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THE RELOCATION OF A HERITAGE BRIDGE

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SUMMARY

This paper describes the history of a heritage bridge in Amsterdam (The Netherlands) and the efforts made for the relocation of this bridge. Investigations were made to determine the structural integrity of the original elements and structural assessments were done to find the remaining capacity for future use. The Ultimate Limit State for the original elements was investigated. Lateral stability was checked and based on the historical use and the required future use the fatigue loads were calculated for the different cross sections and for critical connections. These calculations showed that a required residual service life of 30 years after relocation was technically possible for this bridge. Some pros and cons for the re-use of this bridge are also discussed.

Keywords: *Movable bridge, Heritage bridge, Relocation, Bridge Engineering, Structural Engineering Construction History, Conservation of structures.*

1. INTRODUCTION

In 2000 a bridge in Amsterdam, originated from 1930, was dismantled (and placed into 'storage') because it was impossible to rearrange the deck layout in order to integrate a light rail in the existing deck structure.
The bridge is one of the members of a family of bridges in Amsterdam all designed by the same engineer, Ir. W.A. de Graaf (1880-1970) and the same architect, mr. P.L. Kramer (1881-1961). Reference to these bridges is made as the "Kramer" bridges.

1.1. Location

The bridge crossed over one of the waterways, the 'Oostertoegang'[1] between the canals of Amsterdam and the "het IJ", the open waterfront behind the main train station of Amsterdam. 'Oostertoegang' also was the original name of this bridge.

1.2. History

The bridge has been in service from the year 1930 until 1973 as part of one of the important connections between the city centre of Amsterdam and the motorway around the city. See Fig. 1. From 1973 till 2000 the bridge was part of a pedestrian and bicycle route; no motorized traffic was allowed on the bridge. See Fig. 2 The lifting mechanism has been out of commission from the period that the bridge was no longer in use as part of the main road infrastructure. Only the lifting cables were removed. The lifting towers, counter weights and other mechanical parts were left in place. In 2000 the bridge was put out of commission and put

[1] The translation of Oostertoegang is East entrance.

in open air storage in the harbour district with the purpose to re-locate the bridge to a location unknown at that moment.
The bridge was named as a part of the heritage of the city of Amsterdam and therefor it was not allowed to demolish this bridge. In 2000 the bridge was hoisted from its supports and laid down for some use in the future. Also, all authentic parts of both abutments, in the form of the swing gates, lifting towers, handrails, natural stone brickwork and bridge operator's house were taken apart and put into storage. Without any protection against the elements the bridge remained in storage until 2018. In 2004 it was decided to relocate the bridge.

Fig. 1. *Bridge at its original location*

1.3. Investigations on the opportunities for the bridge

As part of the project to relocate the bride an investigation was performed to determine the steel grades used in the primary and cross girders and the rivets. The fatigue damage of the bridge was investigated based on the known historic use, the elapsed time in storage and the desired use in the coming years, all in relation with the way the bridge was put together.
The re-use of the original wooden deck was not an option, the deck had to be replaced. It is replaced by a modern steel orthotropic deck. The old and new steel elements will be connected by bolt connections, although the original steel is weldable. Because it is a heritage structure a requirement was that all additions should be easily reversible.

1.4. Present state

In 2019 the parts of this bridge are in the shop for refitting and conservation. The new steel deck has been built. One of the two new abutments is poured, but not yet cladded with natural stone and masonry. All new piles for the foundations are placed.

1.5. Future

In the year 2004 it was decided to incorporate the bridge in a new road for local traffic as part of the redevelopment of the Westerly Harbour District. The execution of these plans was postponed, what also delayed the re-location of the bridge. The bridge will be placed on a set of new abutments in concrete with a steel pile foundation. The abutments will be cladded with a part of the original granite stone work. The actual re-locating is planned for autumn 2019. At the new location the bridge will become a part of the entrance of a dwelling area and to be used by motorized traffic again with a required remaining service life of 30 years. The lifting mechanism however will not be re-used. The four lifting towers will be re-installed with dummy cables. As many parts as possible of the old bridge should be re used. If it was in any way possible to use the original girders as load bearing elements this should be done.

