# Puente peatonal de Mary Elmes (Harley Street). Diseño de un puente peatonal en Cork desde el concurso conceptual a la construcción

# Mary Elmes (Harley Street) pedestrian bridge. Design of an urban pedestrian bridge in Cork. From conceptual design to construction

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**RESUMEN**

El puente peatonal de Mary Elmes, constituido por un solo vano de 66 metros sobre el río Lee en el centro de Cork, se abrió al público en Julio de 2019, y es el resultado de un concurso de ideas del ayuntamiento de Cork, en Octubre de 2016. El esquema resistente consiste en una viga de acero en forma de espina de pez, con sección cajón que soporta dos voladizos de anchura variable (2.75 a 3.5 m). La viga metálica varía su posición vertical a lo largo del vano, estando más alta que la zona de paso peatonal en centro de vano y más baja en la zona de apoyos, creando un efecto arco. La estructura es muy esbelta (L/44 en centro de vano y L/32 en apoyos) y es totalmente integral con los estribos. El estudio del comportamiento dinámico fue una parte importante del análisis estructura

**ABSTRACT**

Mary Elmes bridge is a new 66m single span pedestrian and cyclist bridge opened in cork in July 2019, the bridge was selected by Cork City Council as the preferred option in an open competition in October 2016. The structural system consists of a central spine steel beam that supports two cantilevers of varying width (2.75 to 3.5m), the steel beam varies its vertical position along the span, being higher over the walking areas at midspan and under the walking areas at support creating an arching effect. The structure is very slender (L/44 at midspan and L/32 at supports) and it is fully integral with the abutments. The dynamic behavior was a significant part of the design.

**PALABRAS CLAVE:** pasarela peatonal, calculo dinámico, estructura metálica, esbeltez.
**KEYWORDS:** pedestrian bridge, dynamic analysis, steel structure, slenderness.

## 1. Conceptual Design

The competition launched by Cork City council in September 2016 requested a single span pedestrian bridge crossing the river Lee between the historic bridges of St.Patrick's and Brian Boru (see figures 1 & 2). The requirements for a single span, sympathetic with the local architecture while providing a walking with 5m was proven challenging, particularly in the context of a constrained site, with heavily trafficked roads in both quays.

**Figura 1. St. Patricks bridge from Brian Boru's Bridge**

The new bridge is located in between the oldest crossing in Cork, a three-span stone arch from 1789 and Brian Boru bridge a 1911 rolling bascule that was refurbished later as a fixed bridge while leaving the old steel truss.

**Figura 2. Brian Boru bridge from O'Connell bridge**

As an additional challenge, the flooding levels are 300mm higher than the existing footpath at the quays which will require a slender structure in order to satisfy the hydraulic requirements.

The initial concept, developed by the winning team which included ARUP providing Engineering services and Wilkinson Eyre (WKE) providing bridge architecture focused on a structure with a central spine beam (see figure 3) aiming to conceal the structural depth within the railing height.

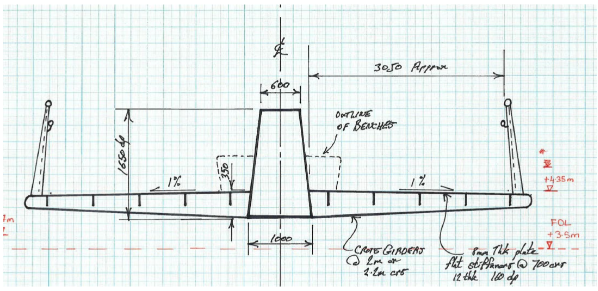



**Figura 3. Conceptual cross section**

It was clear that for a 66m span which is the distance between the quays, a significant depth would be required and also the landings would impose a challenge in providing support while allowing the pedestrians to circulate.

By lowering the position of the beam along the span until it passes under the pedestrian walkway several benefits were achieved:

- A small arch effect (though very shallow) was introduced in addition to the main flexural behaviour of the central box
- If a fully integral abutment with two rows of piles was created, the system behaves as a fully constrained beam in service which allows for an increase in slenderness.
- Finally, if the pedestrian walkway is incorporated into the structural system, the position is very favourable to stiffen the system even further as it acts as a tension flange for sagging (+) moments at midspan and also for hogging moments (-) at supports. Increasing the structural efficiency.

