHOP 2

Movable Bridges

Ponts mobiles

Bewegliche Brücken

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Design and Construction of a Bascule Bridge

Etude et construction d'un pont basculant

Entwurf und Erstellung einer Klappbrücke

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SUMMARY

The design and construction of a new bascule bridge is described. The bridge is a single-leaf overhead counterweight frame type, spanning 20 meters, carrying a roadway and a sidewalk. The factors affecting design are discussed, and the changes in design philosophy to effectively provide a structurally redundant deck are described. Restrictive conditions during construction are discussed together with problems experienced during the fabrication and construction stages.

RESUME

L'article présente le projet et la construction d'un nouveau pont basculant. Cet ouvrage comporte un portique à contre-poids suspendu, d'une portée de 20 mètres, supportant une chaussée carrossable et un trottoir. Les auteurs exposent les facteurs admis dans le calcul ainsi que l'évolution survenue dans la philosophie de l'étude en vue de parvenir effectivement à une structure du tablier redondante. Ils fournissent en outre les conditions restrictives admises pendant la construction et les problèmes survenus durant les étapes de fabrication et de mise en œuvre.

ZUSAMMENFASSUNG

Entwurf und Erstellung einer neuen Klappbrücke werden beschrieben. Die Brücke weist eine Klappe von 20 m Spannweite mit Fahrbahn und Gehweg und obenliegendem Gegengewicht auf. Die Hauptfaktoren für den Entwurf und die Bemessungsphilosophie, welche zu einem redundantem Tragsystem führte, werden erläutert. Die einschränkenden Randbedingungen während der Erstellung sowie Herstellungsprobleme werden ebenfalls beschrieben.

1. INTRODUCTION

1.1.1 The Oswego River Flows through a rural area with minor industries, and has a dam with tainter gates across at the village of Phoenix. Navigation is maintained by means of a Canal Lock System between the appropriately named Lock Island and the right bank of the river.

1.1.2 There used to be a factory on the Island and access to the island was provided by means of two heel trunnion type Bascule Bridges, one each at Culvert Street and Bridge Street. Access from Phoenix to the left bank of the river was obtained by means of a double leaf trunnion bascule type bridge at the extreme southern tip of Lock Island connected to a five span concrete encased steel arch structure, all known as the Lock Street Bridge.

1.1.3 The factory has long gone, and the Lock Street Bridge had become so deteriorated that the sidewalks had been closed to pedestrian traffic in 1985. The main structures and machinery had become so bad that it was decided that rehabilitation was impractical. A full feasibility study was commissioned by New York State Department of Transportation, (NYSDOT) to investigate the needs of the community, consider alternative options for replacement, and to undertake the necessary environmental impact study.

1.1.4 Because the existing bridge was the only crossing within a radius of 5km, it was a vital link between the communities of Phoenix and West Phoenix, New York and the new bridge needed to be constructed before demolishing the old one.

2. DESIGN

2.1 Options For The New Bridge

2.1.1 Three low level options with movable span and two high level fixed span designs were considered. The high level fixed spans were rejected as creating a detrimental impact on the area, from greater visual intrusive appearance and a greater property and land use requirement. The three low level options were

a) Reconstruction on the Lock Street Alignment;
b) Reconstruction just to the south of the existing alignment but entering Phoenix at Church Street.

Both these alternatives were above the dam.

c) Realigning the west approach to the river crossing and constructing the bridge below the dam and entering Phoenix at Culvert Street.

2.1.2 After holding local meetings NYSDOT issued its final transportation project report recommending the adoption of the third option -- the Culvert Street Alignment.

2.1.3 This finally adopted alignment for the bridge provided for a crossing of the Oswego River and navigation canal in two distinct elements. The first portion was a 192m long fixed bridge over the river from the left bank to Lock Island, below the dam in a position of much less visual impact than the existing Lock Street Bridge. Because of the decision to construct at a low level a movable span was therefore required to continue the roadway from Lock Island to the East Bank, to permit passage of river traffic.

2.2 Selection of Movable Bridge Arrangement

2.2.1 The selection of the type of movable bridge to be utilized was governed by a number of factors, not the least of which was the overall appearance of the bridge. A bascule bridge had already been tentatively assumed to be the type of movable bridge required, as being the most economical, and the design options considered were a rolling lift bridge, a simple trunnion bridge and a heel trunnion bridge.

2.2.2 The simple trunnion bridge was the type in use at the existing Lock Street Bridge, and even though it creates the least visual impact, the additional cost of constructing the counterweight pit and the potential maintenance problems of a pit below water level were considered to outweigh the advantages. The rolling lift, with overhead counterweight is possibly the least attractive movable bridge, aesthetically, and is subject to possible instability of the bascule piers. The heel trunnion type, with overhead counterweight was therefore recommended, particularly as the existing Culvert Street Bridge was of this type, and would thus result in little visual change overall in the area.

2.3 Detail Design Of Bridges

2.3.1 The decision to use a heel trunnion arrangement having been made, it was considered that the bridge should be made as redundant as possible. The normal practice with structures of this type is to have two edge girders with the hanger links connected at approximately 2/3 of the distance from the heel trunnion to the toe. The deck is then supported on cross girders between the edge girders. With this arrangement the hangers will be transmitting fluctuating deflections to the counterweight frame as vehicles move across the span.

2.3.2 It was decided to revise the usual sequence of support, by supporting the deck on multiple underdeck beams. These beams are themselves supported by the heel trunnion beam at the pivot pier, and a toe beam at the rest pier. The hanger links are connected to the toe beam directly over the rest-pier bearings. The main trunnion bearings are mounted on the ends of the heel trunnion beams.

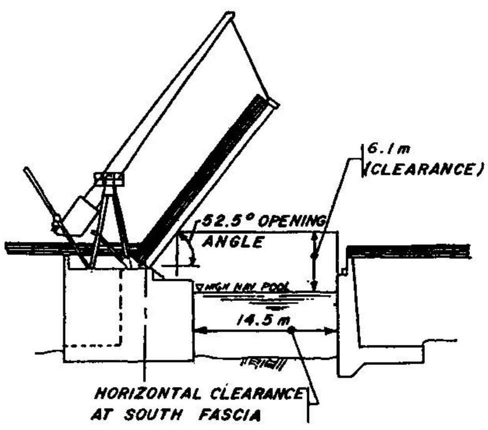

Fig. 1 Elevation with bridge open

2.3.3 The advantages of this arrangement are three fold. First, the rest pier is only supporting the live load from the bascule leaf. All dead load is supported by the overhead counterweight, but no live load is transmitted through the hanger links to the operating machinery. Second, the bascule leaf is effectively redundant, as there are eight primary load supporting beams spanning the canal, instead of the usual two through-deck girders. Finally, the longitudinal members never undergo stress reversal as is the case when the hangers are connected to the interior of the span.

2.3.4 The deck itself is a concrete-filled steel grating spanning across the stringers with haunches to provide for the final deck elevations. The Contractor was given the option to form these haunches in concrete or to provide steel fillers, and he elected to supply tapered steel sections. The deck is designed to act as simply supported panels to carry all dead load, but acts compositely with the stringers to support the live loads.

2.3.5 The counterweight boom arms were designed as tapered box sections, and were treated as free cantilever arms laterally connected at the tip. The counterweight box provides full torsional stability at the rear end, but the tip is effectively free except for the transverse strut connecting the two tips. It was for this reason that the box section was selected, in order to obtain the maximum lateral-flexural stability.

2.3.6 The A-Frame towers provide support to both the main counterweight boom trunnions, and also to the pivot of the main operating struts. Each leg was designed as an unbraced I Section, stabilized laterally only at the top junction position.

2.3.7 The link arms are not vertical, but crank in 787mm from the boom center lines to the center line of the live load shoe bearings. This arrangement was selected because of the necessity to position the A-Frames outside the roadway line, to eliminate eccentricity on the counterweight boom trunnions, these being the most heavily loaded moving elements of the entire structure, and to keep the overall length of the toe beam to a minimum. The horizontal load components of these links are resisted by the tip strut, which itself is laterally stabilized by a 45° K-brace back to the 1/3 points of the counterweight booms.

2.3.8 The counterweight was designed as a steel box to contain balance concrete. This resulted in a larger box than would have been necessary if cast iron counterweights had been used, but the proportions are similar to those utilized with the existing Bridge Street bascule span and concrete is more economical. The shape of the counterweight box resulted from trimming off the bottom outer corner at the same angle as the raised span - 52.5°.

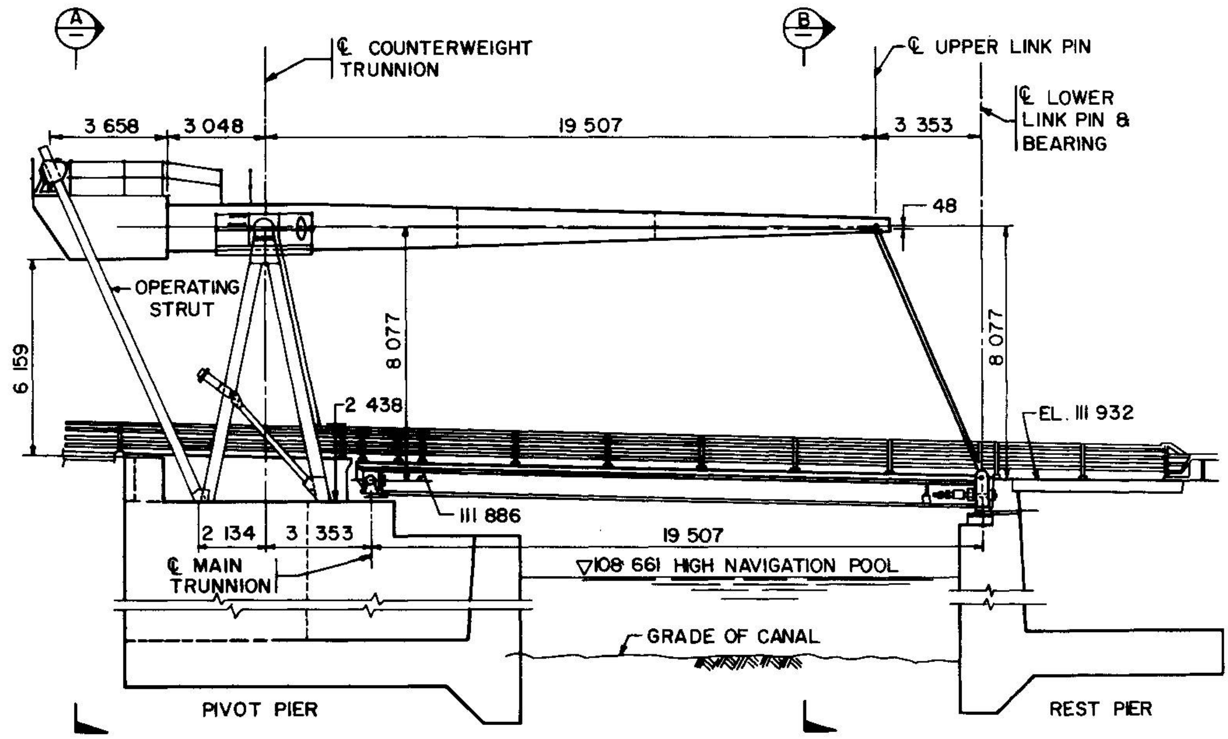

Fig. 2 Lock Street Bridge Replacement-South Elevation

2.4 Mechanical and Electrical Components

2.4.1 The decision to use an electric drive mechanism with rack and pinion operating machinery was made as a result of the need to keep maintenance of mechanical and electrical items down to a minimum. This need arises from the location of the bridge in an area subject to severe winter weather, within the New York State "Snowbelt". Canal traffic does not operate from early December through mid-April, during which period the bridge is down, and is unattended. Electric drives with direct operating mechanical components were selected as being more robust than hydraulic systems, and the entire drive system was mounted on top of the counterweight box, inaccessible to the public. The drive system consists of a 15kw motor operating through a 120:1 speed reducer via 178mm diameter shafts directly coupled to the pinions engaging the toothed racks on the operating struts. Emergency operation is through a hand crank mounted at the top of the South A-Frame tower at the maintenance platform.

2.4.2 When the bascule leaf has been lowered into the closed position, the span is fixed in place by span locking bars mounted on the outer face of the fascia girders and operating through the toe beam over the rest-pier bearings. These locking bars are operated by electrically driven linear actuators.

2.4.3 All trunnion and link arm bearings are lubricated bronze shells with steel shafts or casings. This bearing type was selected for simplicity, and to keep maintenance to a minimum.

2.4.4 The remaining item of interest is the counterweight stop block which is positioned to prevent overrun of the opening mechanism, and thus also to protect the A-Frame towers from being hit by the counterweight. Any such inadvertent impact forces will be transmitted directly to the foundations.

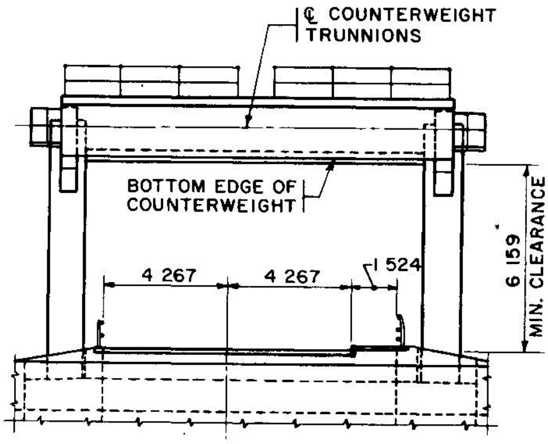

Fig. 3 Section A (Bridge Closed)

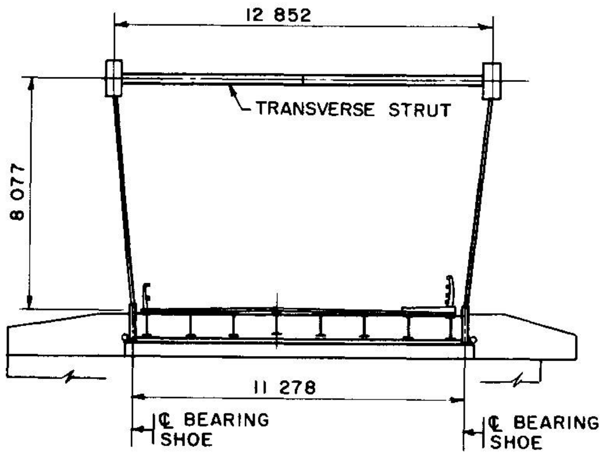

Fig. 4 Section B

3. CONSTRUCTION

3.1 Foundations

3.1.1 The primary restraint on the construction of both the fixed and bascule spans was the necessity to limit any sediment producing operations to an absolute minimum during the fish spawning season. This resulted in a prohibition of any work operation in the river that was not directly protected by coffer dams in the months of April to September. The pivot and rest pier foundations of the bascule bridge were designed as shallow spread footings on rock with transverse keys for lateral forces. They were constructed by means of providing a sheet piling cofferdam around the footing outline, excavating to bed rock and dewatering the cofferdam, placing reinforcement and filling with concrete to the top of the spread footing. This provided an effective seal to the sheet-piling and the new pier construction could be continued inside the cofferdam with no effect on the river water. The sheet piling was cut off at canal bottom level after the completion of the pier construction. This final operation was undertaken outside the spawning season.

3.2 Superstructure of the Bascule Bridge

3.2.1 All structural steelwork was fabricated at New Castle, Pa, all mechanical components made in Birmingham, Alabama, and everything was shipped by roadway to the job site. Installation of all items was undertaken using a mobile crane on the Lock Island, or on the Phoenix side of the canal lock.

3.2.2 A problem occurred during the assembly of the counterweight boom trunnion shaft to the boom hub, as a result of a failed weld, which resulted in cracks forming in the trunnion hub and boom webs. The Contractor elected to repair this area using a bolted splice detail instead of welding, as a result of a combination of time constraints, and concern over the effect of excessive welding in the vicinity of the trunnion hub. Extreme care had to be taken in the analysis of this section of the booms, as it is the point of maximum stresses for both bending and shear, and all these forces had to be accommodated in the bolted splice details, prepared by Steinman.

3.2.3 The sequence of erection was initiated by the positioning and tie down of the combined grillage bases for the main trunnion bearings and A-Frame towers. These bases allowed for a small amount of adjustment in the positioning of the A-Frames and main trunnion bearings, but still required very exact positioning.

3.2.4 The installation of all deck leaf steelwork was then undertaken. The concrete fill to the deck grating was placed in winter, and a full curing operation was undertaken to ensure that the temperature of the concrete was maintained above 7°C. This consisted of positioning a pontoon barge underneath the deck span to cover the full width of the canal, providing heavy duty plastic sheeting to close the gap from the pontoon barge to the fascia girders, covering the deck concrete with insulated blankets after placement, and then operating space heaters in the volume between the pontoon barge and the deck for the full curing period. This very effective method was only possible because the canal is normally shut down during winter.

3.2.5 The A-Frame towers were constructed independently of the deck installation. The counterweight booms had their bearings fitted to the shafts and were lifted into place with the tips supported on temporary shoring. The link arms were then installed with the bearings fitted top and bottom, and at the same time the counterweight box was assembled on top of temporary heavy-duty shoring. After all connections had been made, the counterweight concrete was placed inside the steel box. The machinery was then installed, and tested.

