The design of two large span arch bridges in the port of Rotterdam

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The design of two large span arch bridges in the Port of Rotterdam

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Abstract

The existing movable Caland Bridge in the Port of Rotterdam is a part of the 'Havenspoorlijn' (Port Railway Line). Because of the high intensity of both the shipping and railway traffic the existing bridge has a lack of capacity. Therefore the port of Rotterdam designed a new railway track to bypass this crossing. In this new route (Theemswegtracé) two new arch bridges and a 4km long concrete simply supported beam structure will be realized. This paper will be about the design of the two arch bridges, one with a single span of 172,8m and the other with a continuous beam over 4 supports with a total length of 269,1m.

Keywords: Bridge, railway, steel, steel-concrete, arch, stability, dynamics

1 Project location

The new route of the Port Railway Line bypasses the busy Brittannië-harbor over the lock 'Rozenburgsesluis' and will join the existing railway line with a bridge over the 'Thomassentunnel' in the highway A15. This paper will be about both bridges.

The bridge over the Thomassentunnel is a double-track steel arch supported beam bridge with a main span of 156,1 m and ramps of 56,6 and 52,4 m, respectively. The total length of the bridge is 269,1 m. The center-to-center distance between the main girders is 12,1 m. At the south-eastern ramp this distance fans out up to 13,3 m to accommodate the curve in the track.

The bridge over the Rozenburgsesluis is a double-track steel arch supported beam bridge with a single span of 172,8 m. The total length of the bridge is 176,8 m. The center-to-center distance between the main girders is 18,1 m, so that the arch encloses the curved track.

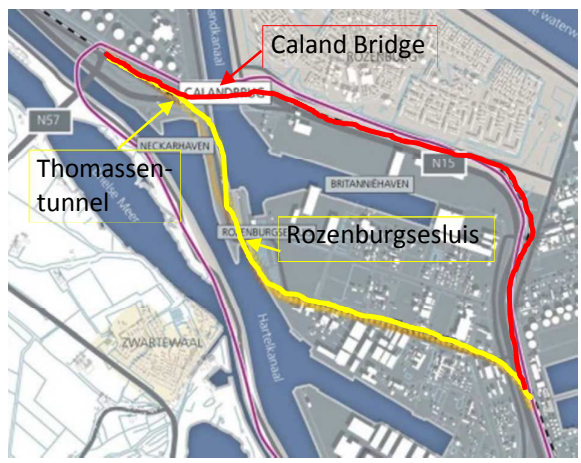

Figure 1: Map of the project area. Red: old route; Yellow: new route (www.portofrotterdam.com)

2 Architectural design

The bridge over the Thomassentunnel has been specifically designed as a steel arch supported bridge. This means a relatively heavy main girder and a slender arch. This type of structure is architecturally more suitable for the surrounding landscape as it emphasizes the length of the bridge and therefore better fits in the scale of the

industrial landscape it is placed in. Structurally this is also an advantage for designing the ramps on either side of the bridge. The large main girder provides the bending stiffness for uneven loading of the main span and is also strong enough for the side spans. In cross-section the high main girders are very effective to reduce sound emission to the surroundings.

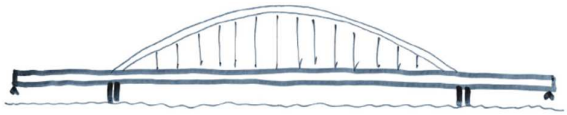

Figure 2: Architectural vision, bridge over the Thomassentunnel

For the bridge over the Rozenburgsesluis the more park like surroundings of the lock demanded a softer shape, so in detailing the arch birth the round shape of the arch itself is more pronounced. Structurally the same system is used as the bridge over the Thomassentunnel.