Fig. 2. *Bridge as pedestrian and bicycle bridge in 2000.*

1.6. Overall dimensions

The original bridge can be described as a lifting bridge. The overall length is 27.9m and overall width is 12.8m. The main span of the bridge is 27.5m, the centre to centre distance between the main girders is 12.4m Fig. 3

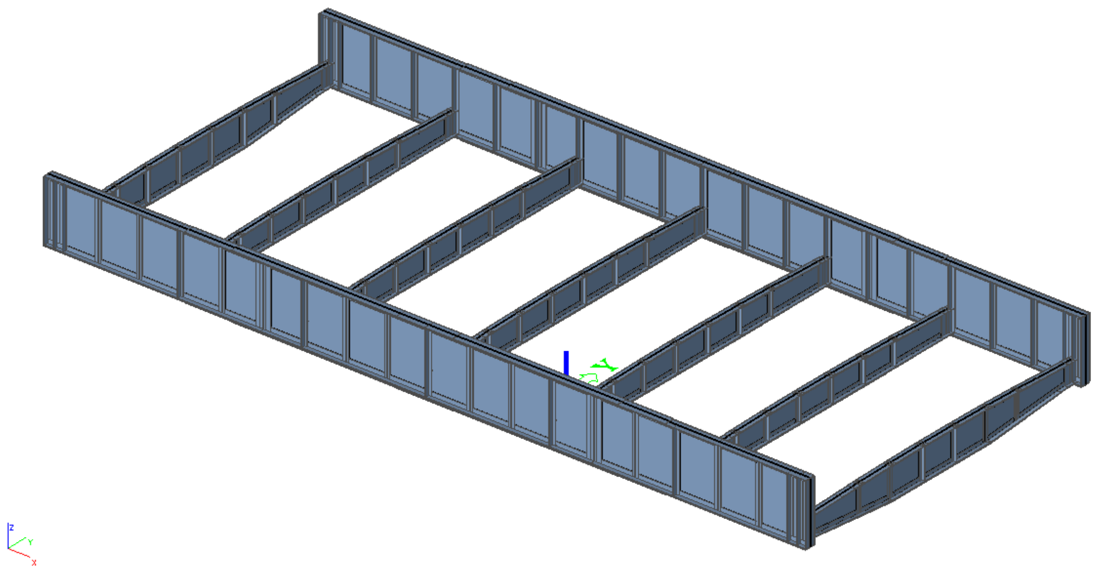

Fig. 3. *3D image of the main and cross girders.*

The main girders are made of steel plates and angle irons riveted together. The height is 2.346m and a width of 0.400m. Web thickness 16mm, the flanges built up to 3 layers at midspan, the thickness of each layer is 16mm.
The cross girders are also made of steel plates and angle irons riveted together. The maximum height is 1.135m and a width of 0.300m. Both flanges built up to 3 steel plates at midspan, the thickness of each plate is 10mm, the web has a thickness of 12mm. The wooden deck rests on a system of steel elements. These elements are H shaped beneath the central road deck and U shaped under the side decks.
There are three types of cross girders with different heights. 3 times with a height of 950mm, 2 times with a height (H) of 900mm (Fig. 4) and at both supports with a height of 1135mm.