3.2.6 Final balancing of the overall structure was undertaken first by means of calculation of the theoretical requirements for transverse and overall balance. After installation of the necessary balance blocks, direct reaction load readings were taken at the toe beam for the closed position, to obtain the correct lateral balance, and final longitudinal balance was determined by torque figures obtained from strain gauges fixed to the main drive shafts. These readings were taken during opening and closing of the span as the exact balance was required between 35° and 45° open position, with an approximately 18kn "heavy" state at the toe beam required when the bridge is closed.

3.3 Barrier Gates

3.3.1 A final item of interest is the provision of the barrier gates designed and installed to prevent automobiles from driving into the bridge or canal when the span is raised. These are cable net gates of the energy dissipation type, which operate on a similar principle to the arrestor wires on a naval aircraft carrier. A net is stretched across the traffic lanes. If a vehicle drives into the net, the attachment bars at the ends of net pull through a series of circular rollers, and the kinetic energy of the vehicle is dissipated in the bending and straightening of the attachment bars, as they pass through the rollers. These barriers are designed to halt a 2 050 kg vehicle travelling at 110 kph in less than 8m.

3.3.2 The net, is normally parked at a clear height of 6.25m above the roadway, and is only lowered to roadway level immediately before the raising of the bascule span.

Particularités et évolution des ponts mobiles

Besonderheiten und Entwicklung beweglicher Brücken

Special Features and Evolution of Mobile Bridges

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RESUME

Les ponts mobiles sont des ouvrages particuliers faisant appel à des connaissances dans des domaines aussi variés que le génie civil, la mécanique et l'électro-mécanique. Bien que le nombre de réalisations en la matière soit faible en regard de celui des ponts fixes, un retour sur les trentes dernières années permet de déceler une certaine évolution portant sur les types d'ouvrages, les structures et les équipements spécifiques.

ZUSAMMENFASSUNG

Bewegliche Brücken sind spezielle Bauwerke, die Kenntnisse auf diversen Gebieten wie dem Bauwesen, dem Maschinenbau und der Elektronik erfordern. Obwohl die Anzahl ausgeführter Bauwerke im Vergleich zu den festen Brücken bescheiden ist, zeigt die Rückschau auf die letzten dreissig Jahre eine gewisse Entwicklung in der Typenwahl, der Tragstruktur und der spezifischen Ausrüstung.

SUMMARY

Movable bridges are special structures requiring a broad knowledge from various areas of civil and mechanical engineering and electromechanics. Although the number of existing movable bridges is much less than that of fixed bridges, looking back over the last thirty years shows a certain development in the choice of type, the structural form and the specific equipment.

1. INTRODUCTION

Les ponts mobiles sont des ouvrages de type complexe, qui associent à des structures courantes de tabliers de ponts des dispositifs spéciaux qui permettent de les manoeuvrer pour augmenter par intermittence le tirant d'air de la voie franchie, et dont la bonne conception requiert des connaissances faisant appel à la fois au génie civil (pour les structures porteuses du tablier et des appuis), à la mécanique (pour la définition et la réalisation du mouvement de l'ouvrage), à l'électro-mécanique et à l'hydraulique (pour l'accomplissement des manoeuvres).

Dans la grande majorité des cas la voie franchie est fluviale ou maritime, mais il peut aussi s'agir d'une route susceptible d'être empruntée épisodiquement par des convois exceptionnels de grande hauteur.

2. PARTICULARITES DES PONTS MOBILES

2.1 Principaux éléments constitutifs d'un pont mobile

Outre le tablier franchissant la brèche (le plus souvent désigné sous le nom de volée) et les appuis qui le supportent, un pont mobile comporte généralement :
- un dispositif d'équilibrage, dont le rôle essentiel est de réduire la dépense d'énergie nécessaire à la réalisation du mouvement,
- un dispositif de manoeuvre assurant cette réalisation,
- des dispositifs spéciaux garantissant le bon déroulement des manoeuvres,

qui permettent l'effacement du tablier pour laisser passage au trafic sur la voie franchie, puis son retour à la position initiale.

A quoi s'ajoutent de nombreux équipements annexes, liés à l'environnement plutôt qu'à l'ouvrage lui-même, mais indispensables à la sécurité des usagers (tour de contrôle, barrières mobiles, signalisation lumineuse, signalisation sonore, etc.)

Ce qui explique qu'il n'y ait que peu de rapport entre le coût d'un pont mobile et le coût d'un pont fixe de portée et de largeur identiques.

2.2 Types de ponts mobiles

En l'état actuel des choses les ponts mobiles se répartissent en cinq types principaux qui sont :
- les ponts levants, parfois appelés ponts-ascenseurs, dont le tablier subit une translation verticale,
- les ponts rétractables, également appelés ponts roulants ou ponts-brouettes, dont le tablier subit une translation longitudinale,
- les ponts tournants, dont le tablier subit une rotation autour d'un axe vertical,
- les ponts-levis, dont le tablier subit une rotation autour d'un axe horizontal,
- les ponts basculants, dont le tablier subit une rotation autour d'un axe horizontal, simple s'il s'agit d'un pont à axes, et à laquelle s'ajoute une translation horizontale s'il s'agit d'un pont à secteurs (pont Scherzer),

si l'on excepte les ponts transbordeurs dont les caractéristiques de service ne sont plus adaptées aux contraintes du trafic moderne, mais qui pourraient connaître des applications nouvelles dans le cadre d'aménagements particuliers tels que circuits touristiques, bases de loisirs, parcs d'attractions, etc.

Enfin, bien qu'il soit a priori possible de faire appel à tous les matériaux, il convient d'observer que, pour des raisons évidentes de légèreté et de robustesse ainsi que de facilité et de rapidité de manoeuvre, les ponts mobiles sont dans leur immense majorité des ponts métalliques.

2.3 Conception générale

L'intérêt technique d'un projet de pont mobile réside principalement dans le fait qu'il s'agit pratiquement chaque fois d'un cas particulier, les conditions d'implantation, la topographie des lieux, les contraintes d'exploitation, etc. faisant généralement qu'il est difficile, pour une même largeur de brèche, de transposer directement une solution mise en oeuvre auparavant dans un autre site.

Avec la nature des sols de fondation, les possibilités d'accès et le recul disponible en rives sont les premiers facteurs techniques à prendre en compte dans le choix d'un type de pont mobile. Ainsi les ponts rétractables exigent de disposer d'un recul important, alors que les ponts levants, les ponts-levis, les ponts basculants à axes et certains ponts tournants n'en demandent pas, ou peuvent se satisfaire de peu. Par contre les ponts tournants et les ponts rétractables permettent sans difficulté des franchissements biais que les ponts levants et les ponts basculants acceptent moins bien.

Viennent ensuite des considérations sur l'exploitation globale du franchissement, telles que :
- l'importance du trafic maritime ou fluvial, qui peut conduire à écarter un type d'ouvrage dont certains éléments pourraient être heurtés accidentellement par un bateau, comme les ponts tournants à deux volées,
- le nombre et la durée des manoeuvres, qui peuvent orienter vers un type de pont se mouvant rapidement,

ainsi que sur l'aspect de l'ouvrage et son intégration dans le site.

Sur ce dernier point les ponts levants, les ponts-levis et les ponts Scherzer à secteurs supérieurs ont assez mauvaise presse en raison de leurs superstructures généralement visibles d'assez loin, surtout pour une brèche de quelque importance. Si les ponts basculants à axe ou les ponts Scherzer à secteurs inférieurs peuvent échapper à ce reproche en dissimulant en partie leurs éléments dans des chambres de manoeuvre, la réalisation de celles-ci, souvent soumises à de fortes sous-pressions, peut renchérir sensiblement le coût de l'opération.

Reste enfin à définir le dispositif d'équilibrage et le dispositif de manoeuvre dont la nature et le type conditionnent étroitement le choix de la structure du tablier et de ses appuis, en n'oubliant pas l'action du vent qui peut être très pénalisante pour les ponts levants, les ponts-levis et surtout les ponts basculants.

La mise au point du projet consiste donc à trouver le meilleur compromis technique et financier entre toutes ces options.

2.4 Aspects particuliers

Aux éléments précédents, s'ajoutent un certain nombre d'équipements annexes (guidages, calages, verrous, amortisseurs, etc.) mécaniques (moteurs, freins, etc.) et électriques (contrôleurs, ralentisseurs, interrupteurs de fin de course, etc.) dont la conception et le choix judicieux sont seuls garants du bon fonctionnement du pont mobile.

D'autre part le tablier peut livrer passage :
- soit à un trafic uniquement routier,
- soit à un trafic uniquement ferroviaire,
- soit à un trafic mixte, à la fois routier et ferroviaire,

ce dernier cas étant assez fréquent dans les zones portuaires sillonnées par de nombreux branchements industriels.

S'il n'est pas possible de séparer les trafics et de prévoir deux ponts disposés côte à côte et manoeuvrant de concert, un pont unique peut porter à la fois la chaussée et la voie ferrée, mais au prix d'un alourdissement de la charpente et d'un renforcement du platelage qui devient plus complexe.

3. EVOLUTION DES PONTS MOBILES

3.1 Aperçu général

Alors que les ponts mobiles construits immédiatement après la dernière guerre, souvent pour remplacer des ouvrages détruits lors des combats, ne présentaient pratiquement pas de différences avec ceux qui les avaient précédés, recourant aux mêmes types, aux mêmes charpentes triangulées, aux mêmes contrepoids massifs, etc., pour aboutir à des réalisations aussi disgracieuses, une certaine tendance au changement s'est dessinée dans les années 50, souvent orientée vers les appuis (Pont levant de Recouvrance, à BREST, pont basculant de MARTIGUES, etc.), les tabliers conservant des formes assez traditionnelles auxquelles le développement de la construction soudée apportait néanmoins déjà quelque allègement. Tendance qui s'est affirmée au cours de la décennie suivante, à mesure que progressaient les connaissances dans le domaine des matériaux aussi bien que dans la maîtrise des méthodes de calculs, mais au sujet de laquelle il est difficile de parler véritablement d'évolution étant donné le petit nombre de ponts mobiles qui sont construits chaque année, dont la plupart de dimensions assez modestes.

3.2 Forme des ouvrages

Les améliorations apportées dans l'élaboration et la fabrication des aciers de construction, ainsi que les progrès accomplis dans leur mise en oeuvre ont permis, en faisant appel à des aciers soudables de nuance E 36 ou E 355 livrés en forte épaisseur de simplifier considérablement les structures des tabliers.

Ainsi les profils en caisson ouverts, obtenus à partir de cornières ou de poutrelles en U reliées par des barrettes multiples, qui constituaient jusqu'alors les barres des poutres triangulées ou des treillis, ont ils été remplacés par de véritables caissons formés de plats assemblés par soudage, avec obturation à leurs extrêmités par des diaphragmes étanches, ou par des profils en I reconstitués par soudage. Apparue sur le pont levant du Martrou à ROCHEFORT-sur-MER en 1967, cette évolution s'est confirmée au début des années 1970 avec les ponts du HAVRE aux poutres Warren très épurées, presque identiques à celles de ponts fixes.

La rigidité transversale apportée par les structures en caisson aux éléments constitutifs des pylônes et des balanciers des ponts levis a également permis de réduire l'importance des contreventements supérieurs, voire de les supprimer sur quelques ouvrages dont l'aspect s'en est trouvé réhaussé.

Dans le même ordre d'idée le remplacement du béton lourd par des gueuses de fonte ou des brames d'acier a contribué à diminuer fortement le volume des contrepoids d'équilibrage surplombant la chaussée des ponts basculants et des ponts-levis, qui, en devenant moins encombrants, ont pu être enfermés dans des caisses placées en queue des balanciers ou des secteurs, libérant ainsi totalement la voie de franchissement de tout obstacle. Des tentatives ont même eu lieu, sur de petites brèches, pour simplifier encore plus les ponts-levis en ne prévoyant qu'une suspension unique avec pylône, balancier et contrepoids implantés d'un seul côté du tablier ; tentatives qui ne semblent pas devoir être encouragées dans la mesure où cette disposition introduit dans la structure des efforts de torsion considérables dont les calculs effectués ne tiennent pas toujours bien compte.

A cet effet les incidents constatés à partir de 1974 sur le premier pont de l'Ecluse François Ier au HAVRE [1] [2] puis l'accident survenu en 1979 au pont Vétillart [3] ont rappelé aux ingénieurs l'importance des calculs détaillés (y compris à la fatigue) et des dispositions constructives soigneusement étudiées, surtout pour les pièces massives ou de formes particulières qui peuvent échapper aux hypothèses des méthodes de calculs classiques.

3.3 Types d'ouvrages

Une certaine défaveur de la part des concepteurs semble, au cours de la décennie écoulée, avoir touché :

- les ponts levants, qui comportent toujours une limitation de tirant d'air et dont les tours de manoeuvre posent souvent de sérieux problèmes d'intégration

dans le site,
- les ponts tournants à deux volées, très vulnérables au choc des bateaux,
- les ponts basculants, très sensibles à l'action du vent en position levée, dont l'esthétique peut être discutable ou qui nécessitent des chambres de manoeuvre assez couteuses,

peut être en raison d'incidents survenus ici et là sur des ouvrages de tels types.

Par contre plusieurs ponts-levis ont été construits (Pont du Bassin Jacques Cartier à SAINT-MALO, pont de SAINT-VALERY-en-CAUX) ainsi que quelques ponts rétractables dont :
- le second pont de l'Ecluse François Ier au HAVRE,
- le pont de CHERBOURG,
- les ponts de la voie des écluses à SAINT-MALO,

avec chaque fois une solution différente pour l'effacement du pont auxiliaire, qui constitue souvent la pierre d'achoppement pour ce dernier type d'ouvrage dont un des avantages est d'offrir peu de prise au vent.

3.4 Platelages

Les platelages en bois, peu durables et mal adaptés au trafic moderne, ont été remplacés dans les années 50 soit par des dalles mixtes, associant une tôle de platelage plane ou cintrée à une faible épaisseur de béton, soit par des caillebotis en acier ou en alliage léger [4] .

Aux premières, qui étaient robustes mais lourdes, les seconds opposaient leur légèreté et leur facilité de pose qui se sont à la longue révélées être plutôt des inconvénients en raison :
- de leur mauvaise tenue au trafic, avec le desserrage répété des boulons de fixation ou la rupture des pattes d'attache, spécialement pour les unités en alliage léger,
- des nuisances sonores provoquées par le passage des véhicules lourds sur des éléments détachés, disjoints ou déformés,
- des problèmes de corrosion induits par l'accumulation, dans les tabliers et dans certains mécanismes, de débris divers qui y maintienne une humidité constante, ainsi que par le dépôt de produits de déverglaçage des chaussées adjacentes entraînés par les pneumatiques des véhicules,

ce qui les a fait progressivement abandonner.

Actuellement la quasi totalité des platelages, supérieurs ou inférieurs, sont constitués par des dalles orthotropes recouvertes de revêtements minces antidérapants à base de résines. Des tentatives ont été faites pour utiliser des revêtements épais, parfois plus confortables, mais qui n'ont pas eu de suite étant donné le supplément de poids ainsi apporté et les précautions particulières qu'il convient de prendre pour assurer sa bonne tenue lors des manoeuvres.

Lorsque le pont mobile livre passage à un trafic mixte, la présence de rails n'étant pas compatible avec la mise en oeuvre d'un revêtement de chaussée mince, le problème a été résolu en séparant carrement les deux types de circulation sur le tablier.

3.5 Dispositif de manoeuvre

Aux systèmes de transmission classiques utilisant essentiellement des engrenages,des crémaillères ou des bielles, se sont peu à peu substitués des dispositifs mettant en oeuvre des vérins hydrauliques qui allient simplicité, puissance et robustesse, mais dont l'adoption a parfois apporté quelques modifications dans la conception des ouvrages, notamment en exigeant de prévoir des caissons anticorrosion.

4. CONCLUSION

Les particularités des ponts mobiles ainsi exposées montrent qu'il s'agit d'ouvrages ayant un caractère très spécifique, et qui nécessitent des études assez poussées aussi bien pour la définition de l'aménagement envisagé que lors de la mise au point du projet d'exécution.

L'attention des maîtres d'ouvrages doit notamment être attirée sur le fait que le coût de ces études n'est, comme celui de la construction, en rien comparable avec ce qui se pratique pour les ponts fixes.

BIBLIOGRAPHIE

1. MAQUET J.F., Avaries du pont mobile de l'Ecluse François Ier. Journées de l'Association Française des Ponts et Charpentes de juin 1978 - Thème III.
2. ROUDIER J., Fissuration du pont basculant de l'Ecluse François Ier. Journées de l'Association Française des Ponts et Charpentes de mars 1983 - Thème I.
3. ROUDIER J., Rupture du balancier supérieur du pont-levis de l'Ecluse Vétillart. Journées de l'Association Française des Ponts et Charpentes de mars 1983 - Thème I.
4. MEHUE P., Platelage des ponts et passerelles métalliques. Bulletin "Ponts métalliques" n° 9 de 1983 - Office Technique pour l'Utilisation de l'Acier.