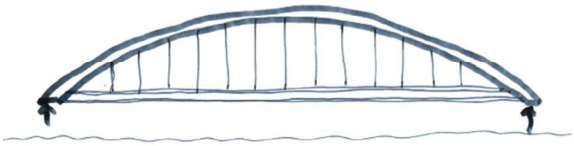

Figure 3: Architectural vision, bridge over the Rozenburgsesluis

Figure 4: Artist impression, bridge over the Rozenburgsesluis

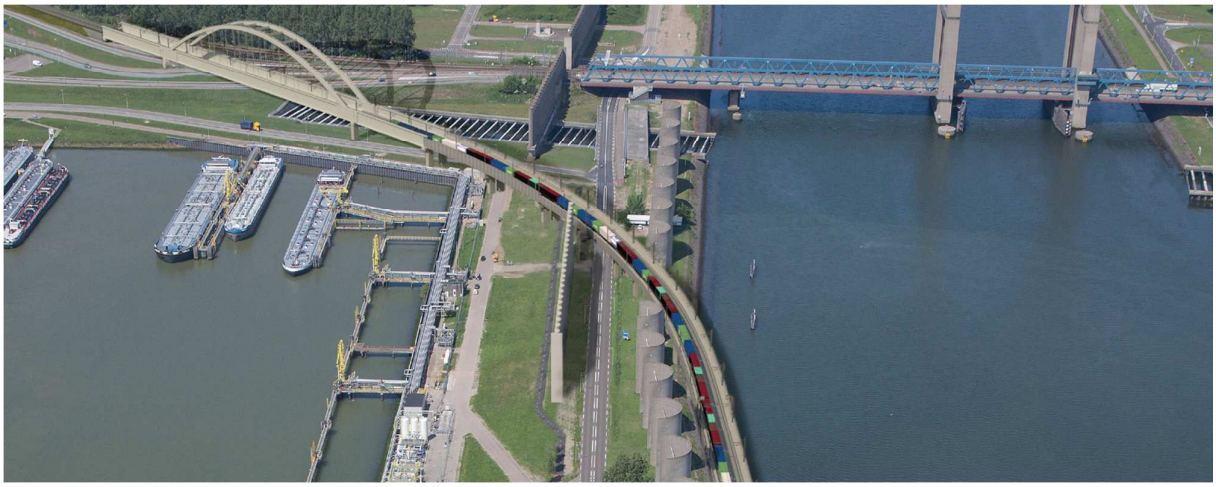

Figure 5: Artist impression, bridge over the Thomassentunnel

Figure 6: Artist impressions

3 Arch stability

In order to check whether the arch is stable under all conditions, the buckling stability was analyzed. Because the check for buckling length via the standard calculation rules is fairly conservative, the finite element model was used to calculate the eigenvalue.

The calculated eigenvalue for the bridge over the Thomassentunnel is $\alpha_{cr} = 5.97$ for the horizontal buckling form of the arch and $\alpha_{cr} = 17.04$ for the vertical buckling of the arch. For the bridge over the Rozenburgsesluis this is $\alpha_{cr} = 5.69$ for the decisive buckling form of the arch. See Figure 7 and Figure 8.

In both bridges the sideways stability of the arch is decisive for the stability of the entire structure. This is due to the fact that for architectural reasons no wind bracing is present between the two arches. This way the sideways behavior is similar to a Vierendeel girder.