During the conceptual stage, a 1.8m deep beam at midspan was suggested with the beam going to 2.2m deep at the supports.

## 2. Detailed Design

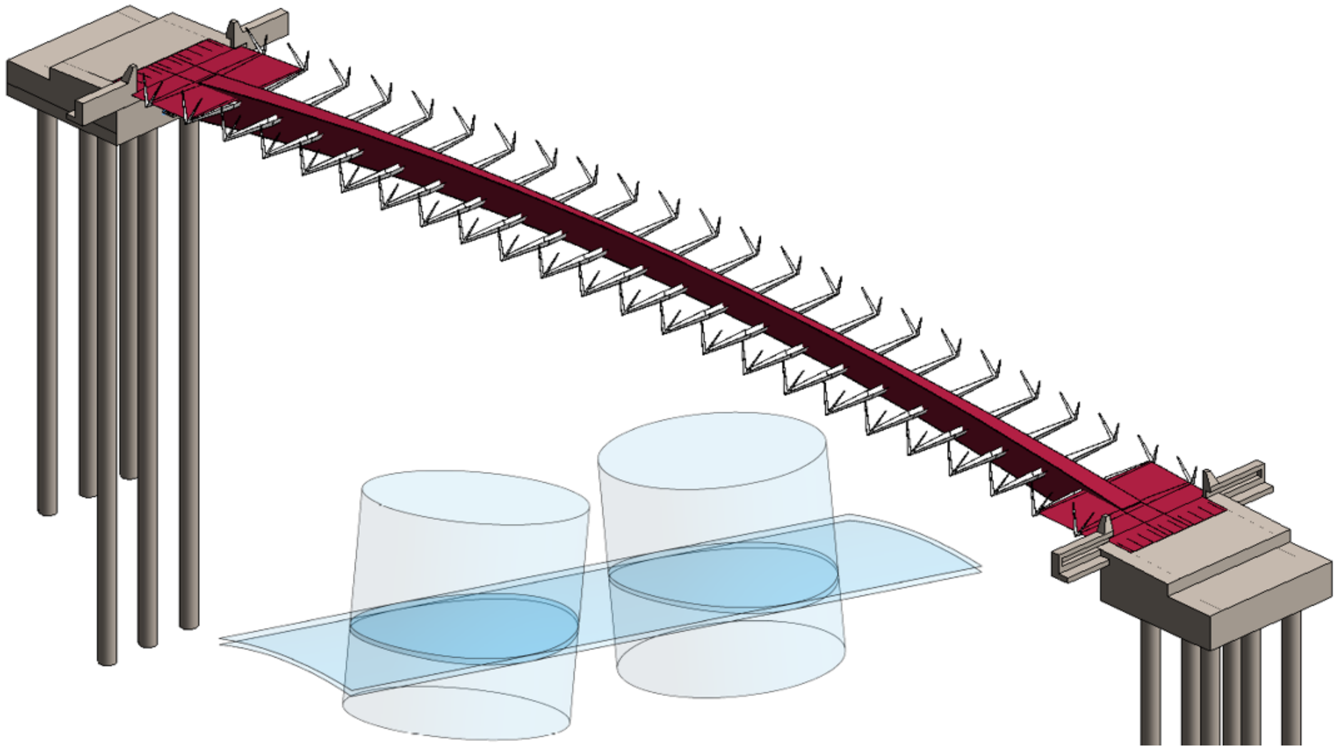
### *2.1 Final bridge geometry*

After the competition award and in discussions with the architect (WKE) the main geometrical parameters for the bridge were set up. From the initial concept, it was clear that the architect required certain elements of the bridge to be respected in the detailed design.

- The top and bottom slab of the main beams will be contained in the surface of large diameter cylinders of slightly different radii, providing the difference in depth in the beam between main span (1.6m) and supports (2.0m).
- The webs will be located in a different cylinder of very large radius, producing a significant variation in width for the top and bottom flange as the intersection with the previous cylinders moves the flanges down towards the abutments.
- The bridge will vary in plan following two different cylinders with maximum

width at midspan and minimum at supports, allowing for bank seats to be placed at midspan without reducing the functional width.

