Renovation of a Historic Railway Lift Bridge

Jurgen Voermans  
*Royal HaskoningDHV, Rotterdam, The Netherlands*

Jaco Reusink  
*Engineering Dept. Municipality of Rotterdam, Rotterdam, The Netherlands*

Contact: jurgen.voermans@rhdhv.com

Abstract

This paper describes the history, the design, the structural assessment and the major challenges of the extensive renovation project of a historic railway lift bridge in Rotterdam, The Netherlands. The bridge was completed in 1927 and was the first one of its kind in Western Europe. It is considered as a living example of early 20th century bridge engineering art.

Keywords: renovation, historic bridge, vertical lift bridge, structural assessment.

1 Introduction

The Koningshaven bridge is a vertical lift type movable bridge. The bridge was completed in 1927 and was replaced by a tunnel in 1993. The bridge was designated as a National Monument in 2000 and to preserve the bridge for future generations an extensive renovation project is in preparation.

This paper describes the history, the design, the structural assessment and the major challenges of this extensive renovation project.

2 Brief History

2.1 Swing Bridge

The construction of the Koningshaven bridge was part of a major project extending the connection of the Amsterdam-Rotterdam railway line to the Moerdijk-Antwerpen railway line. This part of the project involved the accomplishment of a double track railway through the densely populated Rotterdam inner city and the crossing of the Nieuwe Maas river. The Koningshaven bridge was completed in 1877 as the southern part of the river and canal crossing between the northern shore of the Nieuwe Maas river through the Noordereiland and the southern shore of the Koningshaven. The Koningshaven bridge consisted of a steel arch with a span of 80 m on both sides of a symmetrical swing bridge with a total length of 54.5 m. Openings of 20 m allowed vessels to pass on either side of the center pivot pier. As time progressed the swing bridge no longer satisfied the requirements of the busy navigation and railway traffic. The width of the bridge openings was too narrow and the bridge opened frequently. Both navigation and railway traffic were seriously obstructed. Several collisions occurred as a result. The collision on 10 May, 1978 with the German steamship “Kandenfels” was decisive in the discussion to replace the swing bridge by a vertical lift bridge.
2.2 Vertical Lift Bridge

In 1927 the swing bridge was replaced by the current vertical lift bridge designed by ir. P. Joosting (1867-1942), chief engineer of the Bridge Construction Department of the Dutch Railways. The vertical lift bridge consists of two towers with a movable bridge span in between which is moved up and down by means of ropes.

On two occasions the bridge was heavily damaged: the first time by acts of war during World War II; the second time by the vessel Nedloyd Bahrein which collided with the bridge deck of the open bridge on May 10, 1978. Both times the bridge was reinstated.

2.3 Tunnel

With the construction of the Willemsspoortunnel, a 2796 m long 4 track railway tunnel which enables railway traffic to pass beneath the Maas river and a large part of the city, the railway bridges spanning the Maas river became superfluous. The tunnel was opened on September 15, 1993.

After fierce protests against the proposed demolition, the Koningshaven bridge was designated as a National Monument. The municipality of Rotterdam obtained the ownership of the bridge from Dutch Railways. At that time the bridge already suffered from decade’s lack of proper maintenance.

2.4 Today

Today the bridge is highly admired as a memory to the industrial era and the glorious past of the city of Rotterdam. The bridge is regarded by many people as an icon. In recent years there have been several initiatives to use the bridge as part of a tourist attraction. The most recent idea by Hef Experience Foundation consists of accommodating a restaurant, several walking and climbing trails and a museum on the bridge. The lift span is intended to be used as an open air elevator for a maximum of 200 people to enjoy a panoramic view of the skyline of the city.

There are both supporters and opponents among the citizens of Rotterdam for these initiatives. Supporters want to give a second life to the bridge by giving it a new function as explained above. Opponents don’t want to turn the bridge into a tourist attraction believing that the bridge is retired and should be respected only by looking at it. The historical bridge is a monument and hence the object is protected by cultural property law. Modifications to the bridge that are in violation to the monumental value are not permitted. This makes it nearly impossible to implement any new function on the bridge.