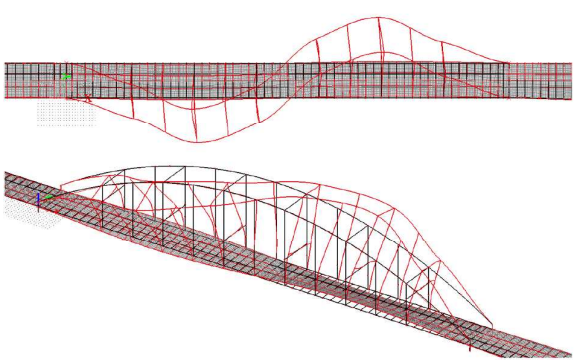

Figure 7: Decisive eigenvalue 5.97 of the Bridge over the Thomassentunnel

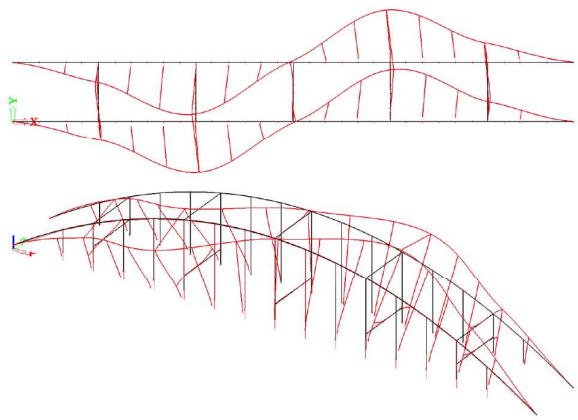

Figure 8: Decisive eigenvalue 5.69 of the Bridge over the Rozenburgsesluis

The eigenvalues were used to determine the critical buckling force and subsequently the buckling length, after which a complete stability evaluation was performed in accordance with the Eurocode. The result of this evaluation is that the stability is guaranteed.

4 Concrete deck

The deck of the arch bridges is made of concrete. To save weight this concrete deck has a very high slenderness with a thickness of only 400mm. The consequence of this high slenderness is large quantity of reinforcement.

The concrete deck is structurally connected to the steel cross girders, utilizing composite behavior. Disadvantage is that in main span direction the concrete deck will be loaded in tension, as we have a tied arch bridge. As a result a quite large amount of reinforcement was needed, which ended up at four layers of Ø32-100 rebar in

longitudinal direction. This is the maximum amount which can be applied in practice.

4.1 Crack width control

Despite the large amount of reinforcement, the calculated crack width does not meet the strict requirements according the Eurocode. This is mainly because of the shrinkage of the concrete, which is impossible to avoid in a calculation according the governing standards. Another big factor is the fact that the deck is placed in the tension zone of a tied arch bridge. Added permanent weight and live load will cause extra tension forces in the concrete deck. To still be able to design such a slender deck, without problems due to crack width, extra measures during the construction phase of the bridge have been implemented.

These measures are:

- Apply an elastic coating (according to OVS, the Dutch Railway Design Guidelines) on the topside, to prevent penetration of chloride and aggressive substances.
- Specification concrete mix
    - Cement type: CEM III/B 42,5N
    - Environment: XC4, XD3, XF4
    - Additional material: gravel – 100% - 4-16mm
- Adding synthetic fibers to control crack width
- Tune exact time of casting and used curing agents with curing process of concrete, so the peak temperature will occur at night.
- Use a curing compound in combination with covering of the curing concrete. This is done to prevent cracks due to plastic shrinkage.
- Monolithic finishing of the concrete.
- Remove concrete formwork only after 28 days.
- Cast concrete only on clouded days combined with a temperature between 0°C and 25°C.
- Isolate the concrete with special isolating foil, correctly placed without any gaps. Remove isolating foil only when the difference in concrete temperature and the upcoming predicted night temperature is less than 15°C.
- If the outside temperature is lower than 4°C, protect the freshly casted concrete until the average compression strength reaches 5 N/mm^2 (according to Dutch code NEN6722).
- No casting of concrete directly against areas with a temperature less than 0°C.

4.2 Connection between concrete and steel

To ensure full composite behavior, the concrete must be connected to the steel cross-beams. This connection is realized using welded shear studs on the top flange of the cross-beams and the web of the main girder. The shear studs are checked for strength, fatigue and serviceability according to the Eurocode. This resulted in a large amount of shear studs in the zones were the forces are transferred between arch birth and start of the deck.