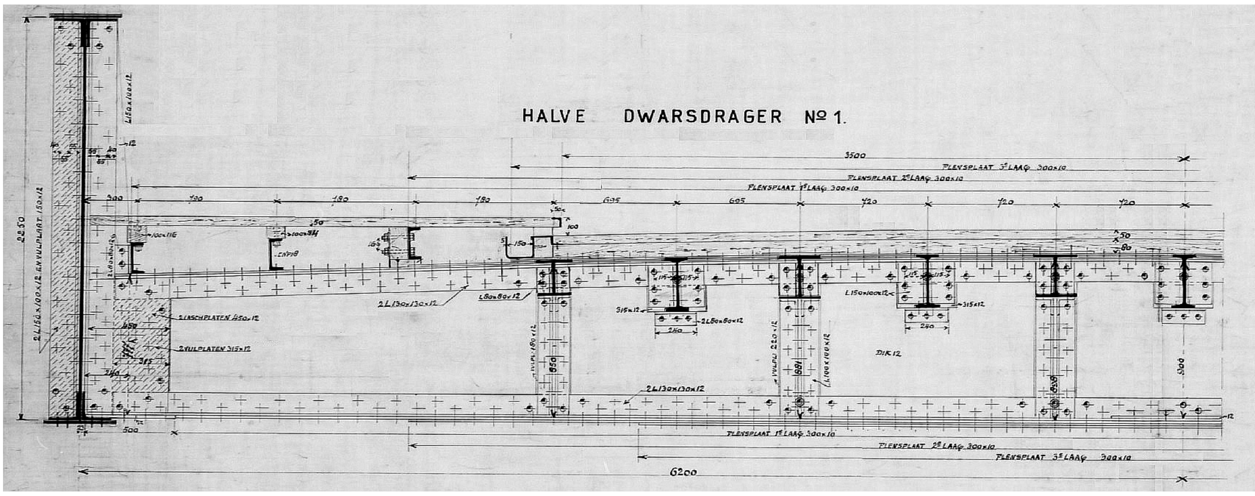

Fig. 4. *Fraction of an original drawing of the cross girders with H=900mm.*

(With Fig. 4: 'Halve dwarsdrager No 1.'translates as 'Half cross girder #1'.)

The horizontal stiffness is maintained by a horizontal lattice system connecting the bottom flanges (not shown in Fig. 3).
The wooden deck planks had a thickness of 50mm, the species of the wood is unknown.
Four lifting towers were present, also fabricated from steel plates and angle irons, riveted together.

1.7. Steel grades

The original steel grades for the plates and the rivets were unknown. A survey was commissioned to gather all information regarding yield strength, ultimate strength, chemical composition and Charpy V-notch values. This resulted in a steel grade similar to S235 with a ductility between JR and 0. All rivets have a steel grade comparable to S275. Probably the original grade of the steel was 1.B with a minimum tensile strength of 36 kg/mm^2 for the plates and 1.C for the rivets with a minimum tensile strength of 42 kg/mm^2 (according to A.V.IJ. 1911 [4]) Fig. 5

A. V. IJ. — Tabel. — 3e blad.
Eischen waaraan de materialen bij beproeving moeten voldoen. (Hoofdstuk IV, § 18: MATERIAALE

BENAMING van de groep en van de soort.	Aanduiding der soort (kwaliteits-aanwijzer).	Dikte van plaat, staaf, enz. d. in mM.	TREKPROEF. Breekspanning in K.G. per mM². Minimum	Maximum.	Rek over de meetlengte in percenten. Vergel. § 16 onder 7⁰. Minimum.	BUIGPROEVEN. Dikte van de proefstaaf in mM. Vergel. § 16, onder 8⁰.	Koude buigproef. Vergel. § 17, onder 5⁰. Dikte van de kern waarom buiging plaats heeft in mM.	Buighoek in graden Minimum.	Warme buigproef. Vergel. § 17, onder 6⁰. Dikte van de kern waarom buiging plaats heeft in mM.	Buighoek in graden Minimum.	Hardingsbuigproef. Vergel. § 17, onder 7⁰. Dikte van de kern waarom buiging plaats heeft in mM.	Buighoek in graden Minimum.	Bijzondere proeven.	min
III. VLOEIIJZER.	1. A.	≤ 25 > 25	42	50	20	d 25	2 × d 2 × 25	180 180	— —	— —	2 × d 2 × 25	180 180	In strook van 6—10 mM. dikte en 50 mM. breedte in roodwarmen toestand door tapschen doorslag van 80 mM. lengte, die van 20 op 30 mM. taps is, een gat slaan van 30 mM. De strook mag daarbij niet scheuren.	
1. Profiel-, Universaal-, Staaf- en Klinkbouten-ijzer.	1. B.	≤ 25 > 25	36	44	22	d 25	1 × d 25	180 180	— —	— —	1 × d 25	180 180	Proef als 1. A.	
	1. C.	—	42	50	22	—	geen	180	—	—	—	—	Van klinkbouten de koppen uitsmeden bij klinkhitte tot eene dikte van 2 tot 3 mM. Klinkbout waaiervormig plat smeden tot $^1/_5$ van de dikte; geen andere dan kantscheurtjes mogen zich vertoonen.	St ketel
	1. D.	—	34	42	25	—	geen	180	—	—	—	—	Proef als 1. C. Een stuk van de staaf ter lengte van 2 × d in roodwarmen toestand tot $^1/_3$ der oorspronkelijke lengte opduiken, zonder kantscheuren.	St ketel Kl dan brugg

Fig. 5. *Part of the Dutch code A.V.IJ [4] describing the requirements for material properties.*

2. OBJECTIVE OF THE RE-LOCATION

The main objective was to preserve this heritage bridge with as much of the original parts and materials as possible. A report [1] was commissioned to put a cultural historic value to the different elements. In this report a high value was given to most of the elements of the bridge and its abutments, only the deck and the cross girders were less valuable.