Structural and Mechanical Rehabilitation of Old Bascule Bridges

Rénovation structurale et mécanique de vieux ponts basculants

Bauliche und mechanische Erneuerung alter Klappbrücken

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SUMMARY

Rehabilitation of old bascule bridges usually makes good economic sense. Carrying capacity, traffic capacity and machinery and control systems can be improved by structural and mechanical rehabilitation. Often, the work has to be staged and traffic maintained on one half of the bridge deck. The paper is illustrated by case histories.

RESUME

La rénovation de vieux ponts basculants est rentable. La capacité portante, la capacité du trafic ainsi que la machinerie et les systèmes de contrôle peuvent être améliorés par une rénovation structurale et mécanique des ouvrages. Souvent, il faut entreprendre les travaux par étapes tout en maintenant le trafic sur l'autre moitié du tablier du pont. L'article est illustré par des exemples.

ZUSAMMENFASSUNG

Die Erneuerung alter Klappbrücken ist im allgemeinen wirtschaftlich. Tragfähigkeit, Leistungsfähigkeit für den Verkehr, Maschinerie und Kontrollsysteme können durch bauliche und mechanische Erneuerung verbessert werden. Meist sind die Arbeiten in Abschnitten auszuführen, um den Verkehr auf einer Brückenhälfte aufrecht zu erhalten. Der Beitrag wird durch Beispiele erläutert.

1. GENERAL

Many bascule bridges, mainly constructed in the period of 1920 to 1950, have insufficient carrying capacity (only vehicles of 30 to 50 t max.). The traffic capacity is not satisfactory either, due to a narrow carriageway width of 5.5 to 6 m. Furthermore, in many cases they are in poor condition.

Therefore, a replacement with a new high level bridge without a movable span to disturb the traffic is very often considered. However, calculations very often show that this is a very expensive solution, especially inside a town where the long bridge ramps require a lot of land.

A new bascule bridge to replace the old one is then the next solution considered. However, it might be difficult to change the final alignment of the new bridge in relation to that of the old bridge. An interim bridge could then be constructed and the new bridge be positioned in the same alignment as the old one.

All the above solutions are quite expensive, and therefore in many cases it makes good economic sense to repair, strengthen and widen the old bridge.

2. STRUCTURAL REHABILITATION

2.1 Bascule Span

Double leaf bascule bridges very often consist of two main girders, each of a double web steel girder (hollow section), with I-profiled cross beams with a distance of 4 to 5 m and I-profiled secondary longitudinal steel girders wearing a wooden deck. Figure 1. shows a typical section of the original deck on a bascule bridge at Frederikssund in Denmark.

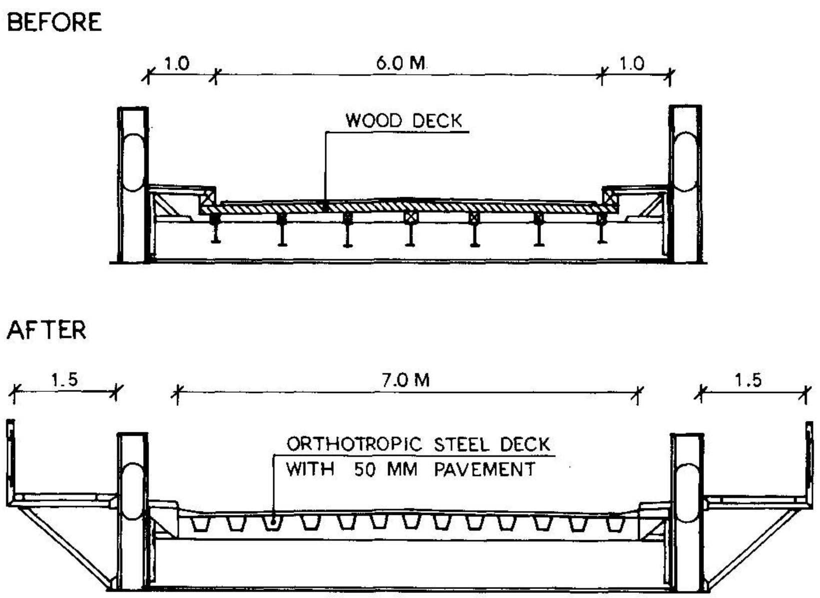


Fig. 1. Frederikssund Bridge. Rehabilitation of Bascule Span.

In order to save load and carrying capacity the deck including the secondary longitudinal girders was replaced by a modern orthotropic steel deck as shown in the figure. Normally the deck plate has a thickness of 12 mm and is supported by trapezoid formed ribs with a centre line distance of 30 cm.

The width of the carriageway was at the same time increased from 6.0 to 7.0 m by constructing new sidewalks outside the main girders by means of steel supports bolted to the side of the main girders and covered by a deck of aluminium profiles.

Due to the new sidewalks it was necessary to move the control tower. When removing the new control tower it was completely rebuilt and equipped with a modern control system, see below.

At the above mentioned bridge the 12 mm steel deck plate was paved with 50 mm asphaltic mastic. The steel deck was sandblasted to SA 3 and primed with a bostic adhesive. A 4 mm layer of softer mastic was applied as a waterproofing membrane. Two layers of asphaltic mastic of 20 and 25 mm were applied and friction stones rolled into the surface of the upper layer before cooling down.

However, a greater reduction of weight is necessary, the steel deck can be paved with 5 to 6 mm thin resin pavement of acrylic or epoxy basis. This has been done at the rehabilitation of other bascule bridges in Denmark (Guldborg Bridge and Sønderborg Bridge). The resin pavement is applied after sandblasting of the steel deck, normally in two layers, and the upper surface applied with corns of bausit which gives an excellent surface friction.

The service life of the asphaltic mastic is about 20 to 25 years, whereas the resin pavement has a service life of 15 to 17 years.

The orthotropic steel deck is normally connected to the existing cross beams in a flexible way to adapt the different levels, and a careful level survey has to be carried out. In many cases the connections must be made by friction bolting, as the old steel of the cross beams is not weldable.

2.2 Side Spans

Side spans are normally rehabilitated in accordance with the same principles as the bascule spans.

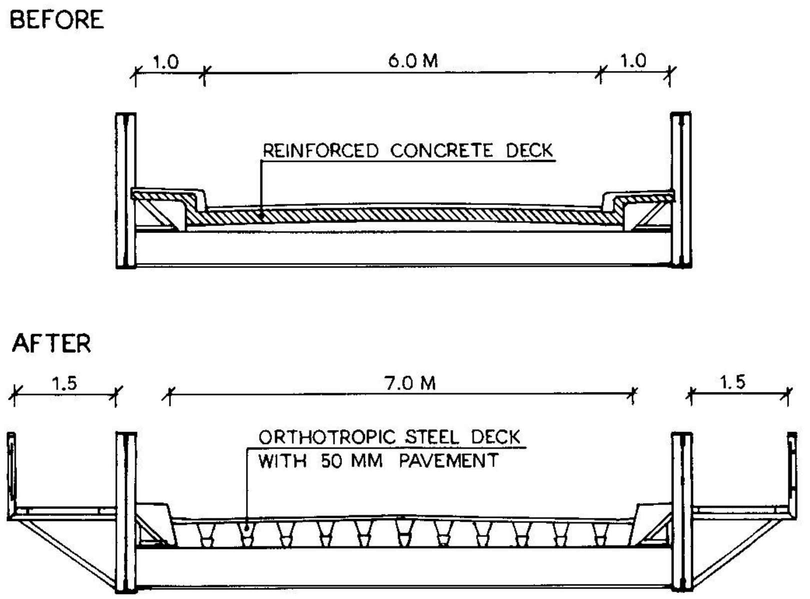


Fig. 2. Frederikssund Bridge. Rehabilitation of Side Spans.

As dead load often is an dominating factor for the main girders of longer side spans it is important to reduce weight. In many cases, as shown on the typical section, figure 2, the old deck consists of a reinforced concrete slab, supported by secondary longitudinal steel beams.

The weight of the concrete slab is normally quite essential and a replacement with an orthotropic steel deck - especially with a thin resin pavement - will normally save so much weight that the live load can be increased by 50% resulting in a satisfactory load-carrying capacity.

At the same time the composition of the orthotropic steel deck and the existing cross beams will often increase the moment of resistance of the new cross beam to the double.

The connection between the cross beam and the main girders is then often the weak point. However, supplemental friction bolting will normally solve this problem.

3. MECHANICAL REHABILITATION

3.1 Machinery

In some cases the old machinery is in good shape and only a few changes are necessary, such as new electric motors.

A replacement of the steel machine parts (such as gear, wheels and racks) is normally an expensive affair. In some cases an application of an electric hydraulic machinery to replace the old machinery can be a more economic solution.

A typical hydraulic machinery is shown in figure 3.

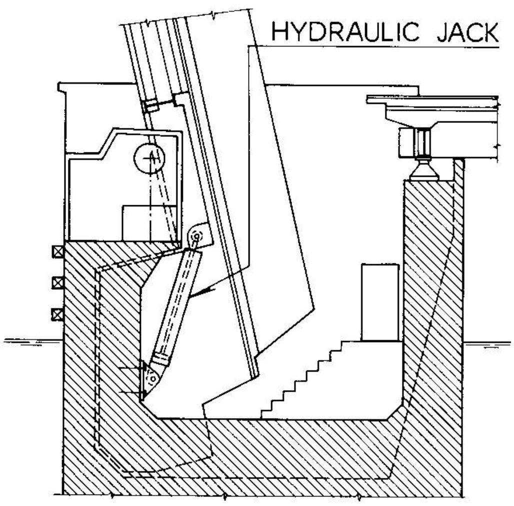


Fig. 3. Typical Hydraulic Machinery

An advantage of such replacements is that the hydraulic machinery can be installed while the old machinery is still functioning. Thus, the bridge can be operated nearly without interruptions.

3.2 Control System

To operate an old bridge a lot of individual actions have to be taken: Start machinery, sea signals, road traffic signals and lifting barriers, release brakes and locking arrangement, sea signals, raise bascule spans, etc. etc.

By automatic electric operation this can be done by touching a few buttons. The correct succession of activities can also be insured.

During the last years all these operations have been computer controlled, so the raising speed of the bascule spans can be regulated in the correct way.

All this means that a bascule bridge can be operated by a less skilled personnel and a saving is hereby obtained.

It therefore makes good economic sense, when rehabilitating the bridge to modernize the electric control. Furthermore, it is often difficult to get spare parts for the old steering system.

4. DESIGN PHASES

Design phases can be divided into:

1) Special investigation of the old bascule bridge (condition and load-carrying capacity)
2) Economy study of bridge type (high level or bascule bridge)
3) Preliminary design and budget (preliminary design of chosen type and corresponding economic estimate)
4) Detailed design and tender documents
5) Tender (orientation and evaluation of bids (tender report))
6) Supervision (site supervision during construction with home office back-up)
7) Daily running and maintenance

Re 1) Also the substructure shall be carefully investigated, e.g. underwater piers and scour protection by diver and sea-bed levelling by echo soundings.

Dubious steel parts of machinery shall be carefully examined by ultra sound or magnetoflux. Samples for chemical testing of weldability shall be taken.

Re 6) During the rehabilitation works it is important that the site supervision is in close contact with the design engineer ("home office back-up"), as the actual conditions often differ from the design assumptions.

Re 7) Manuals shall be made to describe how to operate the renovated bridge, sea/road traffic regulations, how to repair the new machinery, how to service machinery and how to maintain the structures (length of interval for painting), etc.

Instructions for regular routine inspections shall also be made.

5. TRAFFIC PROBLEMS

In many cases it is not possible to find a deviation route when rehabilitating the bridge. Then, the rehabilitation has to be made in two phases (only one half of the carriageway can be rehabilitated at a time). Therefore, a new orthotropic steel deck panel for a bascule span has to be made in two parts, which must later be connected by in situ welding.

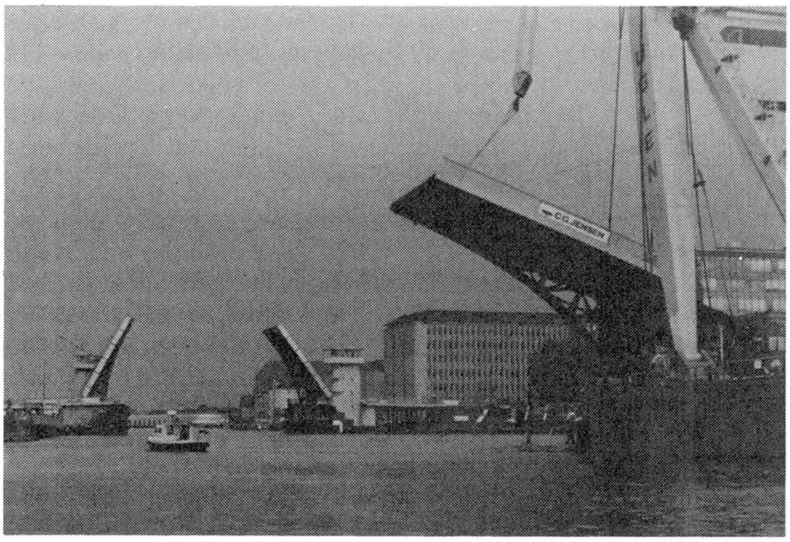
Traffic light regulation is necessary when the original carriageway only has two lanes. Traffic queue lengths must be calculated and local population advised accordingly. Some activities require a full traffic stop and should take place during night hours. Ship traffic restriction should be announced in due time.

Special attention shall be paid to secure that the bascule equilibrium is maintained and a comprehensive planning for changing the weight of the structural elements of the bascule span is necessary.

6. KNIPPELS BRIDGE

In the centre of Copenhagen city, the northern harbour bridge between Copenhagen and Christianshavn was in a poor condition.

The bridge is owned by Copenhagen Harbour Authorities and Copenhagen Municipality.



The bridge is a double leaf bascule bridge from 1939 with a bascule span of 35 m (free navigation clearance) and a deck width of 27 m.

The main pier concrete was deteriorated to a large extent. Some of the main trusses of the bascule span were badly corroded and the side span concrete needed repairing, etc.

Fig. 4. Knippels Bridge. Bascule Span Moved by Floating Crane.

An economy study indicated that the most feasible solution was to rehabilitate the old bridge under the condition that traffic on one half carriageway was maintained.

As the bascule span steel structures needed repair at a workshop, a special solution was found: The bascule span structures were divided (cut) at the bridge centre line into two halves which could be operated separately, see figure 5.. Hereby, one half could be taken by a huge floating crane to the nearby situated workshop. This is possible because the bascule span structures consists of four steel truss main girders.

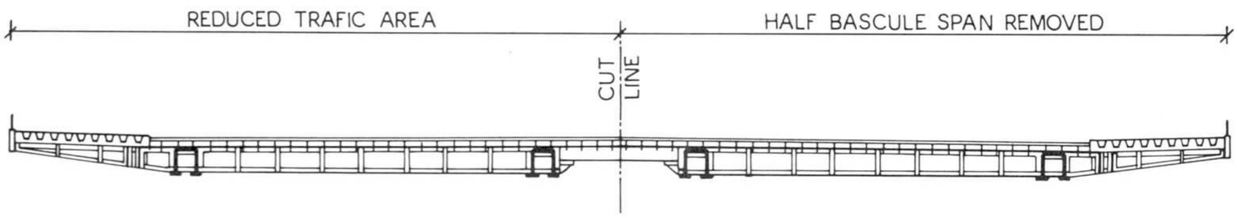


Fig. 5. Knippels Bridge. Cross Section of Bascule Span.

At the workshop the steel main trusses are rehabilitated by additional friction bolted profiles when needed. All steel is sandblasted and painted. The halves of the bascule spans are replaced and put in service. Hereafter the other side of the bridge can be rehabilitated.

In the meantime piers and side spans are repaired.

The old mechanical machinery is replaced by new hydraulic machinery controlled by a modern electronic system, operated by one man (instead of two men).

Wind Loads on Movable Bridges

Effets du vent sur les ponts mobiles

Windlasten auf beweglichen Brücken

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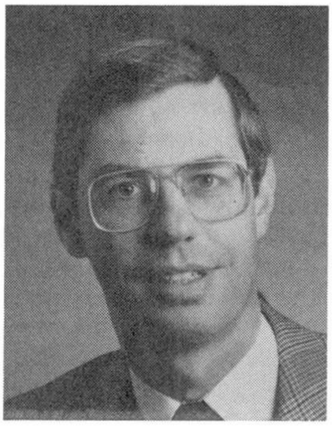

Arie Reij, born in 1947, took his master's degree in civil engineering at Delft Univ. of Technology. After having worked in research for more than 14 years, he joined the mechanical eng. div. of Rijkswaterstaat in 1989. Since 1 January 1991 he has been head of the Policy Analysis Section of the Civil Eng. Div. of Rijkswaterstaat.

SUMMARY

This paper examines the question of the maximum wind load at which it is safe to open existing movable bridges. For that purpose a draft procedure has been established for dimensioning the moving gear of bridges for wind load. The fundamental difference between a movable bridge and an ordinary structure had to be analyzed before these principles could be applied. A number of computer simulations were carried out with a stochastic wind load on a bridge deck, enabling the variation coefficient of the response and dynamic amplification factor to be derived.

RESUME

Cet article aborde la question de savoir jusqu'à quel niveau de la charge de vent les ponts mobiles peuvent encore être ouverts. Pour calculer cette charge maximale, on a mis au point une méthode en vue de dimensionner le mécanisme de commande en fonction de la charge due au vent. Pour pouvoir appliquer ces principes, il a fallu analyser les différences fondamentales entre un pont mobile et un pont fixe. Un certain nombre de simulations ont été effectuées par ordinateur pour mesurer la charge aléatoire due au vent exercée sur un tablier de pont, ce qui a permis de déduire le coefficient de variation de la déformation et le coefficient de majoration dynamique.