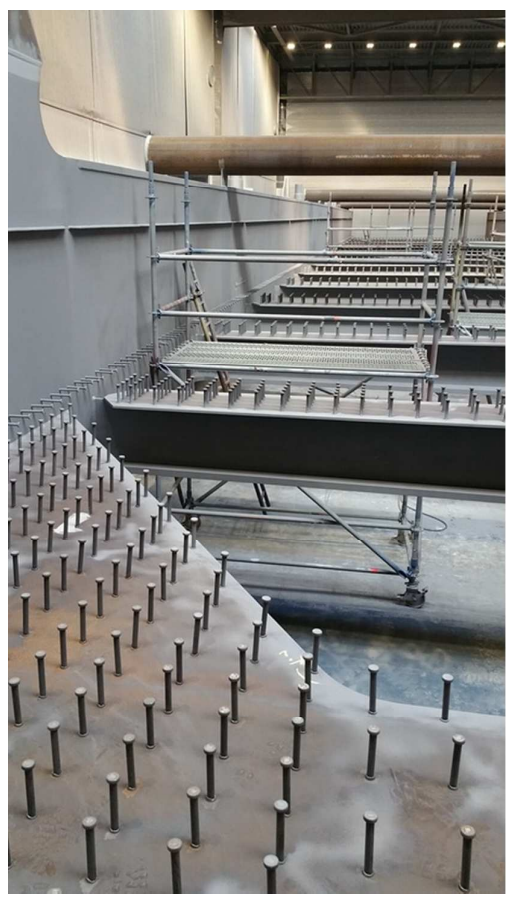

Figure 9: Example of studs placed in such a bridge

4.3 Stiffness of concrete

The actual stiffness of the concrete depends not only on the material, but also differs between the cracked and non-cracked stage. Therefore the forces in the concrete have been calculated using the un-cracked stiffness, while the forces in the steel main girders and arch have been determined using the cracked stiffness.

5 Combined response of structure and track to variable actions

Over a length of 4km the track is placed on a series of concrete bridges, before reaching the first steel bridge. Because of this the combined response of structure and track to variable actions is very important. Since the track crosses several different types of structures with different stiffness's and also is fitted in a curve, it is not possible to calculate this behavior in a simple manner by hand. We added a length of about 500m of track and adjacent structures to the FEM model of the bridge, where the detailed model of the bridge is used and the adjacent concrete spans are modeled with beam elements. A separate beam element introducing the rails itself is connected via dummy beams with the correct stiffness and slip behavior of track in ballast. This is an elaborate version of the model shown in article 6.5.4.4 of EN 1991-2.

Focus areas fort his model where:

- Not placing the vertical load in the rail element itself, since local bending between the rail mounts is not needed in the checks.
- Not placing the centrifugal load on the rail for the same reason.
- Braking and acceleration forces are placed directly on the rail element, to ensure the correct distribution of horizontal force between the spans.
- Stiffness of the rail mounting and ballast in the direction perpendicular to the track.
- Modeling and physically placing the extensional degree of freedom of the supports parallel to the track.

From this analysis we found that at the bridge over the Thomassentunnel the stresses in de rail exceeded the limit at the transition from steel bridge to concrete bridge. Several solutions for this problem have been analyzed:

- Adding extra rails in the middle of the sleepers to take the braking and accelerating forces.
- Using a rail profile with a larger cross-sectional area.
- Using rail mounts with a lower frictional resistance. This will make sure the loads are spread over a larger length of rail.
- Adding an extra rail expansion joint. Note that a standard extension joint cannot be used, since the track is placed in a curve.

In consultation with ProRail (Dutch railway authority) the solution with adding extra rail is preferred. The extra rail will be applied over a length of 90m (45m at both sides of the bridge transition), to get the stresses down to an acceptable level.