3 Historical Design

3.1 Superstructure

The vertical lift bridge comprises of a through truss-type lift span in 12 equal panels with a span of 53.5 m. The distance between the central planes of the main trusses is 8.8 m. At each end of the lift span is a steel tower about 60 m high from
the base to the top. The vertical clearance at mean high water level provided under the soffit of the lift span is 45 m. By the order of the Government, the towers were designed in a way that the vertical clearance could be increased by 15 m by expanding them. This would give ships with tall masts the ability to sail up the river in the future. Each tower consists of two vertical front legs and two inclined rear legs. The distance between front and rear legs is 15 m at the base of the tower. In the transverse direction the distance is 12.8 m. The legs are braced together on all four faces of the tower except the front and rear face of the first compartment to allow the passage of railway traffic. In the transverse direction this compartment is designed as a portal frame. Both the towers and the lift span consist of built-up steel members made of plates, angles and channels riveted together.

3.2 Substructure

It was decided not to place the towers on top of the fixed spans but to separate both structures. The 50 year old existing cast iron quality was considered not appropriate. The tower supports were located alongside the existing bridge.

Due to the 77.44 degree angle between bridge axis and canal not enough space was available on the existing piers for supporting the front legs. For this purpose the existing piers were widened and new cylindrical piers were built to support the inclined rear legs. To relieve the existing piers it was decided to move the supports of the fixed spans from the existing piers to the new piers. The fixed spans were then on cantilevering and consequently the structure of the fixed spans was adjusted to the changed load distribution.

The soil pressure under the existing piers, which were founded on a sheet-pile caisson, would only slightly increase in the new situation. Each new pier is founded on two open-well caissons with a diameter of 8 m and a height of 10 m and is connected to the existing pier by deep reinforced concrete beams. These beams were partly prefabricated which was advanced at that time.

3.3 Drive System

Initially Joosting wanted to design the drive system according to the principle used in a Waddell type span drive vertical lift bridge. This principle was patented in 1909 by Waddell and Harrington. The drive machinery is placed on the lift span. The advantage of this system is that it is free of skew problems and hence no additional device is needed in order to maintain the two ends of the span at the same elevation during operating. However, Joosting choose to locate the drive system in the second compartment of the southern tower which made this bridge the first vertical lift bridge in the world with an eccentric drive system. Due to the full balancing, the bridge consists of both upwards and downwards motor pull. The motivation for this was the fact that the drive system, including the operator’s house and machinery room, would weigh over 100 Tons. This would cause the 600 Ton structure of the span to become significantly heavier in order to bear this extra load. Further, the weight of the counterweights had to be increased to counterbalance this extra load. Consequently each tower and foundation would be substantially loaded by the extra weight of the operator’s house and machinery room and the extra weight of the span. To maintain the two ends of the lift span at the same elevation additional ropes were used. To convince sceptics of his drive system Joosting built a scale model of the bridge. He demonstrated that his principle of the eccentric drive system worked even when the lift span would be eccentrically loaded.

Figure 3: Lever-type equalizer. [4]
3.4 Counterweight Ropes

The lift span is suspended by 48 counterweight ropes with a diameter of 40 mm, 12 ropes at each corner. The ropes pass over eight cast iron sheaves at the top of the towers and continue down from the other side of the sheaves to the counterweights. The sheaves have a diameter of 3.60 m and contain 6 sheave grooves. Lever-type equalizers are used at the connections to the lift span to ensure equal loading of the ropes.

The counterweight sheave shafts are supported by spherical roller bearings.

*Figure 4: Cast iron sheave supported by a spherical roller bearing.* [4]

The lift span is balanced by two counterweights that consist of a steel frame cast into concrete. In closed position the dead weight of the lift span is not transferred to the live load shoes but to the vertical front legs of the towers. When the bridge is lifted halfway, it is exactly balanced. The balance changes linearly as the counterweight ropes move over their sheaves as the bridge opens and closes. The bridge is not equipped with auxiliary counterweight devices so the bridge drive power has to overcome the imbalance due to the position of the counterweight ropes as well as friction and bend of the ropes. The advantage of this is that the stability in the closed and open position is enhanced by the counterweight ropes.

3.5 Operating Ropes

To pull the span up and down, eight operating ropes with a diameter of 26 mm are used. The ends of these ropes are attached to geared operating drums in the machinery room. The four uphaul ropes extend from the drums upwards and run around intermediate sheaves to sheaves at the top of the southern tower and then to the lift span. At both ends of the lift span they make 90° turns around deflector sheaves upwards to the top of the northern tower where they are attached. The four downhaul ropes are wound in the opposite direction on the drums and run around intermediate sheaves at the base of the southern tower to the lift span. At both ends of the lift span they make 90° turns around deflector sheaves downwards to the base of the northern tower where they are attached.