ZUSAMMENFASSUNG

In diesem Artikel wird der Frage nachgegangen, bis zu welcher Windlast bewegliche Brücken noch geöffnet werden dürfen. Bevor die demnächst erscheinende niederländische Vorschrift angewendet werden konnte, musste zunächst untersucht werden, worin der grundlegende Unterschied zwischen einer beweglichen Brücke und einer festen Konstruktion besteht. Es wurden ausserdem einige Computersimulationen mit stochastischen Windlasten auf den Brückenträgern durchgeführt, woraus der Variationskoeffizient der Verformung und der dynamische Erhöhungsbeiwert abgeleitet wurden.

1. INTRODUCTION

The Public Works Department (= Rijkswaterstaat) designs virtually all traffic bridges on public highways in the Netherlands as well as a large number of other bridges, e.g. over locks. Consequently, the Department is often asked what the maximum wind force is (expressed, for example, on the Beaufort scale) at which a particular bridge can safely be opened. The Regulations for the Design of Movable Bridges (VOBB) are of little use since they only give resulting wind loads. These regulations are currently being revised, and the old VOBB will be replaced by a completely new version based, inter alia, on the EURO-codes and the new Dutch TGB.
These two facts provided sufficient grounds for commissioning a study which was to answer the following two questions:

1. up to what wind velocity can existing bridges still be opened safely, taking into account each specific situation?
2. to what wind load should new bridges, and in particular their moving gear, be calculated to withstand?

2. WIND LOAD ON MOVABLE BRIDGES

2.1. Basic principles

The study of the design values for the wind load on movable bridges made use of the basic principles of the NEN 7600 series (Technical Principles for Building Structures, TGB), which will shortly come into force in the Netherlands. Bridges come under the general heading of "structures" and, in principle, should therefore comply with these standards.

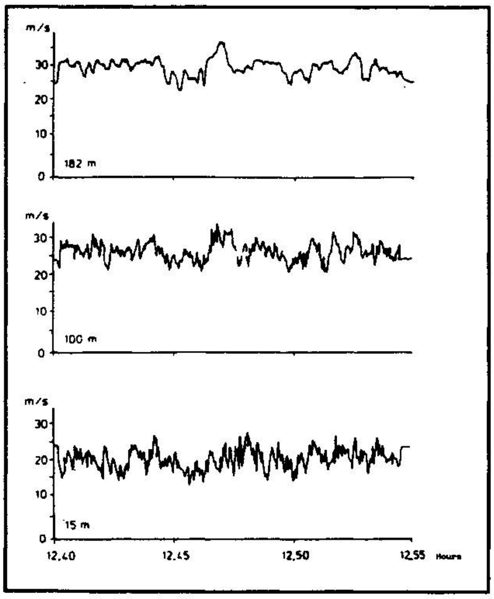

fig.1 Variation in windspeed at 15, 100 and 182m altitude respectively. Watch the decreasing amount of variation, as the altitude increases

As far as wind loads on bridges are concerned, this means, at any rate, that the principles for determining a design value for the wind load have already been defined. In addition, a number of influences which determine wind velocity have been ascertained. The most important are: height, location (coastal or inland), the ruggedness of the terrain and the way wind fluctuations are described. All that remains is therefore "simply" to establish the distinction between a movable bridge and a fixed structure. For this distinction to be defined properly, however, a number of points first need to be considered.

The first is fluctuations in wind velocity over time. If wind speed is recorded at a certain point, a pattern is obtained such as in fig.1. From this it can be concluded that it is possible to define wind speed in terms of an average and a variation with respect to the average. Both that average and the deviation (the gust) have a certain probability of occurrence.
Fig.2 shows this for gusts. It indicates that as the velocity in a gust increases, its probability declines. As a consequence, the (design value) of the windspeed is <u>coupled</u> at a certain probability.

In the TGB it was decided to derive the design values for wind load from the extreme hourly average wind velocity in a storm which occurs an average of once every 12.5 years; wind direction is assumed to be random. In the Netherlands the inland and coastal reference velocities thus become 20.5 and 26.0 m/s resp. Within that hour the highest average gust velocity for 3s is accounted for.

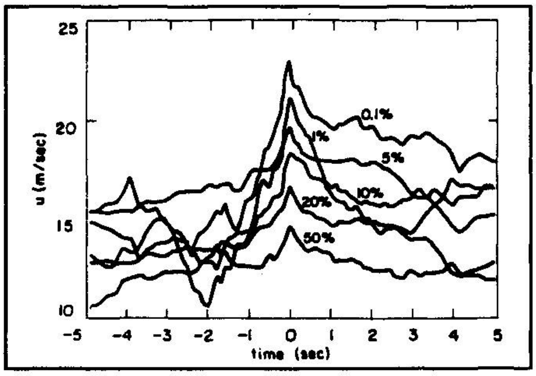

fig.2 Windgusts taken from a population of 1000, with indicated probabilty. As the maximum gustspeed increases, its probability decreases

The second point to consider is the dimensioning method given in the TGB.
The TGB is based on a so called: "semi-probabilistic" approach. This means that:

- dimensioning is based on a certain probability of reaching a limit state
- the uncertainties in o.a. the load and strength are taken into account by the use of partial safety factors.

By judiciously choosing the value of these factors for given levels of uncertainty, the desired probability of attaining a certain limit state can be found or, conversely, the required safety factor for a certain probability can be calculated. It is beyond the scope of this paper to examine exactly how this is done. It is important, however, to know the variation coefficient of the response (e.g. the torque on the motor shaft).

2.2. Numerical simulations

The nature of wind load is such that dynamic effects cannot be ignored. The dynamic response of the structure to a fluctuating load can usually be translated into a dynamic amplification factor. This means that (quasi-)static calculation can be performed. This approach is not always possible for complex problems, and it is necessary to resort to a completely dynamic calculation, the domain being either frequency or time.

Since dynamic influences were expected to have an important impact on the response of the moving gear, it was decided to carry out a dynamic calculation. For this purpose the model shown in fig. 3 was used, which consists of masses and rotational springs. In addition to the usual material damping of 0.7% between the first and second degree of freedom, viscous damping of 10% was applied between the 2nd and 3rd degree of freedom. This causes a damping of 2 to 2.5% for the lowest natural frequency, which closely corresponds to the damping effect of the spring buffer expected on the basis of measurements.

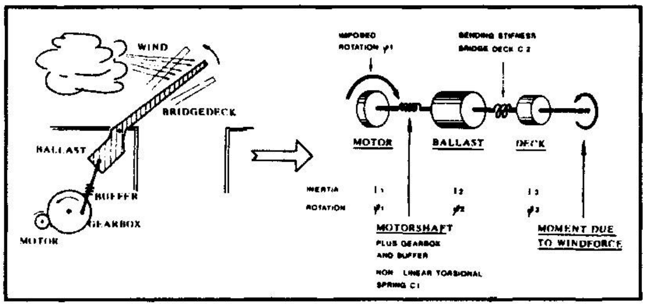

fig.3 Model used to determine the dynamic response of the bridge, as used for the numerical simulations

There is also additional damping, since an efficiency factor of 0.80 has been introduced for the torque transmitted by the gearbox, by applying an opposing torque.

Ideally a calculation in the frequency domain should be carried out, since the response spectrum can then be determined directly. However, in addition to having a favorable (= damping) effect on the response, the buffer also has the awkward effect of producing non-linear system behavior. It has a bi-linear spring characteristic and there is a certain amount of play at the start (fig. 4). It is therefore necessary to resort to a calculation in the time domain and therefore to perform simulations. In addition to wind, also the effects of acceleration and retardation, unbalance, normal braking and an emergency stop were taken into account.

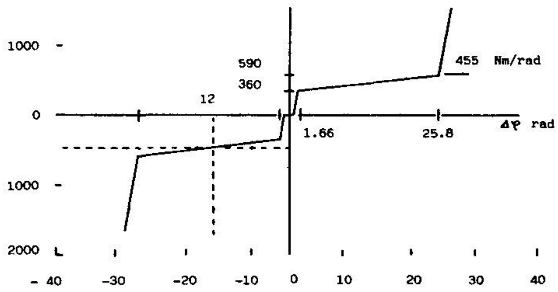

fig.4 Moment-rotation characteristic of C_1 in fig.3

The purpose of the simulations is to determine the response of the system including of the variations resulting from the uncertainties in the load. This therefore means that the variation which may occur in the wind load must be included in the calculation. The calculating of the torque on the motor shaft, for example, is as follows:

1. At a given average hourly wind velocity, wind spectrum and turbulence intensity, one (random) realization of wind velocities is generated.
2. The wind load on the drive mechanism is calculated for these wind velocities, for different opening angles of the fall. This corresponds to proceeding through an opening cycle in small timesteps.
3. For timestep, the response (in this case the torque on the motor shaft) is determined with the dynamic model.
4. The largest torque $M_{(max)}$ from a complete opening cycle is recorded.

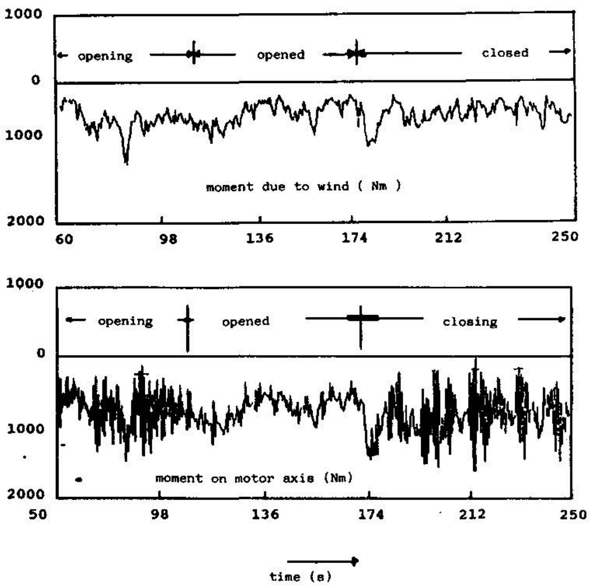

fig.5 Action moment due to wind (upper trace) versus response moment, as a function of time. Watch the dynamic increase of the maximum (torsional) moments.

Fig. 5 shows the wind velocities and the resulting response. To clarify the influence of the dynamic behavior, the wind velocity here has been converted into static action torque on the motor shaft. However, this is a linear transformation and the shape is therefore the same as that of the wind velocity.

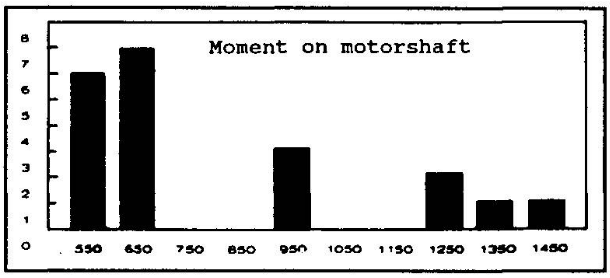

fig.6 Distribution of the torque moments on the motorshaft, obtained from the numerical simulations

By repeating this procedure a large number of times and presenting the results in a histogram, an estimate can be made of the average $\mu M(max)$ and the standard deviation $\sigma M(max)$ of the torque on the motor shaft (see, for example, fig.6). Armed with this knowledge, the partial load factor can be calculated, as indicated in 2.1.

The fact that there is also uncertainty in a number of quantities included in the original calculations as invariant is also taken into account. The figures in table 1 were therefore also used when determining the variation coefficient of (in this case) $M_{(max)}$. Once the variation coefficient of $M_{(max)}$ has been calculated, the required γ-value can be determined.

2.3. The difference between building structures and movable bridges

So far relatively little attention has been devoted to the differences that may exist between a normal building structure and a movable bridge. On the contrary, the basic principles for dimensioning building structures have also been declared applicable to a movable bridge. Nevertheless, it is necessary to examine the similarities and dissimilarities.

symbol	description	distribution	v
P	windpressure hourly average	Gumbel	0,20
C_t	windpressure coeff.	log. norm.	0,12
E	influence of obstacles in the environment	log. norm.	0,11
G	gustfactor	log. norm.	0,16

tabel 1. Overview of parameters used to determine the variation coefficient of the windresponse

When closed, a movable bridge is a "normal" (fixed) structure to which the basic principles of the TGB fully apply. When open, the bridge differs from, say, a building, in three essential respects:

1st The simulations have confirmed that the dynamic properties of the bridge and the moving gear have a significant influence on the response. The dynamic behavior may therefore not be neglected, and must be explicitly accounted for when considering varying loads, such as wind load This is done by applying a dynamic load factor. Guidelines therefore have been deduced for this purpose in the study.

2nd Since a bridge is only open for a limited part of its life, there is less risk of it being hit by a major gust than in the case of a building.

3rd A decision can be made not to expose a bridge to a certain wind load, simply by not opening it.

Point 2 assumes that there is an unlimited opening regime.
In principle, the bridge may be opened at any time, but the duration of the opening cycle may be so short that the chance of the maximum gust occurring in that period is thereby reduced. This is expressed in a reduction factor Ψ, applied to the design windspeed. In the case of an average opening duration of 3 minutes per hour, i.e. for 5% of the time, the factor is at least $\Psi = 0.8$.

In the case of point 3, there is a limited opening regime; i.e. if a particular wind force is forecast (e.g. Beaufort 9), the bridge is not opened as long as the warning lasts. The bridge will therefore be exposed to a lower average wind force than in the case of an unlimited opening regime. If the wind force is limited to below Beaufort 9, for example, the average wind velocity can be reduced by a factor of 0.70.

APPLICATIONS

3.1. Guidelines for new bridges

The work described above has resulted in a proposal for guidelines for the wind

load on movable bridges. The guidelines discuss a number of additional matters which could not be dealt with in the scope of this paper, such as: method of calculation to be used, shape factors and windspeeds to be used in case of a reduced opening regime.
The following limit states are distinguished in the proposal:

A. Failure of the transmission, or parts of it
B. Insufficient motor power, resulting from:
   - the exceeding of the average power available
   - the exceeding of the maximum power available (heat balance of motor)
C. Insufficient braking power

It is indicated which load combinations must be examined, which γ_{W}- and Ψ-factors must be used and how the dynamic load factor can be determined. A distinction is also made between an unregulated motor (for which the torque-speed curve is fixed by is vary nature) and a regulated motor, whose speed is controlled electronically.

As an example, the limit state for exceeding the maximum braking torque, in the case of an opened bridge is given below.

$M_{brake} \geq \gamma_{wind}\ \phi_{wind}\ M_{wind} + M_{unbalance}$, where:

M_{wind} = wind moment, calculated with the wind pressure according to the TGB and reduced by a factor $\psi = 0,8$
ϕ_{wind} = dynamic load factor, calculated according to the proposed guidelines
γ_{wind} = partial load factor for wind, in this combination = 1,1

3.2. Permissible wind load on existing bridges

Although this is the last subject to be discussed, it was the point of departure for the study. The very question to be answered was the maximum wind velocity at which a number of bridges over locks in the province of Zeeland might still be opened.
The results obtained allow this question now to be answered. It is necessary, of course, to take account of the actual situation regarding a particular bridge in terms of the strength of the moving gear, the maximum motor torque available and the braking torque to be applied.

As at the time of writing this paper, this job has not been completed, only an indication of the results can be given here.
It looks as if in the moseyed interesting case (that is the most important bridge), the magnitude of the braking torque is the decisive parameter. If it is too small, the bridge could be blown shut; if it is too high, the moving gear may become overloaded as a result of an emergency stop. Fortunately, however, the braking torque can be adjusted, so that the optimum value can be set. This leads to a maximum permissible reference wind most of 14 m/s. This value might be increased if the reliability with which its occurrence can be predicted is increased. In cooperation with the Royal Netherlands Meteorological Institute (KNMI) studies are being conducted of the extent to which an advanced wind-measuring system at the site of the bridge might contribute to this.

ACKNOWLEDGEMENT

The research described in this paper was carried out by TNO - Building Research in Rijswijk, the Netherlands. It was intensively supported and coordinated by a supportgroup consisting of specialists from Rijkswaterstaat, the Dutch Railways, Delft University of Technology and a Dutch engineering firm.

Designing of the Draw Bridge

Projet de ponts mobiles

Projektierung von Drehbrücken

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Georgy Stepanov, born 1916, received his diploma of bridge engineer at Leningrad Inst. for Engineers of Railway Transport. Since 1946, he has worked at State Inst. for Survey and Design of Bridges. As chief engineer, he supervised the construction of several steel and movable bridges. Georgy Stepanov is now a chief specialist in Dep. for Construction of Metal Bridges.

SUMMARY

Construction of movable bridges is one of the ways to solve the problem of overpassing a waterway. The designing of the draw bridge involves the traffic passage structures and the turning machinery. The main drives used for bridge opening may be of electromechanical or electrohydraulic type. The construction technology ensures uninterrupted navigation. The construction of the bridge is carried out without interrupting road traffic.