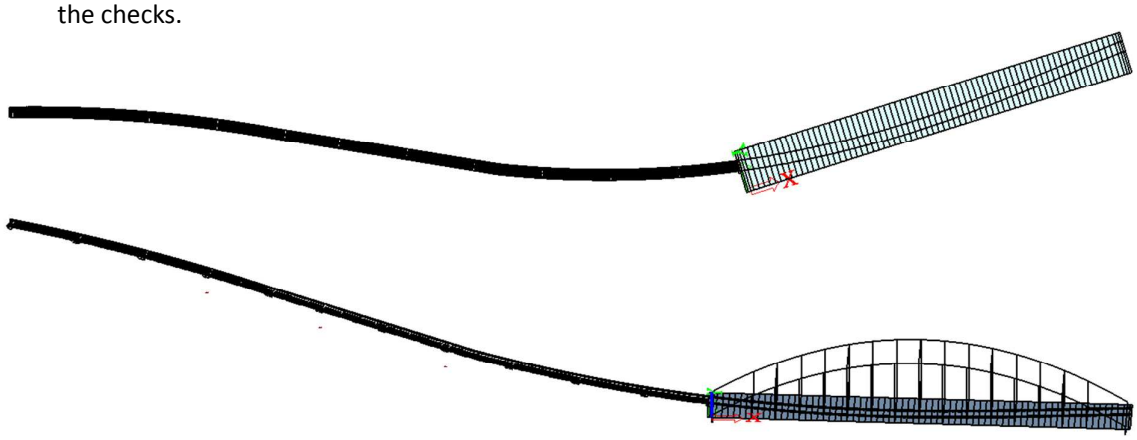

Figure 10: Overview of model (Rozenburgsesluis) for combined response

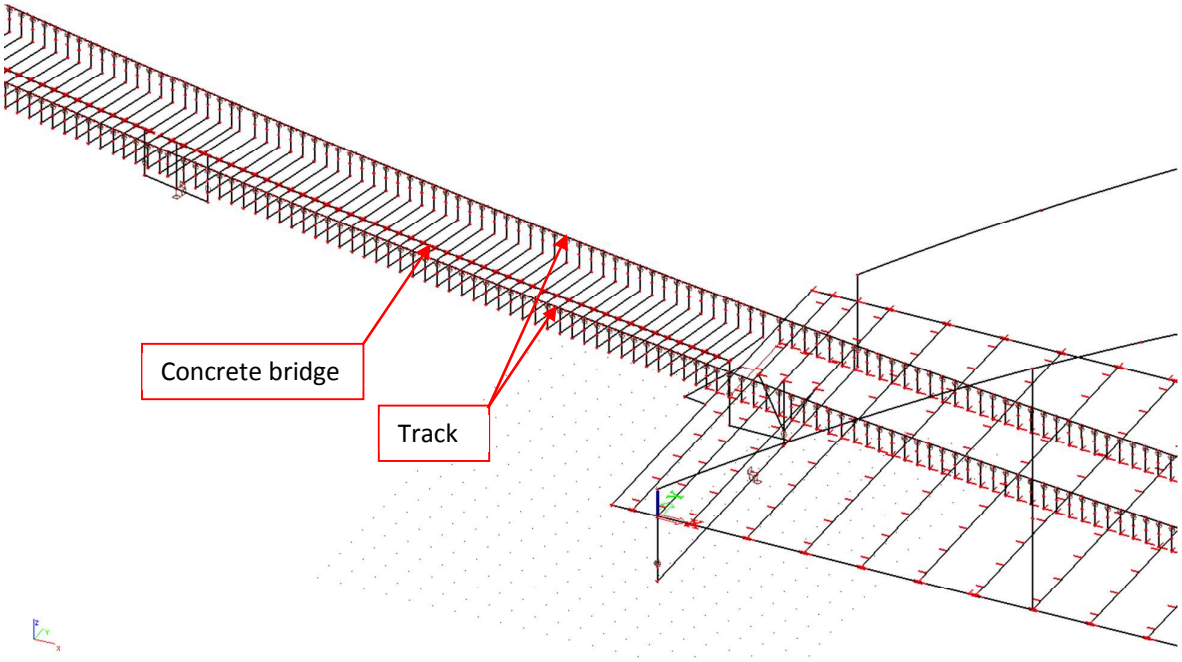

Figure 11: Detail of the model (Rozenburgsesluis) for combined response

6 Dynamics

6.1 Horizontal dynamics

For horizontal dynamic behavior of a railway bridge there are some basic rules given in the Eurocode. According to NEN-EN 1990 art. A.2.4.4.2.4 (3) the horizontal frequency of a railway bridge should not be lower than 1,2Hz. In practice it is impossible for longer bridge to comply with this requirement. There are no further checks or guidelines given in the code for bridges that don't comply. For this project specific requirements have been set if the bridge should not comply with this article. These included dynamic calculation of the horizontal behavior of the bridge, using three different scenarios. The scenarios where:

- Scenario 1:
  - The maximum windspeed is present.
  - The train enters the bridge and due to increasing wind load (on the train) the bridge is brought in a horizontal vibration.
  - A second train enters the bridge from the other side simultaneously.
  - The maximum acceleration of both trains should be less than 0,5m/s².
- Scenario 2:
  - The maximum windspeed is present.
  - The train enters the bridge and due to increasing wind load (on the train) the bridge is brought in a horizontal vibration.
  - A second train enters the bridge from the other side as the bridge has its maximum deflection.
  - The maximum acceleration of both trains should be less than 0,5m/s².
- Scenario 3:
  - The train enters the bridge and due to hunting oscillation a harmonic load is acting on the bridge.
  - The harmonic load induces a horizontal vibration of the bridge.
  - The maximum acceleration of the bridge should be less than 0,5m/s².

At first the horizontal frequency of the bridge is determined. At the bridge over the Thomassentunnel the frequency was slightly

above the requirement at 1,21Hz. Due to uncertainties in the analysis of this frequency, it was decided to check the additional scenarios anyway. For the bridge over the Rozenburgsesluis the frequency was found to be 1,06Hz. For both bridges the additional dynamic calculations have been used to prove the bridge meets the requirements for dynamic behavior.

6.1.1 Results scenario 1 and 2

The analysis of both scenarios is done under the same assumptions. The theory used is:

$$Q_{mod}(t) = \begin{cases} \frac{q_w L_{brug}}{\pi}\left[1-\cos\frac{\pi ct}{L_{brug}}\right] & \text{if } ct \le L_{brug} \\ 2\frac{q_w L_{brug}}{\pi} & \text{if } L_{brug} \le ct \le L_{trein} \\ \frac{q_w L_{brug}}{\pi}\left[1+\cos\frac{\pi(ct-L_{trein})}{L_{brug}}\right] & \text{if } L_{trein} \le ct \le L_{trein}+L_{brug} \end{cases} \quad (1)$$

Interpretation of this theory resulted in the following outcome, given for the governing scenario:

Results bridge over the Rozenburgsesluis:

- Total displacement bridge: 0,0005x8,12=0,0041m
- Total displacement train: 0,0038x8,12 = 0,0308m
- Total acceleration of the bridge: 0,0029 x 8,12 = 0,024 m/s² < 0,5 m/s² OK
- Total acceleration of the train: 0,0113 x 8,12 = 0,092 m/s² < 0,5 m/s² OK

Results bridge over the Thomassentunnel:

- Total displacement bridge: 0,0005x8,19 = 0,0041m
- Total displacement train: 0,0031x8,19 = 0,0254 m
- Total acceleration of the bridge: 0,0046 x 8,19 = 0,037 m/s² < 0,5 m/s² OK
- Total acceleration of the train: 0,0119 x 8,19 = 0,097 m/s² < 0,5 m/s² OK