*Figure 5: Course of counterweight and operating ropes. a. Downhaul rope; b. Uphaul rope; c. Counterweight rope.* [4]

3.6 Levelling ropes

Levelling ropes are applied in order to maintain the two ends of the span at the same elevation during operation to prevent unsynchronized movement. These ropes consist of two pairs of ropes with a diameter of 26 mm with a Z-shaped course. The length of these ropes does not change during raising and lowering of the span. Two ropes run from the base of the southern tower around deflector sheaves at both ends of the lift span to the top of the northern tower; the other two ropes run from the base of the northern tower to the top of the southern tower.
3.7 Machinery

In the machinery room one direct current motor of 200 HP drives through a reduction gearing and a shaft two pinions. These mesh with the ring gears of both operating drums. At both cast in situ drums with a diameter of 2 m brakes are also attached. The machinery can move the lift span with a speed of 0.9 m/s.

4 Structural Assessment

The structural assessment focused especially on the towers that support the lift span. The load carrying capacity of the approach spans and the lift span was not considered critical because of the absence of railway traffic since 1993. The towers were assessed in the situation during normal operation as well as the renovation situation.

4.1 Situation During Normal Use

In the situation during normal use the bridge is by default open for navigation traffic. This is the governing situation for the structure of the towers and their supports since the lever arm of the wind load on the lift span is largest when the movable deck part is raised.

The response of the structure was determined by a 3D FEM beam model. The structure was modelled as a full rigid jointed frame. The rocker pin bearings supporting the front legs can only transfer compression forces and only permit rotation along the transverse axis. Bending moments at the base of the front legs about the bridge axis can be transferred depending on the magnitude of the axial compression force. To account for this local nonlinear behaviour the bearings were modelled by a pressure only line support over the width of the bearing. The bearings supporting the rear legs were modelled as fixed supports.
structures in case of reconstruction and disapproval. A reliability level of $\beta = 3.3$ for renovation level was used. Further reduction to rejection level was not applied. The following limit states were verified:

- loss of equilibrium
- resistance of the structure
- instability of the structure

The wind load was determined according to NEN-EN 1991-1-4 which resulted in significant increase in wind loads. At 65 m height the wind loads increased from 1.5 kN/m² to 3.0 kN/m² including shape factors. This aspect turned out to be of great importance for the assessment.

Based on a linear buckling analysis the structure was classified as a sway frame. Second order effects and imperfections were accounted for partially by the global analysis and partially through individual stability checks of members according to NEN-EN 1993-1-1 Art. 6.3. In the global analysis the internal forces and moments were determined by performing a geometrical non-linear analysis taking into account the global second order effects and global imperfections. In the individual stability checks of members the second order effects in individual members and individual member imperfections were taken into account based on a buckling length equal to the system length.

The governing force effect originated from wind transverse to the bridge axis and resulted in bending moments in the legs which were too high and uplift occurred at one side of both line supports. However, equilibrium of the structure was not lost.

A second analysis was performed where the lift span was assumed to be lowered halfway. This led to a significant reduction of the bending moments at the base of the front legs. The members satisfied the requirements regarding strength and stability. However, some connections could not carry the acting forces. In the original design the connections between members were idealized as pinned connections. Only normal forces were transmitted through these connections. In the frame analysis the connections are rigid and hence also secondary forces occurred which were not accounted for in the original design. This resulted in local exceedance of the actual structural strength. The connections can practically be strengthened by replacing the rivets with prestressed injection bolts or by full welding since the combined action of rivets and welds for force transfer is considered not acceptable. Any strengthening will seriously affect the visual appearance and requires approval by the independent Monument Commission.

To comply with the analysis it is prescribed that at an expected wind speed $\geq 85.7$ km/h (including gust) the lift span needs to be lowered halfway to relieve the towers. Therefore the structural reliability of the structure is dependent on the operational reliability of the drive system.

### 4.2 Situation During Renovation

During renovation the old lead based paints will be fully removed by safe means and replaced by a new corrosion protection system.

To prevent contamination of the environment due to lead paint removal and to apply the new corrosion protection system the steel structure has to be fully sealed and conditioned. A scaffolding will be erected that will be wrapped in plastic foil.