RESUME

Le pont mobile est une des solutions au problème du transport sur un cours d'eau. Le projet d'un pont mobile est une étude complexe comprenant des constructions civiles et mécaniques. La commande principale d'ouverture et de fermeture est électromécanique ou électrohydraulique. La technique de construction d'un pont mobile assure généralement une navigation continue, et au cours d'une rénovation, un trafic automobile continu.

ZUSAMMENFASSUNG

Die Drehbrücke stellt eine mögliche Lösung zur Wegkreuzung mit einer Wasserstrasse dar. Ihr Entwurf ist komplex, gilt es doch, die Funktion der Verkehrsüberführung mit der des Drehmechanismus in Einklang zu bringen. Der Antrieb kann elektromechanisch oder elektrohydraulisch erfolgen. Das Bauverfahren muss unbehinderten Schiffsverkehr erlauben, während eine spätere Instandsetzung bei laufendem Strassenverkehr erfolgen soll.

When constructing bridges over navigable rivers or other water-ways, it is necessary that free navigation be ensured with due regard for its further development. It is known that these requirements are met by observing the bridge clearance which depends on the class of the river.

Construction of a high-level bridge is seen as the simplest solution of the above problem. However, such an approach is not always justified and sometimes is even technically unfeasable.

Construction of the bridge with a movable span is another possible solution of the task related to the problem of overpassing a water-way. The height of other spans of the bridge may only slightly exceed the calculated navigation horizon level.

When deciding on the method to be used for spanning a water-way, one compares the above variants as to their carrying capacity, construction cost, maintenance charges etc.

Designing of the movable bridge is a sophisticated task. As a construction, the bridge is seen not only as a structure intended for carrying traffic load but also as the turning machinery of the bridge which includes electric equipment, signalling and communication facilities. Once actuated, the turning mechanisms lift the span to allow a clear passageway for vessels.

The profile and size of the bridge clearance of the movable spans in drawn position depend on the class of the water-way and are specified by appropriate items of the request for proposal. The bridge clearance of the movable span in bridged position as well as the height of the adjoining navigation spans will be determined proceeding from the local navigation conditions.

Preference is given to the following kinds of movable bridges,i.e. bascules (single-leaf and double-leaf), swing spans and the vertical-lift type.

The railway and combined-traffic bridges use mostly a vertical-lift system (Fig.1). Other systems may be used when explicitly specified by appropriate items of the request for proposal.
A single-leaf bascule railway bridge over the Neva river should be mentioned here to illustrate the case. The lifting span of the bridge gives 42 m clearance what is explained by high architectural requirements this particular bridge was to meet. This is a deck bridge, with the axis of rotation being stationary and the counterweight rigidly fixed. In bridged position, the axis of rotation is unloaded and the counterweight is not wedged.

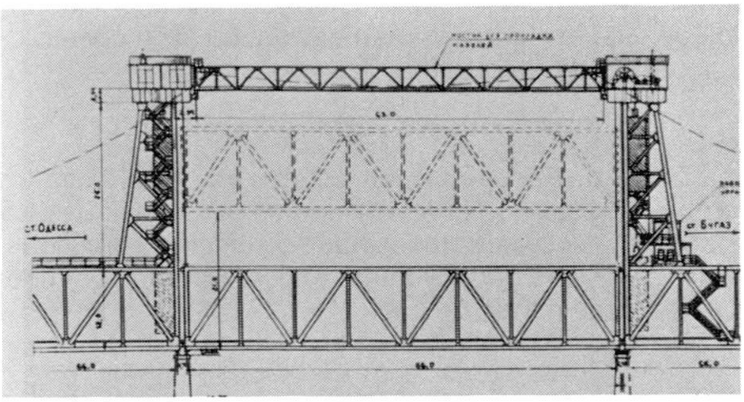

Fig.1 The vertical-lift bridge

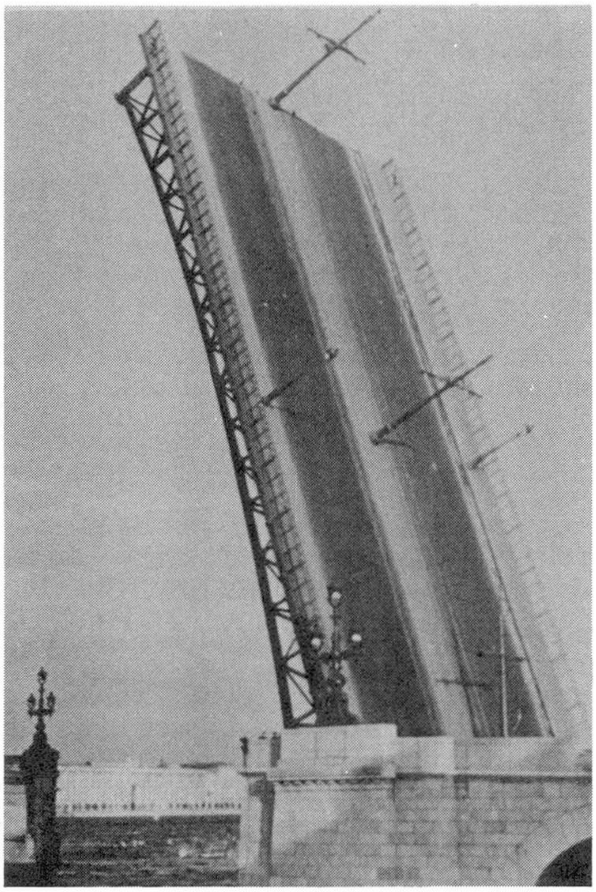

Fig.2 The single-leaf bascule bridge

The motor-road and town movable bridges are designed to employ a single-leaf bascule system for the clear span of up to 55 m (Fig.2) and a vertical-lift type for the larger span length. As examples of the long-span motor-road vertical-lift bridges we may mention here a bridge over the Severnaya Dvina river in the city of Arkhangelsk that was constructed in 1990, with a calculated length of the span of 84m, and a bridge that is now being constructed over the navigation pass of the dam, which is designed to protect Leningrad from floods, with a record-breaking span length of 120.45 m. These are deck-type bridges which have orthotropic roadway. The towers of the first bridge are of a box type and are mounted directly on the supports, with the counterweights arranged inside the towers. A movable span of the second bridge is of a unique structure: it has no towers - to lift the span to the height of 9 m (to get the bridge clearance of 25 m), the metal framework structures are used,which are located under the span structure in the piers of the movable span.

The prime mechanisms used to raise the bridge have, as a rule, three drives. These are the main drive, the reserve drive and the emergency drive.

The operation of the main drive is based on electromechanical or electrohydraulic principle. Operation of the reserve drive is similar to that of the main drive, or else it may work powered by an internal combustion engine. As a rule, the emergency drive is operated manually. As many-year experience shows, the latter may not necessarily be installed when the main drive is of electrohydraulic type.

The bridge drives may be operated either automatically or manually. These are equipped with interlock protective devices and signalling facilities to indicate the end positions of the span structure and drawing mechanisms.

Duration of bridge lifting depends on traffic intensity. When the road and waterway traffic is heavy and the bridge is to be opened many times a day, it takes 2 minutes to lift (lower) the bridge by means of the main drive (this procedure includes all operations on lifting and lowering the bridge). When the water-way traffic is low and the bridge is opened 1-2 times a day (usually at night) for a short period of time just to let the waiting ships pass, the duration of bridge lifting may be extended to 5 minutes.

In order to reduce the dead weight of the steel structures and to increase the resistance to corrosion, the rolled metal elements are usually made of low-alloy structural steel with the yield limit of 340 (35) or 390 (40) MPa (kgs/mm^2), which correspond to the standards used for the design of permanent bridges.

The movable span structures are designed to employ welded factory-made elements which are assembled with the help of welded, or combined bolt-and-welded, joints. The joints of elements working under stressed conditions should be designed to use high-strength bolts.

In order to facilitate the production and assembly processes, the parameters of movable bridges are so designed as to make them unified with the existing typical projects and factory standards.

The roadway of the movable spans of highway and town bridges are designed to employ steel orthotropic plates. The roadway is covered with asphalt, the layer of covering being no less than 45 mm thick. Steps must be taken to ensure adequate engagement between asphalt and metal surfaces as well as to protect the metal structures from corrosion.

It is recommended that thin-layer coating (15-20 mm thick) be used to cover the steel orthotropic plates of the single-leaf bascule bridges, with the clear span exceeding 40 m. It was revealed, however, that application of such coating was not a complete success. Therefore, its structure and technology need to be further improved.

Railway tracks on the movable spans are designed to be fixed directly to the steel orthotropic plate or to the metal ties, with the provision of electric insulation of the railway circuits. There should be railjoints at the ends of the movable span, and on a railway bridge there should also be rail locks.

In any position, the movable span must be balanced by counterweights.

During lifting operation, the vertical-lift and bascule spans become slightly unbalanced due to their own weight (overweight). The unbalance should be of such magnitude as to ensure that the movable span, when in bridged position, is snugged against the supports, thus eliminating a possibility of self-opening of the bridge during its exploitation.

In order to regulate the balance of the movable span, one may change the mass of the counterweight within the range of -3% to +5% of its calculated mass by removing or adding up demountable blocks.

Depending on the volume density required, the counterweights are filled with concrete, cast-iron concrete, or cast-iron castings of regular shape formed with the use of cement mortar.

As a rule, the counterweights for vertical-lift bridges are designed to be made of a monolithic concrete reinforced by a steel frame and supporting nets. The counterweights are equipped with devices which are used for hanging them up to the beams of the tower caps so that to remove the load from the carrying ropes (counterweight wedging).

The axes of rotation of the bascule bridges and the pivots of the swing bridges are usually unloaded and do not transfer the vertical load onto the supports. The unloading of the axes of rotation is achieved by the use of bearing members with swinging posts which are placed in one vertical plane. In the process of bridging they get engaged into mesh with the balance beams fastened to the main girders and raise the wing together with the axes of rotation to the height of 2-3 mm. The axes of pulleys on the vertical-lift bridges are unloaded by wedging the counterweights.

All bridges built after 1965 and the motor-road and town movable bridges which are being constructed at present employ a single-leaf bascule system with a rigidly fixed counterweight and a stationary axis of rotation which get unloaded in the normal position of the bridge. The maximum efficiency in this case is achieved when the electrohydraulic drive is used. This ensures a more rational arrangement of the movable span, a reduction in mechanisms mass by 25-30% as compared with an electromechanic drive, and a considerable reduction in the width of the supports along the frontal side of the bridge. Many-year experience shows that this system of the movable bridge has proved to be the simplest and most reliable one.

The hydraulic drive circuit incorporates the systems of interlock and tracking devices which provide reliable lifting of the bridge with the wind intensity of 6. Oil feed from pumping plants to the main pipe-line ensures synchronism in the operation of hydraulic cylinders, no matter how many pumps are used. Thus, one may regulate the speed of rotation of the wing ensuring a smooth start and stoppage.

The towers on the vertical-lift bridges are usually erected on adjoining stationary span structures. This system is less sensitive to deformations in supports of the movable span as compared with the case when the towers are mounted directly on the supports. Moreover, this system makes it possible to use a more simple method to eliminate aftereffects of deformations in supports. A schematic diagram and structural solutions of these bridges have many things in common: counterweight ropes play the role of the working ropes whereas synchronism in the movement of the ends of the span structure is achieved by a system of mechanisms equipped with an "electric shaft".

In addition to staircases, every tower of the vertical-lift bridge should have an elevator to take the personnel and necessary equipment to the control and machine rooms located at the top of the tower.

In order to facilitate the inspection and maintenance of the turning machinery of the vertical-lift bridges as well as to exclude the use of underwater-laid cables within the movable span, it is recommended that a special cable (flying) bridge be provided, especially on the railway bridges.

When constructing the movable bridges, a certain technological sequence in spanning the bridge must be observed in order to ensure uninterruped navigation: the navigation span is bridged upon the completion of the movable span, or else during an inter-navigation period.

Reconstruction of movable bridges is carried out without interruption in navigation. As a rule, urban bridges are also reconstructed without interrupting the intertown traffic. For this purpose, a temporary retractable bridge is constructed to run in parallel with the existing bridge. The experience of exploitation of two bridges over the Neva river in Leningrad shows that this system is the best one to satisfy the conditions of temporary exploitation, and therefore may be recommended for duplicating under similar conditions, for such a solution of the problem is found to be a most simple and reliable in use.

The principal technical solutions implemented by the Bridge Design Institute during the 60th were both progressive and original. Some of the projects were accomplished for the first time in the bridge construction practice. Therefore, difficulties and even failures were unavoidable. Sometimes these resulted in emergency situations during the construction and exploitation of the motor-road and town bridges, mostly of bascule type.

Here are some examples to illustrate the above: a complete fall of the asphalt covering from the wing during a trial lifting of the bridge; two failures in the system of hydraulic drive, one of these being so serious that a hydraulic cylinder was to be replaced a self-opening of the bridge during the storm wind.

A very serious failure occurred in 1982 on a double-leaf movable bridge over the Neva river in Leningrad, after 18 years of its exploitation: a counterweight (750 tons) collapsed from the left-shore wing due to formation of numerous defects in the welded joints of the counterweight structure following the lack of proper maintenance and failure in taking necessary measures to eliminate the defects. Some structural mistakes made at the design stage also contributed to the above accidents. As to the right-shore wing, once the structural and some minor defects were eliminated, the wing has been exploited without any restrictions.

The fact that the Institute has analyzed the causes of the above accidents is an important factor which shall help increase dependability of the movable bridges to be designed.

Lengiprotransmost is a leading institution in this country in the field of complex design of the movable bridges.

With a view of further development of the existing norms in the design of permanent bridges, Lengiprotransmost has worked out a "Guide for the designing of movable bridges" which replaced the earlier edition of 1972. This Guide describes the experience of the Institute in the field of designing the movable bridges as well as the native and foreign experience in the bridge construction and bridge exploitation.

Revitalizing 100 Year Old Movable Railroad Bridges

Revalorisation d'anciens ponts ferroviaires mobiles

Ertüchtigung 100jähriger beweglicher Eisenbahnbrücken

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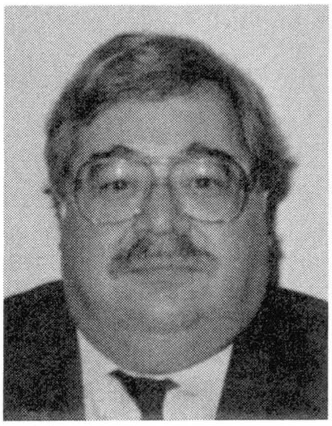

Charles Minervino holds B.S. and M.S. degrees in Civil Engineering from Stevens Institute of Technology, Hoboken, NJ. He has 25 years experience in the inspection, evaluation, rating and rehabilitation of major bridges, specializing in the design of complex rehabilitations of fixed and movable, highway and railroad bridges, and inspection of high level, long-span bridges.

SUMMARY

Three major movable bridges, two rolling lifts and one swing span, on a four track catenary powered railroad, constructed in the late 1800's, were in need of extensive rehabilitation to continue to provide reliable service. This paper describes the design and construction methods used to accomplish the upgrading of the structural, mechanical, and electrical systems within the restrictive confines of an operating railroad and active navigational channels. The use of modern design, fabrication, and construction technology revitalized these 100 year old structures.

RESUME

Trois ponts mobiles importants – deux ponts roulants et un pont à bascule – situés sur une ligne de chemin de fer alimenté par quatre caténaires de voie, tous construits vers la fin du siècle dernier devaient être rénovés afin de continuer à assurer leur service. Cet article présente les études et les méthodes de construction adoptées pour moderniser les systèmes structuraux, mécaniques et électriques dans les limites restrictives d'une voie ferrée de service et de canaux de navigation en activité. Le recours à des techniques modernes de calcul, de fabrication et de montage ont permis de revaloriser ces ouvrages datant de plus de 100 ans.

ZUSAMMENFASSUNG

Drei grössere bewegliche Brücken – zwei Hubbrücken und eine Drehbrücke – auf einer vierspurigen, kettenangetriebenen Eisenbahnlinie stammen aus dem späten 19. Jahrhundert und bedurften einer umfangreichen Ertüchtigung für die weitere zuverlässige Nutzung. Der Aufsatz beschreibt den Entwurf und das Vorgehen beim Umbau der tragenden, mechanischen und elektrischen Systeme unter fortlaufendem Eisenbahn- und Schiffsverkehr. Moderne Technik, Fertigung und Montage verhalfen den 100 Jahre alten Bauwerken zu neuem Leben.

INTRODUCTION

Metro-North Commuter Railroad's New Haven line is a four track catenary powered rail line carrying New York - New Haven commuter traffic and AMTRAK's Washington-New York-Boston service thru Connecticut. The river crossings on the line are movable bridges built in the late 1800's. A.G. LICHTENSTEIN AND ASSOCIATES, INC. (AGLAS) was engaged to perform an in-depth inspection, analysis, design and inspection of construction for the rehabilitation of the structural, electrical, and mechanical systems of three movable bridges:

- Cos Cob Bridge over the Mianus River - twin two-track Scherzer rolling lift bascule deck girder bridges.
- Walk Bridge over the Norwalk River - a four-track rim bearing deck truss 200' span swing bridge.
- Devon Bridge over Housatonic River - twin two-track Scherzer rolling lift bascule thru truss bridges.