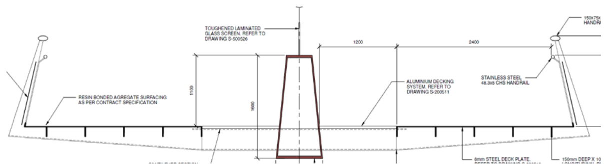
- The railing will have a fundamental role in the structural model with a "V" shaped ribs that will connect into the main beam, developing the transversal behaviour the bridge (see figure 4).



To generate the geometry required for the structural analysis and also the production of the steel drawings from a single source of data, the parametric modelling using Rhino and Grasshopper produced by the architect was transferred to Dynamo for the drawing production and Sofistik for the modelling by developing specific scripts and programming code.

## *2.2 Structural Design. Global static Analysis.*

As indicated in the previous section, the structure is conceived as a fully constrained structure during service. Under pedestrian, wind, and thermal loading, the bridge is fully constrained by the provision of two rows of piles and a concrete pile-cap.



The central box varies in location relative to the pedestrian path but also in width as it follows the large radius cylinders (very close to an inclined plane at 5 º inclined o the vertical) this variation in box size increases the transfer of bending moments at supports (see figures 5 and 6).

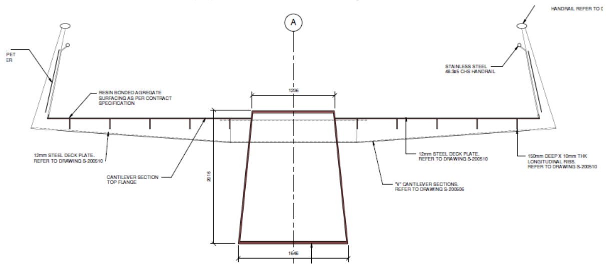


**Figure 6. Cross section at supports**

The plate dimensions of the main beam in the final design were as shown in Table 1 below:

**Table 1. Main box plate dimensions**

| Location | | Plate dimensions (bxt) |
|---|---|---|
| Midspan | Top Plate | 400x50mm |
| | Web | 1600x16mm |
| | Bottom plate | 722x30mm |
| Quarter | Top Plate | 653x50mm |
| | Web | 1720x20mm |
| | Bottom plate | 1200x40mm |
| Supports | Top Plate | 1190x 40mm |
| | Web | 1990x20mm |
| | Bottom plate | 1600x50mm |

These dimensions were the result of the refinement and iterations in the analysis combining the static and dynamic criteria while trying to achieve a structure as slender as possible both in elevation and at midspan in plan.

The global behaviour in service, combines a diaphragm effect of the 10mm thick footways contributing as flanges with a small arching effect which help to reduce the predominant bending in the main box

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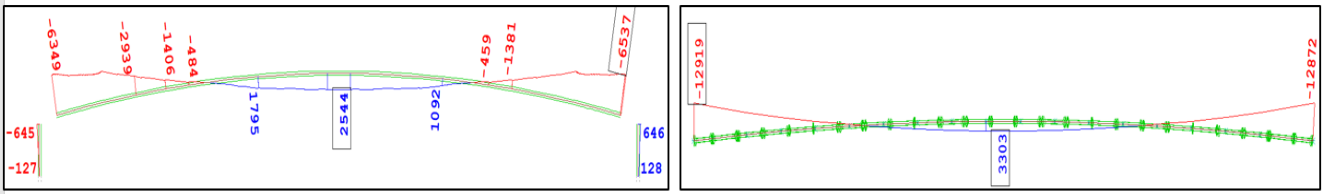


**Figure 7. Main box beam and piles bending forces [kNm] under full pedestrian loading (Left) Integrated bending forces for deck walkway and box beam under full pedestrian load (Right)**