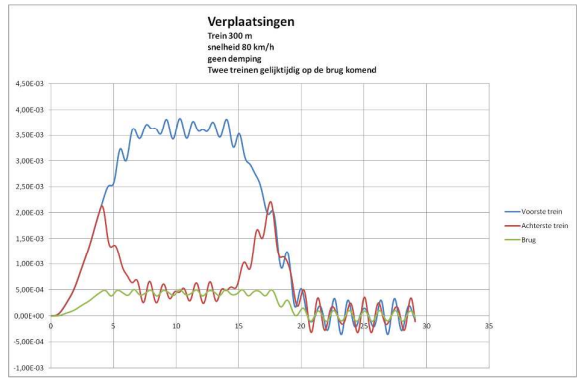

Figure 12: Displacement of bridge and trains

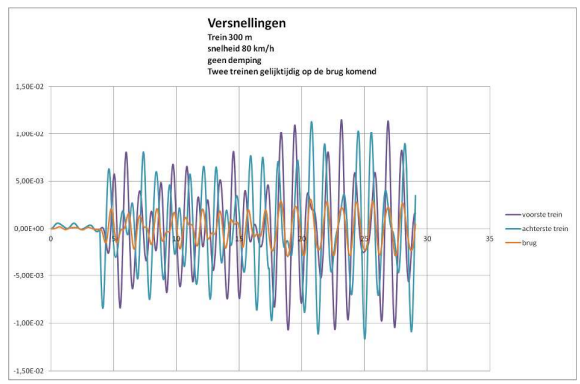

Figure 13: Acceleration of bridge and trains

6.1.2 Results scenario 3

The bridge also complies to the loads and requirements for scenario 3, which was analyzed using the equation:

$$Q_{mod}(t) = \frac{e_0 \Omega_{vet}^2 M\sqrt{N}}{2}\sin(\Omega_{bridge} t) \quad (2)$$

$$\Omega_{vet} = Angular\ frequency\ of\ the\ hunting\ oscillation$$

Further analysis gives us the solution:

Bridge over the Rozenburgsesluis:

$$a_{bridge} = \frac{F_{max} \cdot \omega_{bridge}{}^2}{k_{bridge} \cdot 2 \cdot \zeta} = 0.408 \frac{m}{s^2} \quad (3)$$

Bridge over the Thomassentunnel:

$$a_{bridge} = \frac{F_{max} \cdot \omega_{bridge}{}^2}{k_{bridge} \cdot 2 \cdot \zeta} = 0.498 \frac{m}{s^2} \quad (4)$$

6.2 Vortex excitation

The hangers of the bridge can vibrate in the wind due to vortex shedding (von Karman vortex street). In the Eurocode some formulas for this phenomenon are given. Based on past experience however, for this type of hanger in an arch bridge both the correlation length and the damping are non-conservative.

Based on research done for other bridges, different values are assumed for this project. For the damping

$$\delta_s = 0.001 \cdot 2\pi = 0.0063 \qquad (5)$$

is used, instead of the value 0,02 as defined by EN 1991-1-4 table F.2.

For the correlation length 1/3*L is assumed instead of 6 to 12 times hanger width.

These two adjustments make sure the obtained results are closer to reality. For these two bridges the calculations showed excitations which gave rise to stresses above the cut-off limit of the details used. Since the number of oscillations can quickly become very large, this is an unwanted situation. Therefore we added spirals to the outside of the hangers, to stop the synchronized load from vortex shedding.

Other checks such as rain-induced vibrations and galloping al comply to the requirements as set by the Eurocode.

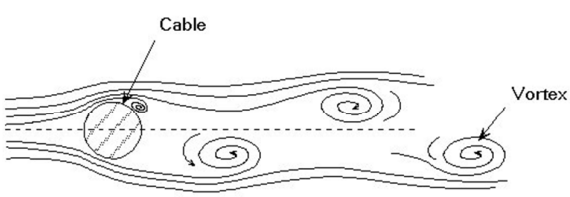

Figure 14: Vortex excitation

7 Fatigue design

Due to the large size of the bridge, the self-weight is high and therefore the stress amplitudes due to a passing train are relatively small. Because of the steel-concrete deck and track in ballast no critical locally loaded steel details are present. By still choosing sensible detailing in the steel-to-steel connections, a large resistance to fatigue is realized.

8 References

Reference [1] is used for the artist impressions of the new bridges

[1] Havenbedrijf Rotterdam. *Theemswegtracé Haven van Rotterdam - Architectonische visie op 4 kilometer nieuw spoor*. Rotterdam; 2017.

[2] Eurocode series