2.1. Re-usable elements

From the start it was clear that the wooden planking of the deck and its direct supporting steel beams were not salvageable. The deck had rotted away in several locations, the steel beams were heavily corroded.

The lifting towers, all fitted with special lighting, will be re-used, together with the main and the cross girders. The lifting cables will be re-installed, but only as a showcase. The lifting mechanism will not be installed again but be put on display nearby. The bridge operator's house will be rebuilt in the style of the original one, but it will be enlarged to function as an entrance to a parking garage. Most of the natural stone will be replaced, because the shape of the connecting roads with the side walls differs largely from the original shape. All existing sculptures will be salvaged and integrated in the new abutments. Eyecatchers will be the original swing gates with cut outs in the shape of a bird. Fig. 6

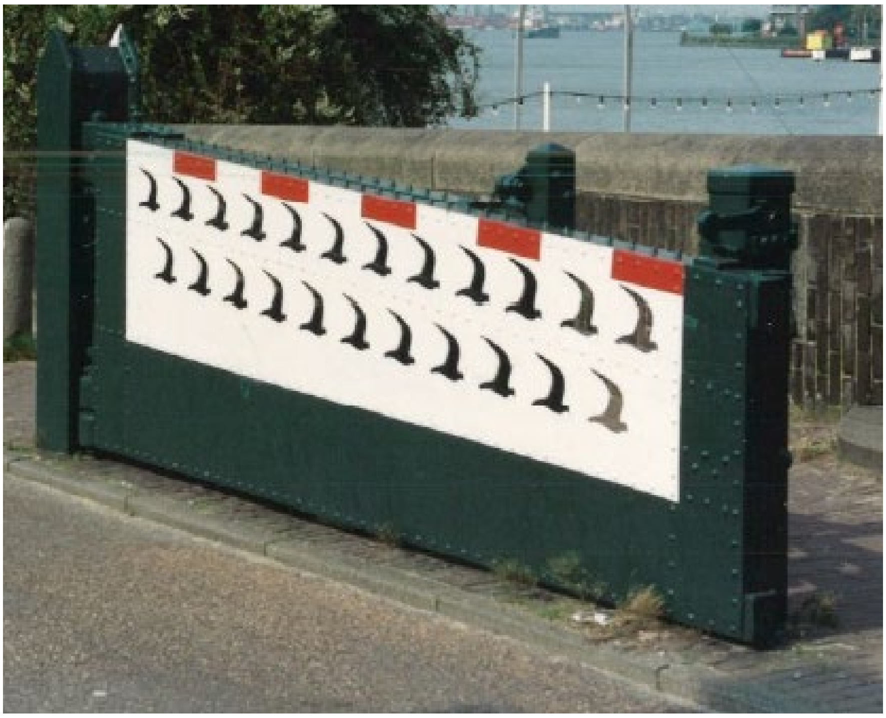

Fig. 6. *The gates with the bird silhouettes.*

As part of the re built of this bridge many rivets will be replaced by pre-tensioned injection bolts. Also (too heavily) corroded parts of the girders will be exchanged with new steel plates. Fig. 7

Fig. 7. *Corroded rivets in bottom flange.*

2.2. New elements

The main (visible) new element will be the deck of this bridge. A modern orthotropic steel deck comprised of a 15mm steel deck plate welded together with troughs (plate thickness of 10mm) will replace the original wooden deck. The new deck is supported by a grid of new steel beams. The new steel beams are bolted on the sides of the old cross girders. For this connection pretensioned injection bolts will be used. The deck layout consists of three parts, a central part for motorized traffic in one lane with a width of 4.8m and two 200mm raised sides of 3.8m for pedestrians. The municipality of Amsterdam requested that during a short period of two years two lanes for motorized traffic should be available. In this temporary situation 2 lanes with a width 3.0m should be available on the bridge. To accommodate this, a part of both sidewalks will be bolted in at a later stage.