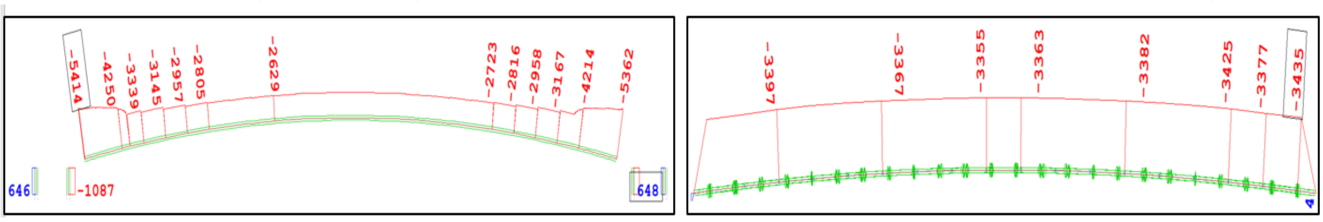


**Figure 8. Main box beam and piles axial forces [kN] under full pedestrian loading (Left) Integrated axial forces for deck walkway and box beam under full pedestrian load (Right)**

As it can be clearly seen from the above diagrams for a fully loaded 5kN/m2 pedestrian loading, the system behaves as a fully constrained beam with the hogging moment (-) even larger than twice the sagging moment (+). It is also important to highlight that the combined moment in the beam $M_{iso}$=2,544+6,539=9,083 kNm represents only 60% of the global bending, the rest being in the arching effect (around 1200 kN axial) and the membrane contribution due to the eccentricity to the beam neutral axis of the pedestrian walkway acting as a tension flange (see figures 7 and 8). Due to these combined effects it is possible to achieve such a slender single box.

## *2.3 Structural Design. Detailing*

### *2.3.1 Abutment*

The abutment consisted of two rows of three piles of 900mm diameter on each abutment. The front row was working mostly in compression while the back row, due to the push-pull effect created by the constraint to rotation imposed on the deck was working in tension. The abutment reinforcement was solved by a series of Strut and Tie models both in plan and elevation.

The abutment deck connection was initially designed as passive by means of steel plates embedded in the concrete but later on changed, upon request of the contractor to a series of 40mm diameter Macalloy bars being stressed after landing the beam in a corbel located at the bottom of the abutment.

2D membrane models, for both the landing and permanent scenarios were developed to verify the flow of forces obtained by the STM models.

### *2.3.2 Deck and ribs*

The railing and ribs, with a marked "V" shape and spaced every 2.7m posed a significant challenge from a structural point of view. Not only was the preferred shape of the ribs triangular, continuing with the inclination of the railing, which is very inefficient structurally. But also due to the requirement of opening a gap in the pedestrian walkway at the maximum transversal bending (close to the box) to provide an architectural grillage for lighting (see figure 9).

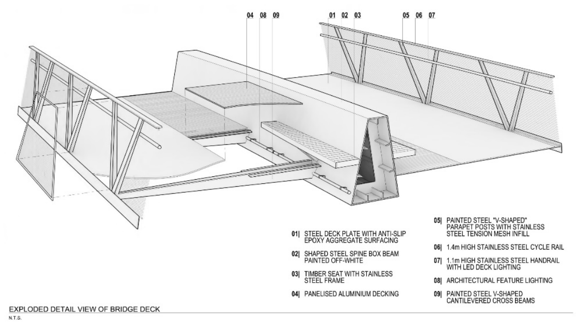


**Figure 9. Architectural concept of the cross section**

In addition to the vertical bending, the ribs are subjected to a significant horizontal shear as they work as a Virendeel beam to transfer the horizontal axial load induced in the main box by the global bending. This value of the horizontal shear, is closer to the vertical shear due to the bending of the cantilever for the full pedestrian loading (see figure 10).