Because lowering the lift span and consequently raising the counterweights in case of high wind speeds will conflict with the scaffolding it has been decided to remove the lift span during renovation. The counterweights will be fastened above in the towers by a steel frame. The advantages are the following:

- Renovation of the lift span in a conditioned environment enhancing the quality of the conservation;
- Enhancing safe working (not at height);
- Reduction of wind load effects on the towers;
- No obstruction to navigation traffic by the lift span that needs to be lowered halfway at high wind speeds.

In comparison with the situation during normal use the area exposed to wind was increased, and hence the wind load on the towers, since the scaffolding is horizontally supported by the
towers. Although the lift span was absent, the total wind load on the towers increased significantly. The results showed that the situation during normal use is governing.

Due to the absence of the dead load of the lift span that works favorably for the fixity of the towers at their supports, at one side of both line supports uplifts occurred in the SLS. This behavior is undesirable during renovation works. Therefore it is not permitted to work on the scaffolding at a wind speed \( \geq 9 \) Beaufort.

Figure 9: Removing of the lift span (photo by M.A.E. Walravens).

5 Renovation of the Steel Structure

In 2016 the bridge will be subject to extensive renovation. The purpose of the renovation is a technical and durability lifetime extension of 15 years at least. No change of future functional use of the monument is taken into consideration due to prescribed severe cost limitations for the renovation. During renovation only minimal disruption of navigation traffic is accepted.

The scope of work is determined by technical inspections, the structural assessment described previously and a risk assessment (technical and personal safety) which is required for movable bridges in the Netherlands.

5.1 New Corrosion Protection System

Inspection results show only local and minor corrosion damage of the bridge although the bridge suffered from 30 years of lack of maintenance, shown in Figure 10 and Figure 11.

Figure 10: Typical corrosion damage at rivetheads.

Up until 20 years ago lead based corrosion protection systems were commonly applied. This coating system on the bridge is far beyond its technical lifetime but is still standing strong and has therefore in practice a lifetime of well over 30 years. However, due to cracking of the thick coating layers and locally poor paint quality, it was decided to replace all paint works.

Blasting to SA2.5 is required and immediate painting after blasting and proper surface roughness conditions for the required adhesive bonding based on ISO12944 environmental class MS/Im. The system comprises of a surface tolerant epoxy (2*160 microns) and polyurethane top layer (60 microns). In this specific situation no zinc based primer will be applied (adhesive problems) and the total thickness will be limited to 350 microns (prevention of cracking of the coating).

Figure 11: Typical local corrosion damage at structural members.
A point of discussion is whether sharp profile edges should be rounded to improve bonding quality. Because of the poor accessibility of the edges this requirement is generally only applied for new structures.

5.2 Standardised Strengthening Scenarios

From other projects it is known that during blasting unwanted surprises relating to unexpected material reduction might be revealed. In order to prevent subsequent project delay a selected number of standardised strengthening scenarios have been pre-designed varying from exchanging rivets, replacing gusset plates to replacing full members for which temporary external support members have to be installed. In relation to material reduction a 4% reduction of nett cross section capacity of a full flange, web or gusset plate is accepted without additional structural checking. In addition all UC values of joints and members are available.

![Figure 12: Worn counterweight sheave](image)

5.3 Renovation of Machinery

All parts of the bridge driving gear are in reasonably good shape. At the moment only the ropes will be replaced. However after removing the ropes the counterweight sheaves showed significant wear. Depending on the outcome of the calculated remaining life time these sheaves can be renewed, renovated or just kept in place.

6 Conclusions

The following conclusions can be drawn:

- Extending the life time of historical movable bridges that are designated as monuments is difficult since the structural modifications due to the technical and personal safety requirements (ex. safety stairs or elevator) are often contradictory to the protected monumental appearance of the bridge;
- The wind load according to NEN-EN 1991-1-4 results in a significant increase compared to the original design wind load;
- The driving gear only requires minor worn based replacements such as cables and sheaves;
- Special attention and non-standard procedures are required for the sensitive high quality application of a full new corrosion protection system on historic riveted structures. Some of the locations are difficult to reach for quality blasting and painting and steel surface can be rough by corrosion;
- Elaborate technical inspections and recalculation proved essential in describing the necessary renovation scope.

7 References