AGLAS engineers conducted an in-depth, hands-on inspection of the structural, mechanical, and electrical systems to establish the physical condition of the elements. The program included measuring deterioration of structural members, materials sampling, non-destructive testing, review of bridge operation, evaluation of wear on mechanical components, and evaluation of wiring and electrical equipment.

Following the in-depth inspection, the structural, mechanical and electrical elements were evaluated. The rehabilitation of the structural steel was geared toward strengthening the existing members or replacing deteriorated components rather than replacing entire members. Many of the mechanical components were suitable for reconditioning rather than replacement. The electrical systems were obsolete, and replacement was necessary.

The deteriorated conditions and the construction methods used to accomplish the revitalization of these three complex structures while maintaining high-volume rail traffic are described herein.

COS COB BRIDGE OVER MIANUS RIVER - COS COB, CONNECTICUT

The original main drive operating system of the twin, deck girder, Scherzer Bascule Bridges consisted of electric motors located on the Operator's House structure, and a lengthy configuration of shafts, couplings and gears running to a rack and pinion located at the centerline of each bascule. The rack is attached to an operating strut pinned to the centroid of the bascule. Rotation of the pinion pulls the operating strut horizontally, rolling the segmental girders of the bascule on the track girders and lifting the span.

The long gearing and shaft system, which included a bevel gear set, had proven to be very inefficient. Inspection revealed that the shaft bearings for the main drive system exhibited excessive clearances, while the gears were only slightly worn. Excessive wear was present at the shaft bearings of the toe and heel lock systems. Lubrication of the machinery had been neglected at several locations where access was difficult.

The original electrical system, built to be powered by a 25 cycle source, had been obsolete since the railroad generating plant ceased to operate. As a temporary measure, motor/generator sets had been installed on the bridge to take standard 60 cycle power from a public source and convert it to 25 cycle power. The electrical system was in generally poor condition, and it was increasingly difficult to find spare parts for the 25 cycle equipment.

The in-depth inspection of the structural steel revealed: severe bottom flange deterioration of track girders and supporting girders, due to spray from the choppy salt water hitting the piers; worn lugs and pockets of the track and segmental girder tread plates, which permitted slippage and misalignment of the bascules as they operated; severe deterioration of steel housing the counterweights; and cracks in the track girder tread plates and their connecting angles. By current design criteria for railway bridges (A.R.E.A.) the tread plate and web thicknesses of the track and segmental girders were inadequate to support the rolling load of the bascule.

Evaluation of the structure produced a rehabilitation scheme that would include: elimination of much of the machinery by placing new motors under the span adjacent to the bascule heel; reconditioning machinery that would remain; replacement of the electrical equipment with a solid state, computer controlled, 60 cycle system; installation of additional side plates and new tread plates on the segmental girders; replacement of the track girders and repair of the supporting girders and counterweights.

The extensive repair work required that the two bascule bridges be inoperative for a three to four month period. The bridges were scheduled for rehabilitation in the closed position during the winter months, when the navigation restriction would have minimal impact. Disruptions to railroad operation were limited to single track outages, and brief periods of double track outage for jacking of the bascule spans.

To permit rehabilitation of the segmental girders and replacement of the track girders, two temporary supports were erected below the existing cross girder. Hydraulic jacks were connected, through a manifold, to an electrically operated pump. Dead load was transferred to the temporary supports by lifting the bascule span 13 mm±, shimming between the cross girder and supports, and lowering the span. This operation was conducted during a two hour period of double track outage. The heel locks were shimmed 1.6 mm to develop full bearing.

Removal and replacement of the track girders and the outer 0.533 m of the segmental girders and repair of the support girders then proceeded. Rehabilitation of a segmental girder involved: removing the existing 51 mm thick tread plate, web stiffener angles and side plates; cutting off and replacing the outer 0.483 m of the circular web; and drilling new holes in the upper portion of the existing web to fasten new, deeper web side plates. The new 127 mm thick tread plates of the track and segmental girders had 51 mm deep, interlocking lugs and pockets. As it would not be permissible to jack up the bascule 51 mm, the segments of track and segmental treads that engage when the bridge is in the closed position were mated and lifted into position together.

To rehabilitate the steel housing the counterweights, it was necessary to remove the counterweight blocks. Each counterweight consists of 285± cast iron blocks, ranging between 1335 N and 20.47 kN. The blocks were lifted out and placed on a track mounted cart that transported them to a storage area.

New machinery and maintenance platforms were suspended from the deck girder span adjacent to the heel of the bascule. The existing main drive machinery that was to remain, as well as the toe and heel lock machinery, was disassembled and reconditioned. To permit disassembly of the toe locks, the toe end of the bascule was temporarily anchored with threaded rods coupled to existing lock pin anchor bolts. A temporary strut was installed in place of each heel lock arm.

The pin assembly and the guide roller and support roller assemblies of the operating strut were replaced. Shafts and shaft couplings for the toe lock drive were also replaced. The bearing journals of the shafts were reground. Shaft bearings were rebored and received new bushings and grease fittings. The heel lock arms received new grease fittings and cast steel fillers. The machinery was reassembled in the field using new turned bolts. New air buffers were installed. New motors and motor mounted pinions and brakes were installed for the main drive system. New gearmotors were installed for the toe and heel locks.

New motor control equipment and a computerized control panel were installed in the Operator's House. The control panel monitors and displays on a screen the entire sequence of operations. If a malfunction occurs, the control system stops operations and identifies the inoperative component. If desired, the bridge operations may be controlled manually from the control panel or from a portable keyboard and screen when plugged into jacks provided at the main drive, toe lock and heel lock machinery.

DEVON BRIDGE OVER HOUSATON RIVER - DEVON, CONNECTICUT

An In-Depth inspection of the twin thru-truss Scherzer Bascule Bridges revealed these serious deficiencies: cracks in the angles connecting tread plates of segmental girders and track girders; the drive pinions had come into bearing contact with the racks, causing the pinion shafts to bend; severe deterioration of end floorbeams, end brackets, floorbeam hangers at track level and of the machinery platform floor system; and significant toe heavy imbalance of the bridges. Additionally, major reconditioning of the main drive and toe lock machinery was necessary to correct the effects of wear. The track and segmental girder web plates and side plates were too thin for the bearing forces exerted by the rolling load of the bascule.

Extensive repairs to the segmental and track girders required that each bascule bridge be inoperative for a two to three month period. To minimize restrictions to marine and rail traffic, the bascules were repaired one at a time. Most of the work was performed with the bascule held by temporary struts in the fully open position. Rail traffic was handled by the two tracks on the twin bascule. Marine traffic was restricted only during repair of the rear segment of track and segmental girders. During this phase of repair, opening of the bascule span required 24 hours advance notice.

The Contractor used two track mounted hydraulic cranes to perform the work. A 267 kN crane, positioned on the track girder span, erected the temporary struts and handled removal and replacement of structural steel and machinery.

The tread plates and outermost side plates were removed from the track and segmental girders. New side plates were added. Removal of the existing tread plates revealed that the bearing edge of the existing web and side plates was worn as much as 9.5 mm. Using the newly installed side plates as a guide, the existing plates were built up with weld metal and ground smooth to develop a uniform bearing surface. The original intention was to re-use the existing tread plates. When they were removed however, inspection revealed cracks on the interior face, and wear of up to 3.2 mm at the surface mated to the web and side plates. New tread plates were fabricated and installed with new connection angles.

The new segmental girder tread plates were fully bolted into place. The new track girder tread plates were only partially secured with bolts for the bascule to be test operated. During the test runs, the lugs of the track girder tread plates were aligned with the pockets of the segmental girder tread plates; and the toe end of the bascule was aligned with the bearings. The first test closure of the bascule produced a 19 mm misalignment of the toe end. The alignment was corrected by a slight longitudinal adjustment of the tread plates on one side of the bascule.

The main drive and toe lock machinery was completely disassembled and reconditioned while the bascule span was strutted in the open position. The pinion shafts were replaced. They had been bent as a result of the wearing of the mating surfaces of the track and segmental girder web plates and tread plates. The wearing had effectively lowered the bascule span and brought the drive pinions into bearing contact with the racks. A cracked differential assembly frame for the south bascule and a section of the main drive shaft of each bascule were also replaced.

The rack support beams were replaced and the racks adjusted to provide proper clearance with the pinions. The bearing journals of the shafts were reground. Shaft bearings received new bushings and grease fittings. Pinions, reduction gears and bearings were rebored to a true cylindrical bore. The gear keyways were recut and fitted with new keys. The machinery was reassembled in the field using new turned bolts. While machinery reconditioning was in progress, the machinery platform members were replaced or repaired with bolted plates and angles.

The worn contact areas of toe lock hooks were built up with weld metal and reground. New grease fittings were attached. The contract drawings provided alternative details for replacement and for reconditioning of the air buffers. After disassembly of the existing buffers, an inspection revealed that they were reconditionable. The inside cylindrical surface was honed to remove degradation; new piston rod assemblies, piston rings, piping and pressure regulating valves were installed. New strike plates were bolted to the pier seat.

Steel repairs were made to return the structure to a Cooper E 72 load capacity. Deteriorated floorbeam hangers were reinforced with steel plates fastened with high strength bolts in existing rivet holes and some new field drilled holes. End floorbeams and end brackets, deteriorated by de-icing salt used at the floor breaks, were seriously deteriorated. The top flange of floorbeams was replaced. End brackets at the heel end were replaced, those at the toe end were reinforced. Structural repairs to the eyebar truss approach spans were made during the double track outage afforded by the bascule rehabilitation.

At the completion of repair work, the bascule spans were rebalanced. Strain gages were placed on the pinion shafts. Strain readings were recorded and torque in the shafts computed for the full operating cycle of the bascule. Computations were made to determine the center of gravity of the span. Based on the torque values and center of gravity computations, counterweight blocks were relocated and additional weights were added to yield optimum balance during operation.

WALK BRIDGE OVER NORWALK RIVER - NORWALK, CONNECTICUT

Operational problems developed with this Railroad Swing Bridge, and became a major concern to the owner. Deformed rollers and worn tread plates, on which this rim bearing swing span is supported, became unreliable and unrepairable. Obsolete motors and electrical equipment added to the uncertainty of the system. Several structural elements of the Swing Span required major rehabilitation.

After a thorough inspection and evaluation of structural, mechanical and electrical components, it was decided to replace all 90 steel roller wheels, and the upper and lower tread plates with new components of higher strength materials. Some shafts and gearing would also be replaced; most of the machinery would be reconditioned. A solid state, computer controlled, electrical system would take the place of the existing equipment.

The extensive repairs required that the swing span be inoperative for a one to two month period. A system was developed to jack and shore the 8900 kN ton deck truss swing span to permit replacement of the roller wheels and tread plates while maintaining Railroad operations. It was necessary to reinforce some of the existing swing span support girders at shoring points. The work was scheduled during April and May, when the navigation restriction would have minimal impact.

Timber and steel cribbing was erected at six locations on the pivot pier. At each location, a group of jacks was connected, through a manifold, to an electrically operated hydraulic pump. To prevent overjacking at any jacking location, each pump had a relief valve set at 150% of the expected load. During a four hour period of track outage, in the early hours of a Saturday morning, the span was lifted and shimmed 9.5 mm. The wedges at both ends of the span were shimmed to develop full bearing. Railroad crews adjusted the rail profile by shimming under the rail plates for a distance of 50'± on both sides of the span. The roller wheels, tread plates and ring gear were removed. The new bottom tread plate/ring gear assemblies were set to grade and to radius measured from the center of rotation. The new upper tread plate was bolted to the drum girder. The 90 roller wheels and connecting bars were then installed. The roller wheel axles were threaded to permit radial adjustment of the wheels.

The vertical drive shaft assemblies, consisting of the shaft, bevel gear, pinion gear and bearings; and the main horizontal drive shaft assembly, were taken to the Contractor's shop. One of the vertical shafts, bent by attempts to operate the span when the wheels were iced up, was replaced. The bearing journals of the other shafts were reground. Pinion gears were replaced. Shaft bearings received new bushings and grease fittings. The machinery was then reassembled in the field using new turned bolts. New motors, motor mounted brakes and pinions, reduction gears and motor supports were installed.

A new centering device was installed at each end of the span. Each new device consists of a motor operated, tapered, vertical, centering pin that is driven down through a pair of rollers mounted on the pier.

The shafts, bearings and linkages of the wedge drive and rail lifter systems were disassembled and reconditioned in the same manner as the main drive system. New motor assemblies were installed.

New motor control equipment was installed in a new room. The computerized control panel, installed in the Operator's House, controls and monitors the entire sequence of operations. In the event of a malfunction, the control system immediately stops operations and identifies the inoperative component. The bridge operations may be controlled manually if desired. New submarine cables were placed in a riverbottom trench between the Operator's House and pivot pier.

Most of the structural repair work, such as replacement of flange angles of truss members and stringers; and replacement of bracing members, was conducted during single track outage. Replacement of expansion bearings for the approach span trusses was performed with the tracks in service, supported by shoring. Replacement of the swing span end floorbeams and end brackets, required a two week double track outage. Major structural/mechanical/electrical components were moved by a rail mounted hydraulic crane. Other materials were transported manually or with the aid of a boat and chain hoists.

Through the use of modern engineering design and construction techniques, these three aged movable bridges were restored to full operation and revitalized for continued service. The $16,000,000 construction project took two years to complete.

The New Galata Bascule Bridge at Istanbul

Le nouveau pont basculant Galata à Istanbul

Die neue Galata-Klappbrücke in Istanbul

Reiner SAUL
Senior Supervising Eng.
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Stuttgart, Germany

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Managing Director
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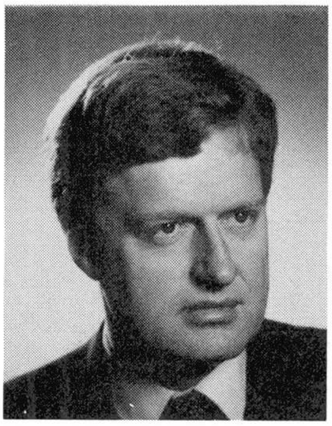
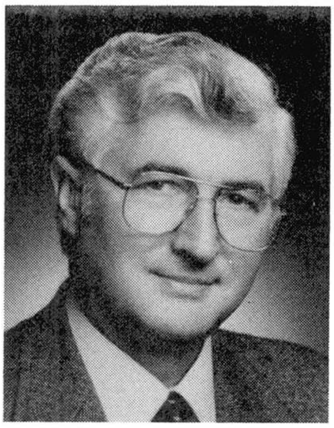


Reiner Saul, born in 1938, Dipl.-Ing. of Civil Engineering, Univ. of Hannover in 1963. Four years with a steel contractor, since 1971 senior supervising engineer with Leonhardt, Andrä & Partner. He was responsible for the design, technical direction and checking of numerous long-span bridges, including also major rehabilitation works.



Wilhelm Zellner, born 1932, got his civil engineering degree at the University of Vienna, Austria. For two years he was in charge of the supervision of a big prestressed concrete viaduct in the Vienna Woods. In 1962 he moved to Leonhardt & Andrä to Stuttgart and in 1970 he became partner in this firm.

SUMMARY

The centre piece of the New Galata Bridge at Istanbul is a bascule bridge with a clear span of 80 m and a width of 42 m. It consists of 4 flaps with TT-section and orthotropic deck. The two flaps corresponding to the traffic direction are coupled at the centre for shear and moments. Opening and closing of the flaps is done by hydraulic cylinders and takes only 3 minutes. The counterweight of heavy weight concrete is not the traditional pendulum, but is fixed directly to the steel structure.

RESUME

La partie centrale du nouveau pont Galata, Istanbul, est un pont basculant de 80 m de portée et de 42 m de largeur. Le tablier comporte 4 nervures basculantes, dont la section est en forme de TT, et une dalle orthotrope. Le joint central à mi-portée permet la transmission des efforts tranchants et des moments. L'ouverture et la fermeture du pont sont assurées hydrauliquement et ne durent que 3 minutes chacune. Le lourd contrepoids en béton ne correspond pas à la conception traditionelle en pendule, mais est directement relié à la structure métallique.

ZUSAMMENFASSUNG

Das Mittelstück der neuen Galata-Brücke in Istanbul ist eine Klappbrücke mit 80 m Spannweite und 42 m Breite. Sie besteht aus 4 Klappen mit TT-Querschnitt und orthotroper Platte. Die Verriegelung an den Klappenspitzen überträgt Querkräfte und Momente. Die Klappen können mit hydraulischen Zylindern in nur 3 Minuten geöffnet oder geschlossen werden. Das Gegengewicht aus Schwerbeton ist kein übliches Pendel, sondern mit der Stahlkonstruktion verbunden.

1. INTRODUCTION

The New Galata Bridge across the Golden Horn links the quarters of Eminönü and Karaköy directly on site of the existing steel floating bridge built in 1912.

The 42 m wide bridge, Fig. 1, consists basically of

- the centre bascule bridge with a clearance of 80 m and the corresponding bascule bridge piers, as described in more detail hereunder.
- the double deck approach bridges with 7 spans of 22.3 m each, with the road and light railway traffic on the upper deck and shops, restaurants and the like on the lower deck, Fig. 2.
- the abutments.