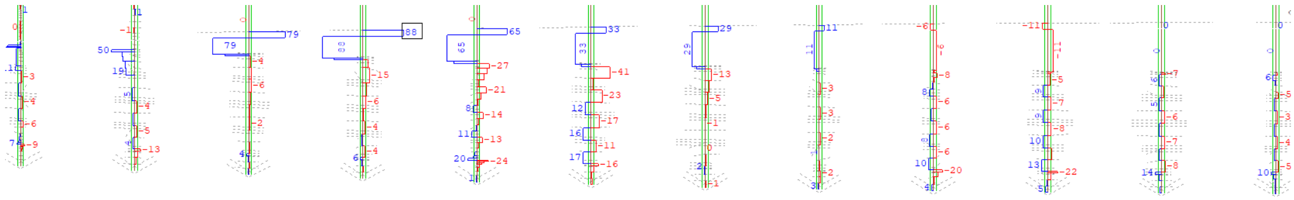

**Figure 10. Horizontal shear in the ribs induced by the global bending (kN)**

In addition, the global geometry of the box, which has every plate curved, and the variability of the pedestrian path in plan and elevation resulting in every rib having a different geometry when intersecting with the main box. In order to simplify the analysis and detailing, the cantilevers where divided in two sections, a constant triangular one (in black in figure 11) and a variable trapezoidal one (in blue in figure 11)

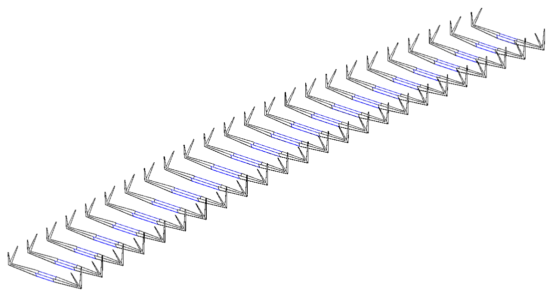

**Figure 11. Rib parametric geometry**

A three-dimensional analysis of the cantilever was developed using Sofistik shell elements (see figure 12). Models for the shortest and longest cantilever were developed, interpolating the results for the intermediate ribs.

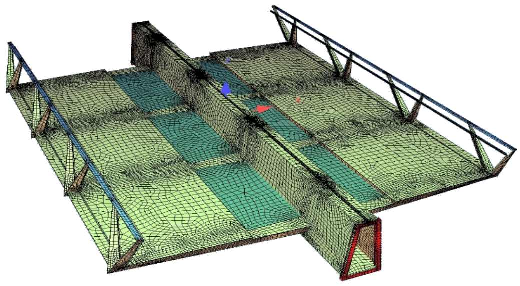

**Figure 12. Three-dimensional model used for the local analysis**

## *2.4 Structural Design. Dynamics*

### *2.4.1 Introduction*

As expected from a lightweight slender structure, the dynamic behaviour had a significant influence in the final design of the structure. In this case, the horizontal vibration was expected not to be a problem, however this was not the case for the vertical vibrations.

The applicable standard, Eurocode 0 [1], requests if the fundamental frequency is lower than 5Hz for vertical and 2.5 for torsional and horizontal a comfort criteria verification should be performed, establishing a maximum acceleration for comfort of 0.7m/s$^2$ for vertical and 0.20 m/s$^2$ for vertical.

This criteria is relatively stringent, when compared, for example, with other European sources such as SYNPEX [2] which establishes the following comfort classes (see table 2):

**Table 2. Defined comfort classes [2] (Cl 4.3.2)**

| Comfort classs | Degree of comfort | Vertical $a_{limit}$ |
|---|---|---|
| CL1 | Maximum | <0.5 m/s$^2$ |
| CL2 | Medium | 0.5 - 1.0 m/s$^2$ |
| CL3 | Minimum | 1.0 - 2.5 m/s$^2$ |
| CL4 | Unacceptable | > 2.5 m/s$^2$ |

Which are less onerous than the Eurocode limits.

Mary Elmes bridge presented 4 modes under 5Hz combining bending and torsional modes (see figures 13 to 16 below):