3. CALCULATING FOR FUTURE USE

3.1. General

All calculations will follow the Dutch NEN8700 [5] and NEN8701 code. This code, not being part of the Eurocode system must be read as an addition to the Eurocode for existing structures. It is a special code for calculations on the renovation of buildings and road and railway bridges. In this code one can find axle load arrangement from the past and the future and how to take into account the change of traffic intensities over the years. Maybe the most important deviance from the Eurocode for new built structures is the use of reduced partial factors for dead weight and live loads.

The philosophy behind the reduced factors is that the structure has proven itself for several years, was considered as safe for use and will not last the normal 50 or 100 years from the moment of recalculation. This bridge was in use for some sort of traffic from 1930 till 2000, being 70 years. The goal was to find out if this bridge was safe to use for another 30 years. This should bring the total lifetime to 100 years. After this period, it shall be discussed what to do with the bridge. If, at some moment, the bridge is not considered safe anymore it can be replaced by a pre-stressed concrete deck without changing the abutments. The calculations for the foundations were performed with the higher weight of the concrete deck.

3.2. Strength

Calculating for strength was straight forward. The bridge will have a single lane for motorized traffic, together with sidewalks on both sides. Since the sidewalks are placed 200mm above the road level, by code there is no requirement to calculate for heavy traffic on the sidewalks, only for an occasional service vehicle. So, it was only necessary to do the calculation with LM1 according to Eurocode in the centreline or next to the sidewalk. LM1 is defined as 9kN/m^2 uniform load and a twin axle load of 2x300kN. Of course, the traffic loads were combined with temperature and wind loads, all according to the Eurocode.

For two years it should be possible to have two-way traffic with lorries on the bridge. This is the period with the widened deck. In case of a second lorry a reduced axle load of 2x200kN is applied. The highest unity check (M_{Ed}/M_{Rd}) found was 0.54 for bending of the main girder. As part of a connection the unity check for stresses in the rivet of the main girder were determined as 0.82. After two years, due to the deck width of 4.8m only two-way traffic for cars will be possible.
Stresses in the different cross sections were calculated as if there were no rivets. All sectional properties were made up with the nett cross section. This reduced the moment of inertia to an average of 80%.

3.3. Fatigue

For this part of the calculation things got more complicated. The historic use from 1930 to 1973 and the future use from 2018 to 2048 must be considered. The period 1973 till 2000 did not contribute to the fatigue load since it was at that time a pedestrian and bicycle bridge. In the fatigue load-model the heaviest lorry has a total vehicle weight of 770kN spread over 8 axels and a total length of 17.8m. The maximum axel load is 110kN. For each lorry type the influence lines were calculated for the main girder, the secondary girders, the new deck beams and the deck. With these influence lines it became possible to find the fatigue load built up in the past and add to this the foreseen fatigue load. Fig. 8 and Tab. 1 from [3].