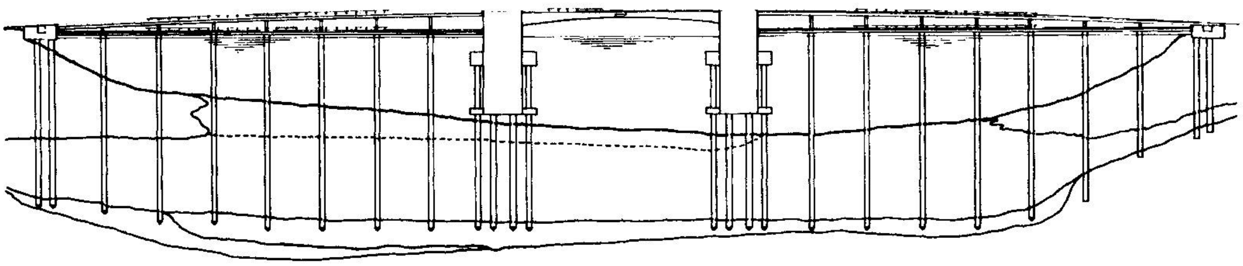
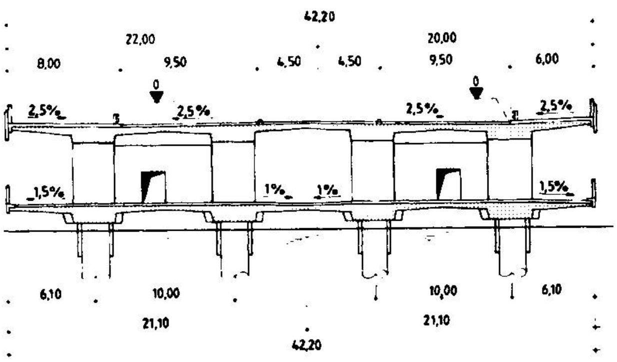


Fig. 1: General Layout



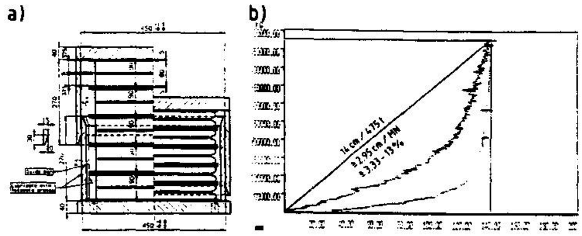


Fig. 2: Cross-Section of Approach Bridge

Fig. 3: Earthquake Buffer
a) Design b) Hysteresis

In between these 2 x 3 elements, buffer bearings are provided, which consist of rubber discs and have pronounced hysteresis, Fig. 3.

Due to a water depth of up to 40 m and poor soil of another 40 m, the bridge is founded on driven or drilled hollow steel piles, with a diameter of 2 m, a wall-thickness of 20 mm and cathodic corrosion protection.

2. DESIGN

2.1 General

The free span of 80 m and the total width of 42 m render the bascule bridge the world's largest, Fig. 4. The total length of the flaps, 54.5 m each, is divided by the axis of rotation into 2 cantilevers of 42.8 and 11.7 m.

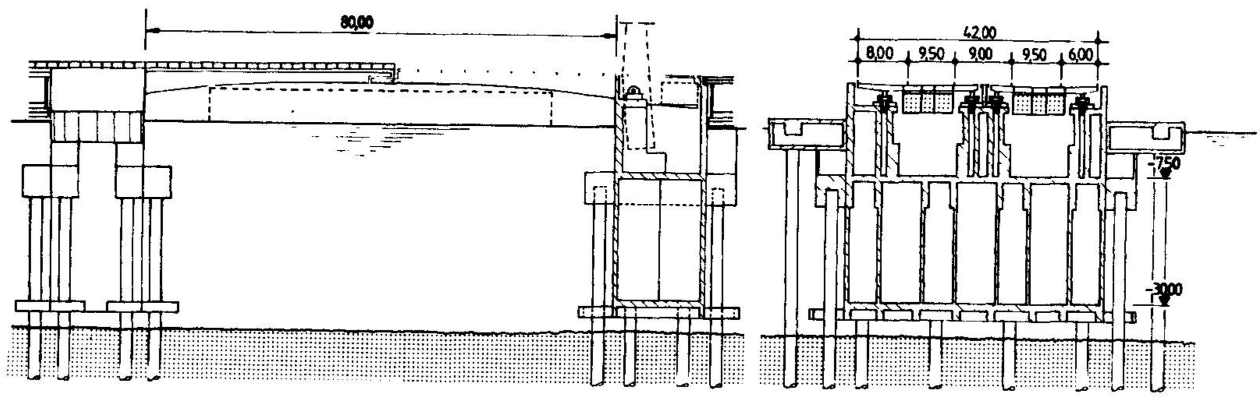
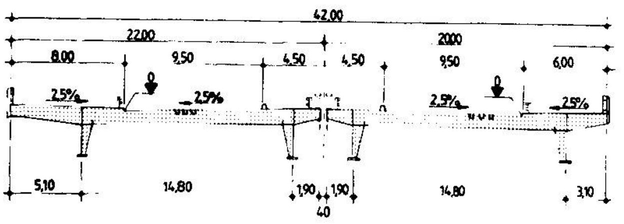


Fig. 4: Bascule Bridge, General Arrangement



Due to the unsymmetric arrangement of the pedestrian footways, the twin flaps have widths of 22 m and 20 m respectively, Fig. 5.

Fig. 5: Bascule Bridge, Cross-Section

2.2 Bascule Bridge

2.2.1 Steel Structure

The steel structure consists of
- the deck plate, with a minimum thickness of 12 mm in the roadway area and 10 mm in the walkway area,
- the trapezoidal ribs,
- the cross girders at regular intervals of 3.96 m; special cross girders are needed at the centre and at the rear,
- the main girders, with a depth varying between 2.3 and 5 m, and a 800 mm wide bottom chord with a thickness varying between 30 and 90 mm.

The total steel weight is 1600 tons corresponding to 350 kg/m².

The wearing surface is a 30 mm thick epoxy modified asphalt layer.

2.2.2 Mechanical Equipment

The main bearings are busher bearings, transverse forces are absorbed by both bearings. At the rear end, a neoprene bearing takes the uplift forces and a pin moved by a hydraulic cylinder the downward forces. At the centre, a double shear connection - retractable by hydraulic cylinders - assures the transmission of shear forces as well as of bending moments, Fig. 6.

The opening and closing of the bridge is performed by a hydraulic cylinder with a diameter of 650 mm, a length of 5 m and a stroke of 4.5 m. In order to open or close the flaps in 3 minutes, a maximum oil volume of about 5 litres per second is needed.

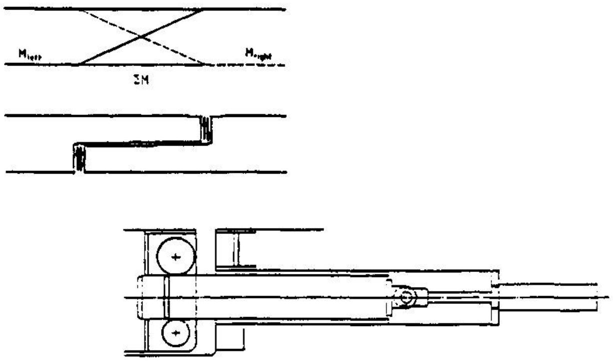


Fig. 6: Centre-Joint Interlocking

2.2.3 Counterweight

The counterweight is fixed by stud shear connectors and 20 prestressing bars Ø 26 mm St 1080/1230 through intermediate longitudinal girders and specially designed cross girders directly to the rear arms. The dimensions of the counterweight are about 7.3 x 3.8 x 9.8 m, its specific weight is 35 kN/m^3 and its total weight - variable from flap to flap - about 1000 tons.

2.3 Piers and Foundation

In the design of the bascule bridge piers, two contradictory requirements had to be fulfilled: For the absorption of the ship impact they had to be stiff and for that of the earthquake flexible. This could be achieved by a pier going down to the seabed and founded on 12 piles, which are fixed to the pier between -13 m and -7,5 m and elastically supported at -32 m, Fig. 7.

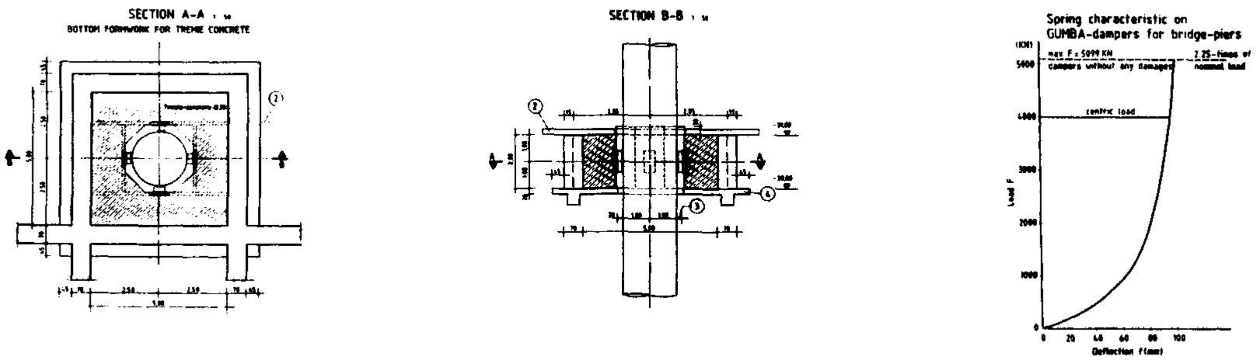


Fig. 7: Bearings at -32 m

In order to avoid an overloading of the piles - or additional piles - the piers are made hollow. In spite of being exposed to a water pressure of up to 35 tons/m^2, the pier walls are not waterproofed, but are reinforced for a crack width of w_{95} = 0,2 mm.

The piles of the bascule bridge pier are filled with tremie concrete B 35 and reinforced in the upper part. They are designed as composite columns, shear connectors were needed at both ends only.

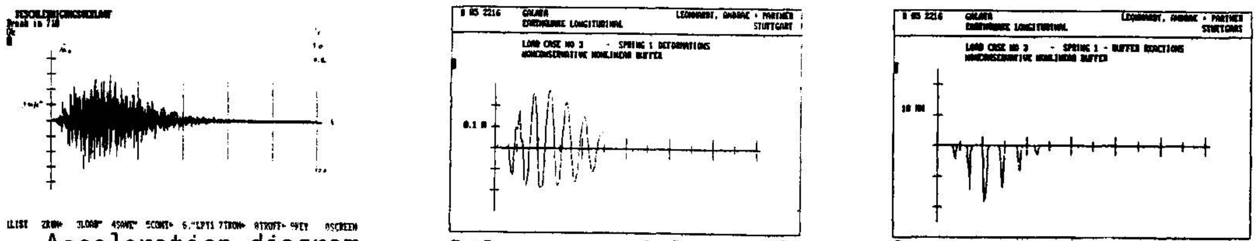
3. SPECIAL ASPECTS OF DIMENSIONING

3.1 Earthquake

The bridge had to be designed for an earthquake with PGA = 0,35 g. For the seismic design, two different approaches were used:

In a first step, a response spectrum calculation was performed, assuming that the different elements of the bridge are completely independent in the longitudinal direction. This calculation was performed for closed flaps, opened flaps and erection stages.

In order to determine the displacements of bearings and jointings and the forces acting onto the buffers, a time-history calculation was performed in a second step. For this calculation, up to 6 time-acceleration diagrams, compatible with the energy content of the response spectrum were generated. The results are given graphically, Fig. 8.



Acceleration diagram Dilatation of Spring 1 Spring 1 Force

Fig. 8: Time-history calculation

3.2 Ship Impact

The bridge had to be designed for the head on impact of a 8000 dwt ship sailing at 2.5 m/s. The corresponding impact force is, according to the "Nordic Road Council Regulations for Ship Impact"

$$P_{[KN]} = 500 \times \sqrt{dwt} = 500 \times \sqrt{8000} \times 1.05 = 40{,}000 \text{ kN}.$$

As a consequence of an eventual ship impact, the loss of buoyancy of the upper or lower part of the pier due to breaching had also to be considered.

As the bascule bridge could, of course, not be designed against ship impact, two worst case scenarios were investigated, Fig. 19:

- the formation of a hinge in front of a pier,
- the loss of a flap between this hinge and C.

These scenarios did not lead to a loss of the other flap and the rear arm with the counterweight respectively.

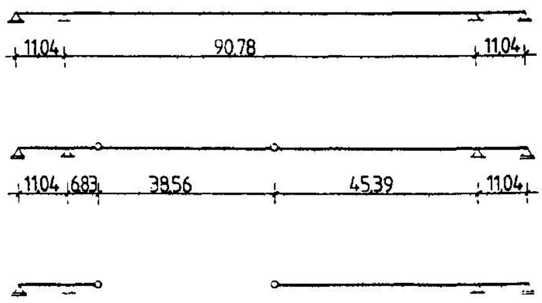


Fig. 9: Worst Case Scenarios

4. CONSTRUCTION

4.1 Piers

The bascule bridge piers were built by the "Lower Slab" method, Fig. 10.

Utmost care had to be given during the complete lowering procedure to the equilibrium between

- weight of the structure, including inside ballast water,
- force in the lowering equipment and the capacity of the piles,
- uplift forces due to bouyancy, increased by air pressure for special steps.

The pier was lowered by a total of 60 DYWIDAG jacks with a nominal capacity of 0.5 MN each, acting onto threaded bars ∅ 36 mm of St 835/1030.

The lower pile bearings were provisionally fixed to a steel structure which was lowered together with the pier. Rollers kept the steel structure in position with respect to the piles and so adjusted the pile driving tolerances. After the pier had reached its final position, the gap between the steel structure and the walls of the pile anchorage was filled with tremie concrete by divers.

In oder to pour the reinforced concrete of the upper anchorages in dry, the anchor boxes of the piles were extended up to the sea level. The lower part of the plug was first poured as tremie concrete to seal the pit, and later the water was pumped out.

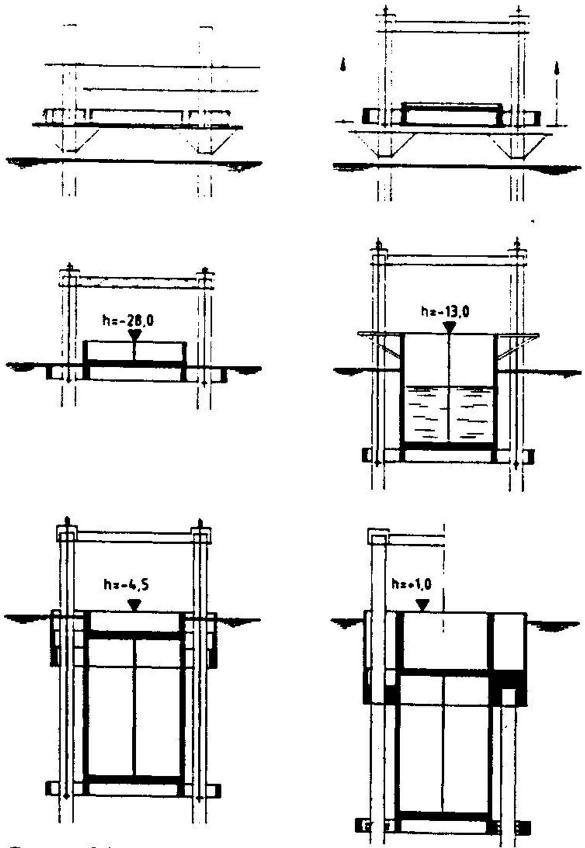


Fig. 10: Lowering Stages of Bascule Bridge Pier

4.2 Flaps

The bascule flaps were fabricated in a specially built up yard about 30 km away from the site. The main girders and the intermediate and outer parts of each element were first fabricated; they were later assembled and put together, with the other elements, according to the sequence indicated in Fig. 11.

The finished flaps were rolled onto a 15,000 t pontoon and floated. They were then rolled off to their final position, where they were adjusted by means of hydraulic jacks and a hold-down of prestressing bars.

Finally, the scaffolding for the lower part of the counterweight was fixed to the intermediate longitudinal plates and the counterweight cast in several layers and prestressed.

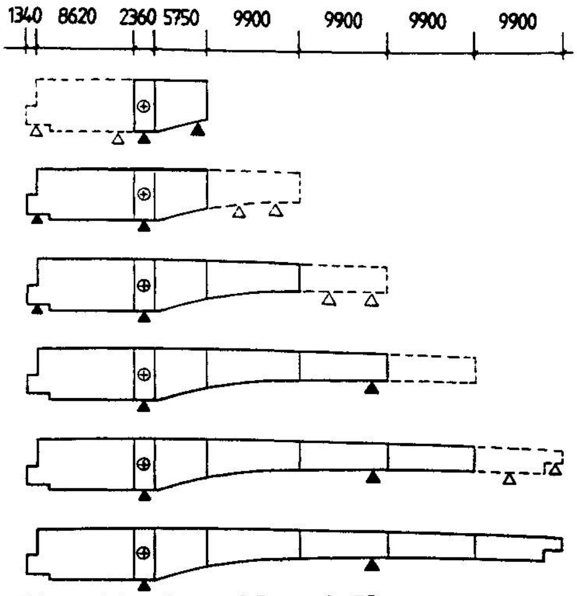


Fig. 11: Assembly of Flaps

5. ACKNOWLEDGEMENT

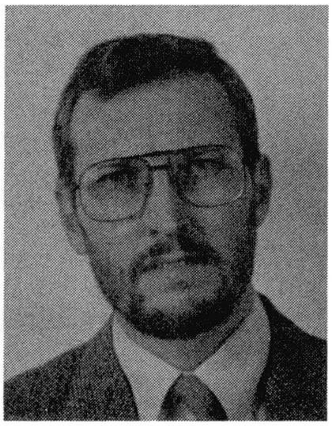
Owner is the 17th Division of the Turkish Highway Administration. The design was done by a joint venture of Leonhardt, Andrä und Partner GmbH, Stuttgart, Germany, and Temel Mühendislik, Istanbul, Turkey. Main contractor is a joint venture of STFA, Istanbul, and Thyssen Engineering GmbH, Essen, Germany. Checking of the design and site supervision are done by Mott Mac Donald, Croydon, UK and Göncer Ayalp, Istanbul.