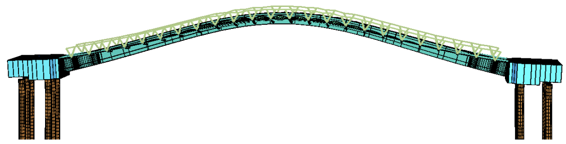

**Figure 13. First mode 2.8 Hz (bending)**

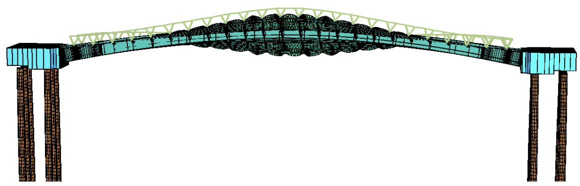

**Figure 14. First mode 3.5 Hz (Torsion)**

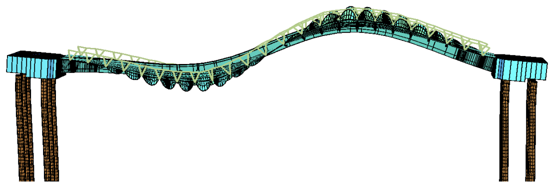

**Figure 15. Third mode 4.8 Hz (Bending 2)**

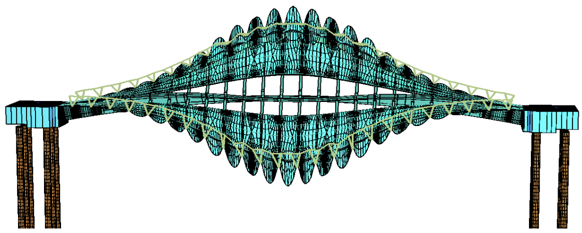

**Figure 16. Fourth mode 4.95 Hz (Bending 2)**

As expected, given the transversal rigidity provided by the walking path and the fixity at the abutments, the horizontal modes are much higher than the vertical.

### *2.4.2 Pedestrian footfall*

For this reason, further analysis on the pedestrian dynamic behaviour was carried out. A new model, using ARUP's own house GSA Oasys was performed in order to verify the dynamic behaviour of the bridge

Using the loading cases recommended by the Irish National Annex to the Eurocodes [4], the following results shown in table 3 below were obtained:

**Table 3. Max vertical accelerations**

| Load | Av max center | Av max cantilever |
|---|---|---|
| Walker | 0.21 $m/s^2$ | 0.29 $m/s^2$ |
| 16xWalkers | 0.37 $m/s^2$ | 0.57 $m/s^2$ |
| 4x joggers | 0.87 $m/s^2$ | 1.22 $m/s^2$ |

As the table above shows, only for the 4 joggers load case, and measuring the acceleration at the end of the cantilever, the acceleration is larger than the 0.7 $m/s^2$, but within the medium comfort criteria of [2], and for the conventional walker crowd expected on the bridge the results are well within the medium or maximum comfort criteria. Consequently, no additional dampers were considered as result of the dynamic analysis.

# 3 Construction

## *3.1 Construction sequence and camber*

Since the structure was procured for construction as a traditional contract which, unlike in a design and build the designer produces the full information of the design in isolation of the contractor's input.

While a fully constrained structure produces a deflection of only 90mm at midspan the flexibility of the foundation against horizontal displacements must be considered, as a consequence of the shallow arch effect, small horizontal displacements on the foundation will have a significant impact in the vertical deflection at midspan. Also, the boundary conditions during construction have a significant impact in the maximum deflection at midspan under permanent loads.

The designer decided to impose a 290mm camber and a limitation on the position of the lifting points (5m from beam ends) along with a lock in device at landing for horizontal displacements if the beam was going to be erected in a single piece with no connection to the abutments until landing as it was the case when the final proposal from the contractor was received.

## *3.2 Landing details*

In order to facilitate the development of the arching effect while landing it was required to constrain the horizontal and rotational displacement of the beam at the supports prior to the completion of the abutment connection a special connection detail consisting of a Macalloy bar reacting against the abutment wall, was developed along with the Contractor (see figure 17).

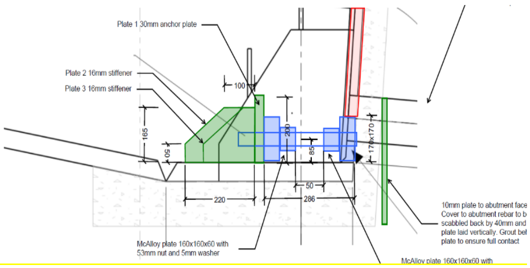


**Figure 17. Local model at the bottom of the beam**

## *3.3 Erection works*

### *3.3.1 Abutments*

The abutments were constructed with temporary traffic management to minimize the disruption of the traffic in the quays as shown in figure 18 below.