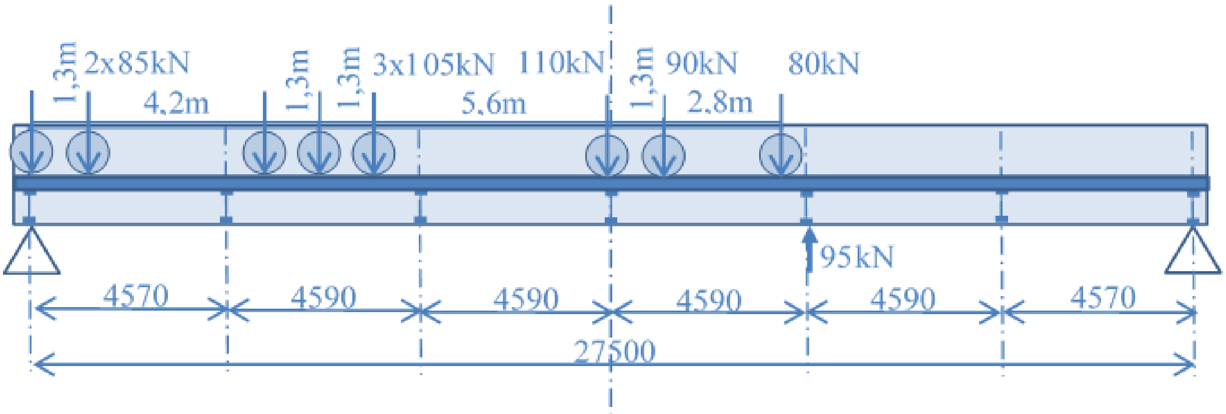

Fig. 8. *Lorrie type 6T12O3A2-heavy (acc. NEN 8701 Table A.3).*

Table 1. *Built up for fatigue from different periods of use*

Part		1930-1973 1 lorry	1930-1973 2 lorries	2018-2020 1 lorry	2018-2020 2 lorries	2020-2048 1 lorry	Total
Cross girder 900 mm	Flanges	0.109	0.187	0.008	0.019	0.008	0.331
	Web	0.000	0.000	0.000	0.000	0.000	0.000
Cross girder near support	Flanges	0.039	0.095	0.003	0.009	0.003	0.149
	Web	0.000	0.000	0.000	0.000	0.000	0.000
Main girder	Flanges	0.001	0.041	0.000	0.007	0.000	0.049
	Web	0.000	0.001	0.040	0.593	0.000	0.634

All parts of the bridge that were considered as a possible weak spot were checked against a reduced allowable stress level based on the fatigue loads for this element and the configuration of the spot itself. To mention some of these spots:

- Mid span of main and cross girder;
- The riveted connection in the web plate;
- The reduced section of the cross girders near the connection to the main girders;
- Main girders near the supports.

3.4. Deformation and vibrations

For deformation the LM1 is governing over other loads, with a combined deflection at midspan of 51.8mm, what is less than 0.002x the span of 27.5m.
The lowest natural frequency is calculated as 4.19Hz, this leads to a vertical acceleration of 0.3m/s^2 under LM4 (pedestrians only). Required is an acceleration less than 0.7ms^2 acc. Annex A2.4.3.2 of NEN-EN1990

3.5. Stability

Only the lower secondary girder (H=900 mm) did not meet the requirements for stability under the load of two lorries of 60T and 40T in opposite directions at the same moment over this girder. It was advised to prohibit the use of the bridge by any two heavy weight lorries at the same time. For other traffic categories two-way traffic will be allowed.

3.6. Overall conclusion

Technically it is possible to re-use this bridge at a new location for a remaining lifespan of 30 years.

4. THE PRO'S AND CONS OF RELOCATING THIS BRIDGE

4.1. The advantages

In this case the most prominent reason for re-locating this bridge, i.e. preserve a monument for the future is fulfilled. A large amount of the steel elements will not be scrapped, and so no new material is required what is good for the sustainability.

4.2. The disadvantages

There is a lot of waste coming from the blasting of the steel parts, old paint with unknown chemical composition had to be taken care of. An unfavourable new deck layout had to be used to fit on the old elements. The bridge will still have a limited remaining lifespan. Cost per sqm is high compared to a total new bridge. Future maintenance will also be costly. Transportation to the new location is difficult and expensive due to the vulnerability of the structure.

4.3. Discussion

In the Netherlands there is a tendency to become a fully circular economy in 2050. The re-location of bridges which are no longer functional at their current location, fits in this. To promote the re-location of old bridges there already exists a website with data of these bridges, called 'Bruggenbank' (https:/www.bruggenbank.nl). The time between getting non-functional and re-location should be kept as short as possible. This reduces the degeneration of the bridge, since responsibility for maintenance during the storage period can become an issue.

5. CONCLUSIONS

For economic reasons we hold the opinion that one should not do a re-location as this one. For sustainability reasons it is questionable, a big gain was not achieved for there still is a lot of waste. Of course, from a historic point of view it is worthwhile to preserve this heritage bridge.

It is recommended to evaluate the condition of the bridge prior to the decision of re-location. This will limit the chance that it turns out to be impossible to re-use the bridge. Before the actual re-location a full survey should be performed and thoroughly documented. For smaller bridges a roofed-in storage is favourable.

6. ACKNOWLEDGEMENTS

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