Myton Bridge, Hull, UK

Le pont Myton à Hull, Royaume-Uni

Myton-Brücke, Hull, England

D.C.C. DAVIS
Technical Director
Acer Freeman Fox
Guildford, UK



On graduating from the University of Leicester, UK, Chris Davis joined Freeman Fox & Partners and was involved in the design of cable supported and steel long span bridges. Chris Davis is now Technical Director responsible for major bridge projects.

SUMMARY

This paper describes aspects of the design and construction of a steel box girder cable stayed swing bridge.

RESUME

La présente communication décrit les caratéristiques du projet et de la construction d'un pont basculant haubané à poutre-caisson en acier.

ZUSAMMENFASSUNG

Dieser Beitrag beschreibt Entwurf und Erstellung einer Dreh-Schrägseilbrücke mit Stahlkastenträger.

1. GENERAL

The Myton Bridge in Hull, UK, crosses the River Hull 250m north of its confluence with the Humber Estuary. The bridge forms part of the Hull South Orbital Road which provides a direct link for heavy traffic travelling between the dock system East of Hull and the trunk road network to the West.

The River Hull serves as a busy and active port used by vessels up to 1500 t gross weight and 60 m in length. As it was not possible to make the bridge high enough to allow all ships to pass underneath it, a movable bridge was required.

Freeman Fox & Partners designed and supervised construction of the bridge on behalf of the client authority, Humberside County Council.

2. DESIGN REQUIREMENTS

2.1 Alignment

It was a planning requirement that the west approach to the bridge should intersect at grade with an existing junction some 200 m from the river. This requirement ruled out the option of crossing the river by a fixed high level bridge. A low level bridge also had considerable aesthetic and environmental advantages which were important because of its location adjacent to Hull's historic Old Town.

The horizontal alignment was determined by land availability and resulted in a crossing of the river at a skew of 22°.

2.2 Shipping Requirements

It was important to minimise the daily requirement for openings of the bridge to permit shipping movements in order to minimise interruptions to road users. The bridge's vertical alignment was, therefore, to be kept as high as possible, consistent with the constraints already discussed, and its construction depth kept to a minimum. This would permit free passage of barges which form the major part of the river's traffic, something which the existing bridges over the river could not do.

Nevertheless, at least one opening of the bridge to shipping was anticipated at every high tide (i.e. two openings per day). A system which could be operated speedily and with minimum interruption to road traffic remained an important requirement.

2.3 Loading Specification and Design Standards

The loading specification adopted conformed to the current British Standards for trunk road bridges. In addition to the basic HA loading, the abnormal 1800 kN HB vehicle was allowed for.

Design of the bridge steelwork was in accordance with the Interim Design Working Rules (IDWR) which were in force at the time, but have since been superseded by BS 5400.

3. DESCRIPTION

3.1 Bridge Superstructure

The structure is a 32 m wide cable stayed bridge carrying dual three lane carriageways and footways. Its spans are 55.6 m and 28.4 m; a navigation clearance between fenders of 30 m is provided. The nose end of the main span is skewed by 22° so that the landing pier can be set parallel to the navigation channel. The back end is radiussed in plan. Longitudinal and transverse

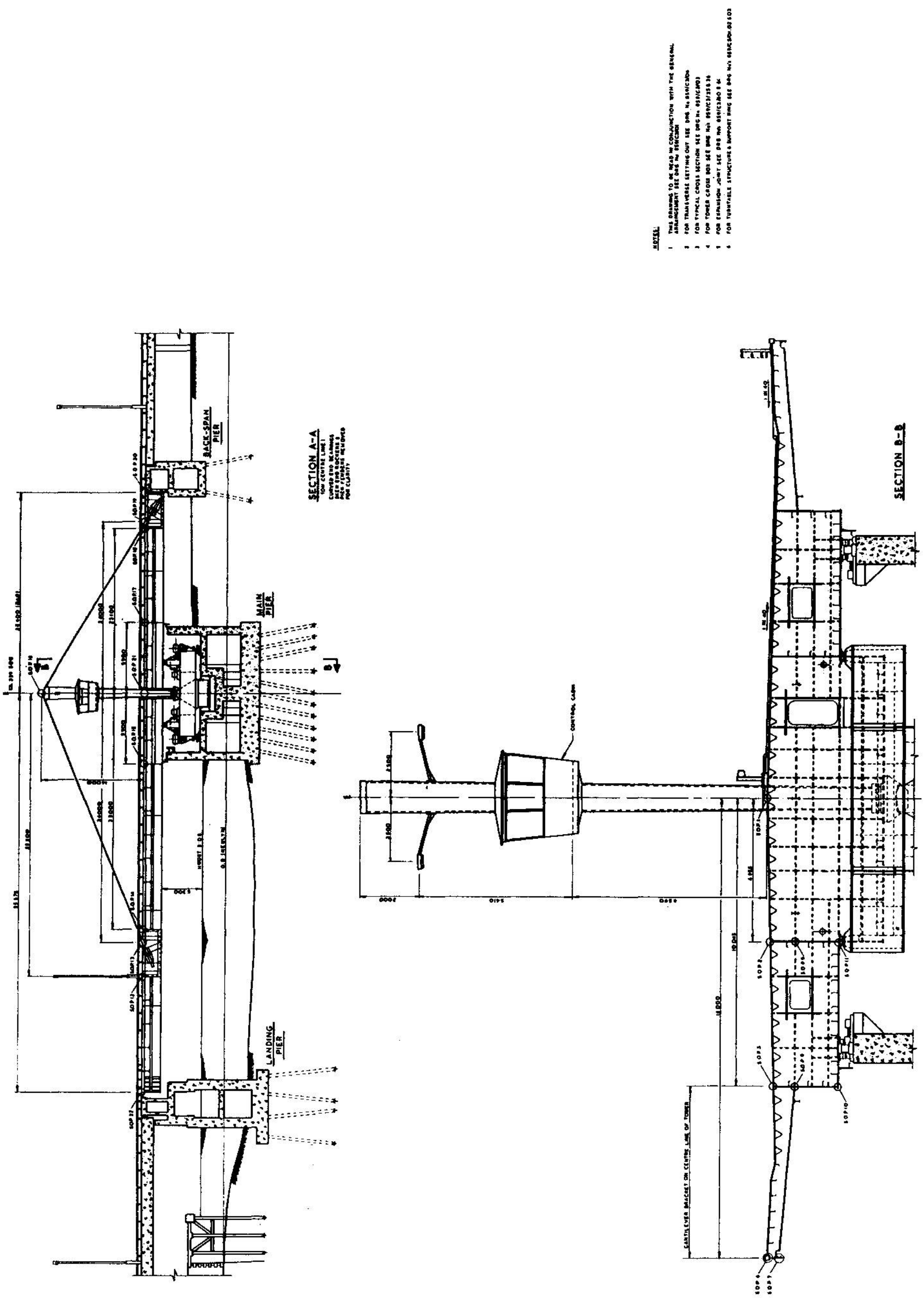


Fig. 1 Longitudinal and Transverse Section

sections are shown in figure 1.

The deck comprises twin steel box girders interconnected by cross girders at 3.3 m centres and surmounted by a steel orthotropic deck plate surfaced with mastic asphalt. Transverse box girders are provided at the main pier and at the anchorages for the cable stays in the main and back span.

The cable stays are formed by six parallel 84 mm diameter spiral strands, each with a minimum breaking load of 6000 kN. These pass over a cast steel saddle which is supported by the hollow steel pylon at the main pier. The socketed stays are anchored within transverse box girders after passing through cast steel splay saddles supported by sliding bearings. The strands are linked by clamps at third points between saddles to present oscillation due to wind excitation. Provision is made in the design for replacement of all stay strands.

Approximately 9000 t of steel is used in the bridge superstructure but, because of the unequal spans, a further 630 t of steel and concrete ballast is required in shorter back span to balance the structure.

3.2 Articulation

At all times, the whole superstructure, including the pylon, rests on a cast steel pivot which is carried by a fabricated steel turntable structure. The turntable is mounted on a 4 m diameter roller bearing slewing ring which is fixed to a drum girder cast into the main pier.

The articulation of the bridge is designed to suit its two operating modes as illustrated in figure 2.

3.2.1 Traffic Position

In the traffic position, two retractable pedestal bearings provide vertical support to the nose of the bridge at the landing pier; two sliding pot bearings support the backspan end. Folding wedge bearings mounted on the main pier wall provide additional support to each longitudinal girder.

A retractable shear key on the bridge's nose engages in a slot in the landing pier to provide transverse fixity.

3.2.2 Slewing Mode

Support to the superstructure in the shipping position, and during slewing, is provided at three points on the turntable structure. Primary support is by the centre pivot. Steel bearings mounted on outrigger girders on the turntable provide stabilising reactions as well as transmitting the slewing torque to the superstructure.

When the bridge is being made ready for opening to shipping, the sequence of operations is as follows. After the folding wedge bearings at the main pier have been disengaged, the nose of the bridge is lifted by hydraulic rams so that the pedestal bearings can be withdrawn. Then, as the rams lower the nose, the whole superstructure pivots so that it comes to rest on the three point support provided by the turntable structure and the back end lifts off and disengages from the backspan pier bearings. Finally, the nose shear key is withdrawn, and the bridge is ready for slewing.

3.3 Operation and Maintenance

The torque to slew the bridge during all movements is provided by twin hydraulic rams connected to the turntable structure and reacting against the main pier wall. In order to prevent over-slewing of the bridge, a buffer unit is mounted on a plinth on the East bank. A similar unit is also provided on the Landing

Pier.

Bridge operations and movements are controlled from the control cabin which is mounted on the pylon and affords the controller a good view of both road and shipping traffic. Sensors and tansducers linked to display panels ensure that the position and mode of all parts of the bridge are known. Interlocks prevent incorrect and possibly damaging sequences of operations being following. CCTV cameras provide additional visual data.

There is provision for future replacement of all mechanical parts of the bridge, including bearings, the slewing ring, actuating hydraulic rams, power packs etc.

3.4 Bridge Substructure

3.4.1 Main Pier

The main pier is an 18.5 m diameter circular hollow reinforced concrete structure founded on precast concrete driven piles. As well as housing the turntable structure and drive mechanism, it contains the principal and standby power supplies, communications equipment and messing facilities. The main pier also gives access to the inside of the pylon, and thence to the control cabin.

3.4.2 Landing Pier

The landing pier is of a similar form to the main pier. There is a direct link to the main pier for communications and other services by a 1 m diameter services tunnel under the river bed.

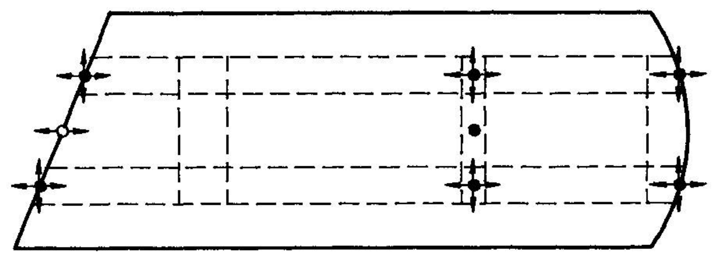
3.4.3 Backspan Pier

Also founded on piles, the backspan pier is radiussed in plan. There is a direct link for services to the main pier.

3.4.4 Fendering

In addition to the fendering mounted on the landing pier, substantial greenhart piled fendering is provided along both sides of the navigation channel. Its function is to guide shipping and to protect the bridge superstructure in the shipping position from damage by shipping collision.

a) Traffic Position



b) Slewing Mode

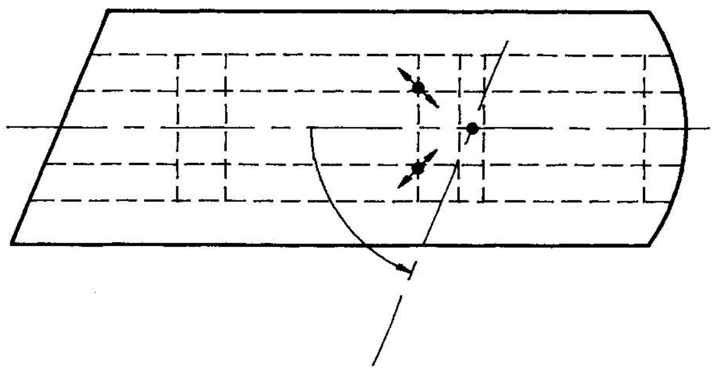


Fig. 2 Articulation

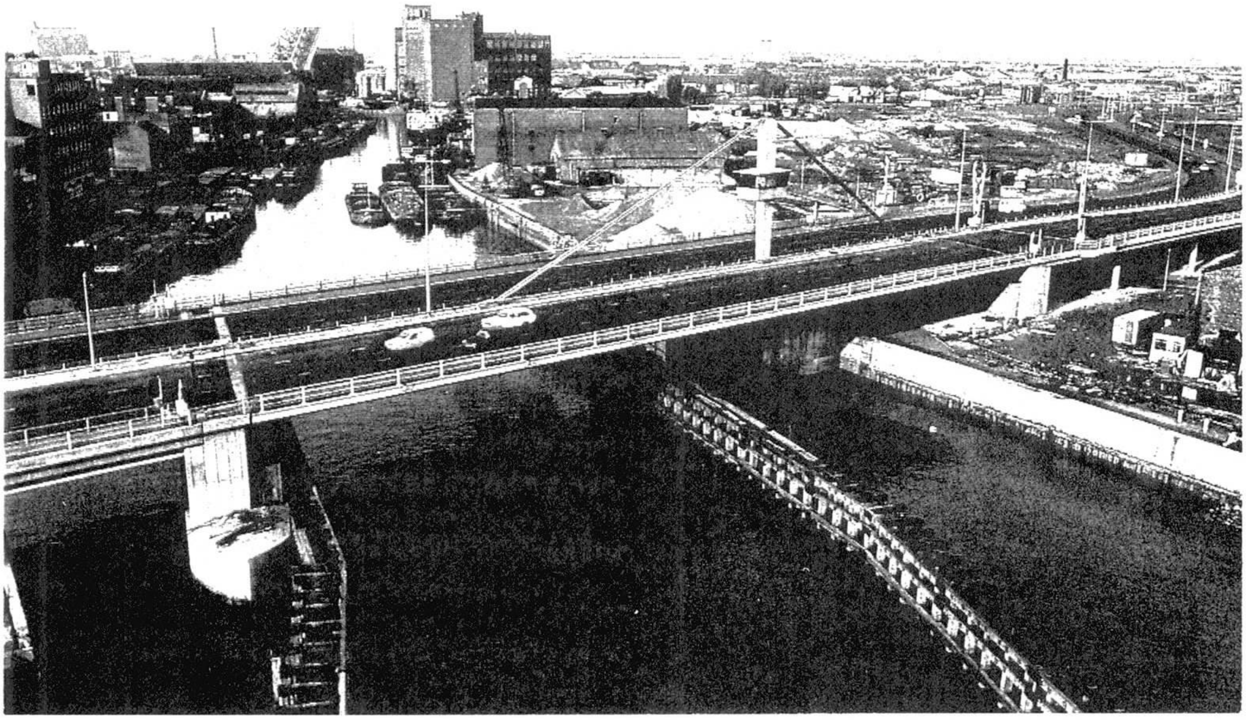


Fig. 3 Myton Bridge, Hull

4. CONSTRUCTION

The steelwork was fabricated and erected the Cleveland Bridge & Engineering Co. Ltd in Darlington, acting as sub-contractor to RDL Contracting Ltd.

The main pier transverse box girder, the turntable box, the pylon and lengths of the longitudinal box girders were assembled at works. All other parts of bridge were assembled at site during erection. The fabricated panels were transported by road to Hartlepool from where they were transported to Hull by ship. The journey from Hull docks to site was completed by road.

The bridge was erected on temporary supports on a line parallel to the shipping position. Site connections between fabricated panels were by welding.

Initially the structure was offset from its final position so that the longitudinal girders were wholly over land and so could be propped by simple trestles. As soon as the whole length of the deck was complete, it was slid into the shipping position, temporary supports continuing to be provided at the nose and back end. The deck was then lowered onto the turntable structure which had already been installed in the main pier. The counterbalancing concrete was then placed in the backspan and the cable stays installed and set to their final length.

After checks of the longitudinal out-of-balance moments by direct measurement with load cells, the whole weight of the structure was transferred to the turntable structure alone. The bridge could then be slewed into the traffic position for final setting of joints and bearings and the temporary supports removed.

The bridge, shown in figure 3, was opened to traffic in December 1980.