They were casting in two stages, allowing for the traffic lanes to be restored as soon as the first stage was completed, reducing the time of a single lane closure in approximately 3 months.

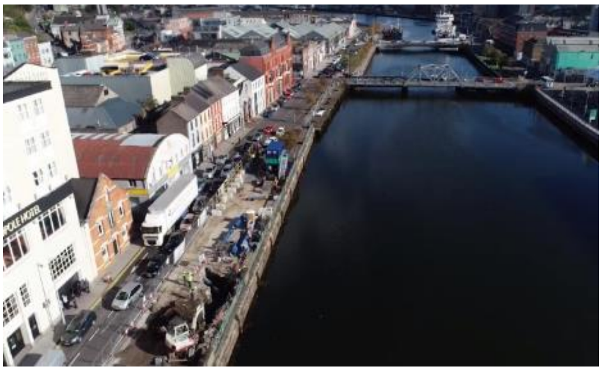

**Figure 18. Aerial view of the North abutment**

*3.3.2 Deck*

The 160 ton deck was fabricated by Thompsons steel in Carlow in full, allowing for the geometry verification of the structure on its final configuration.

**Figure 19. Assembly yard in Cobh**

The steel structure was then cut in pieces removing the ribs and producing three sections for the longitudinal beams and reassembled in a construction yard beside the Lee river in Cobh, located only few miles downstream from Cork city (see figure 19).

**Figure 20. Barge loading in Cobh**

The bridge was then transferred to a floating barge for its transportation to its final location (see figure 20 to 22).

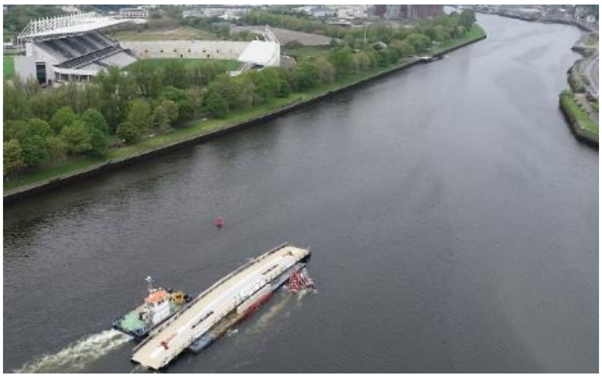

**Figure 21. Bridge floated upstream to Cork city**

The operation of floating under the existing bridges was carried out with maximum accuracy since the tolerance between the river bed and the Brian Boru deck soffit was around 400mm and given the tidal range of the Lee in this location a period of 50 minutes was the maximum window to cross under the existing bridge.

**Figure 22. Bridge crossing under Brian Boru**

The bridge was erected in a single tandem lift using two 750 ton cranes on the $18^{th}$ of May, 2019. As in the previous case, there was a 4 hour window due to the tidal range to complete the lift and secure the landing points both horizontally and vertically.

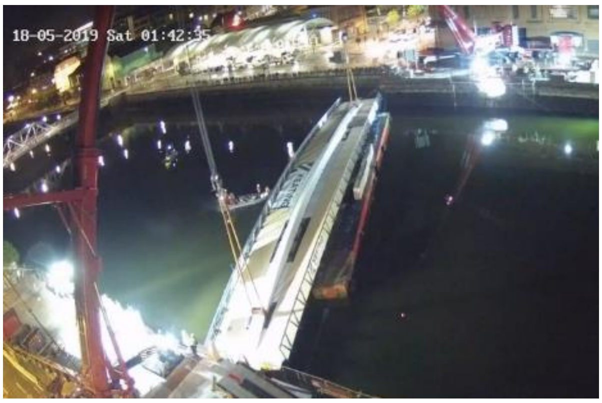


**Figure 23. Bridge erection into final place**

## 4 Post-Construction

### *4.1 Public use*

The bridge has been extremely successful since its opening in July 2019 with great popularity and use for pedestrians, cyclist and surprisingly, people staying in the central benches during the day for hours just enjoying the river views. From a dynamic point of view, while noticeable there is no record of complaints due to excessive vibrations on the structure after several months of continuous public use as shown in figures 24 to 27 below.


**Figure 24. Bridge in high tide daylight**

**Figure 25. Bridge architectural illumination (external)**

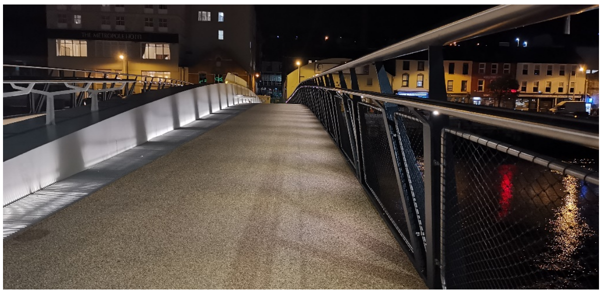

**Figure 26. Bridge architectural illumination (internal)**

**Figure 27. Bridge railing and upstream views**

### *4.2 Post opening dynamic testing*

At the time of writing this paper, the Authors were starting a more detailed calibration of real time measurements on the bridge and comparing them with the model predictions.

As preliminary results, a repeated jumping close to the first mode at midspan has confirmed that accelerations from a reduced number of pedestrians is clearly under 0.6 m/s2 and that the first mode is around 2.7 Hz (see figures 28 and 29).

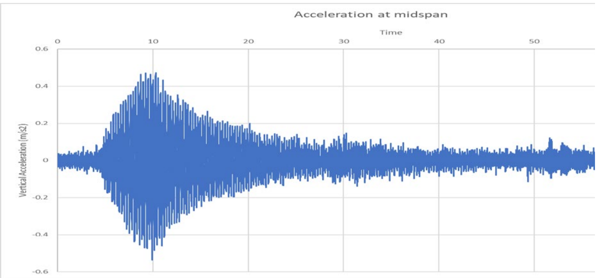


**Figure 28. Bridge railing and upstream views**

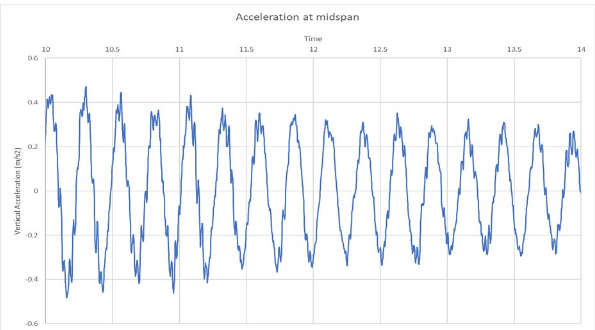


**Figure 29. Bridge railing and upstream views**

It is the intention of the authors to complete a more detailed dynamic analysis with real measurements on the bridge and reproduce them in the model.

## 5. Conclusions

Mary Elmes Pedestrian bridge is a slender single span pedestrian bridge that has raised to the challenge of providing a low profile, while structurally pleasant urban structure, sympathetic with the surroundings.

The bridge required a state of the art approach from the architect and the engineer in order to achieve the final result.

## 6. Acknowledgements

The Authors want to thank John Stapleton, the engineer from Cork city council who was behind this project since its inception and who's contribution throughout all stages has been paramount for the success of the project.

Also, the contribution of the Steel contractor Thomson Steel from Carlow, and the main contractor, L.M. Keating added significant value to the project during different stages.

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[1] EN 1990 (2002) Clause A2.4.3.2 Pedestrian comfort criteria (for serviceability)

[2] Design of Lightweight Footbridges for Human Induced Vibrations. Christoph Heinemeyer, Christiane Butz, Andreas Keil, Mike Schlaich, Arndt Goldack, Stefan Trometer, Mladen Lukić, Bruno Chabrolin, Arnaud Lemaire, Pierre-Olivier Martin, Álvaro Cunha, Elsa Caetano (2009).

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[4] NA to IS EN 1991-2. NA.2.46 Subclause 5.7(3) Definition of dynamic models for pedestrian loads