<table>
<thead>
<tr>
<th><strong>Title</strong></th>
<th>Dynamic Behaviour of Arches</th>
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<tbody>
<tr>
<td><strong>Authors(s)</strong></td>
<td>Fanning, Paul</td>
</tr>
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<td>University College Dublin</td>
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IRS 70778-3 : Appendix G
Dynamic Behaviour of Arches

Professor Paul Fanning
paul.fanning@ucd.ie
Dynamic behaviour of arches

- Assessment of bridges for “real” trains
- UIC 776 – 1R
  - Full dynamic analysis, or
  - Static analysis + dynamic amplification factors
- Dynamic amplification factors
  - Based on simply supported beams
  - Fundamentally not appropriate for arches
  - Overall conservative
WP1.2: Dynamic behaviour of arches

- Phase 1: Desktop study & literature review
  - P1.1 Review and definition of dynamic impact factors
  - P1.2 Origins of existing UIC & Eurocode provisions
  - P1.3 Review and interpretation of available monitored data
  - P1.4 Moving point load models
  - P1.5 Study of velocity effects on existing FE models
WP1.2: Dynamic behaviour of arches

- Phase 2: Experimental Study
  - P2.1 Operational Model Analysis – 2 test bridges
  - P2.2 Quasi-static & dynamic testing
  - P2.3 Guidance notes, further work
Dynamic behaviour of arches

- Assessment of bridges for “real” trains and $\varphi'$
- UIC 776 – 1R, Appendix C
  Static live loads multiplied by: $1 + \varphi$
  Dynamic factor, $\varphi = \varphi' + \varphi''$
  $\varphi' = f(\text{speed, frequency, length})$
  $\varphi'' = f(\text{vertical track irregularities})$

- Determinant length
  Convert arch to equivalent SS beam

- Arches
  - Stiffer, different modes, massive structures, heavily damped, difficult to excite
Origins of existing UIC & Eurocode provisions

- D23 Determination of dynamic forces in bridges
  - (ORE, 1957-1970)
- Combination of experimental, numerical & empirical data
- Experimental data: steel & concrete bridges, no arches
- Numerical studies: British Railways & Czech. State Railways
  - Regression lines fitted to data (D23 RP17 Final Report)
- Final D23 recommendations based on simple theoretical models
- Existing provisions (UIC 776-1R) based on **BUT NOT** identical to these!
- Progression from D23 to UIC 776-1R – not traceable
Review & interpretation of available monitored data

- 3 data sets: Network Rail, Bill Harvey, INES
  - Network Rail variable speeds – most appropriate
  - Bill Harvey single speed
  - INES, 3 bridges along Zaragoza to Alsasua line

- In general:
  - Little or no free vibration response
  - Bridge recovery after bogey traverses
- **P1.3 Review & interpretation of available monitored data**

- **Network Rail data**
  - LEC1/120 Cheddington Bridge, 4.64m

- **Deflection & acceleration data**

<table>
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<td>161</td>
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P1.4 Moving point load models

- Beams & arches:
  - Fundamentally different
  - Equivalent determinant length
    - Not appropriate!

Figure 1.4.4 Dynamic enhancement factors for beam and arch models
Study of velocity effects on existing FE models

- 3D nonlinear models for 2 bridges, static analyses
- Validated against measured deflections with LVDTs
- Transient analysis with increased velocities using these models
- Dynamic amplification factor

\[ \varphi'_{\text{dyn}} = \max \left[ \frac{Y_{\text{dyn}}}{Y_{\text{stat}}} \right] - 1 \]
### Dimensions

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<tr>
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### Masonry material properties

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### Fill material properties

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<tr>
<td>Angle of dilatancy [deg]</td>
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**Griffith Bridge**

31.6 tonnes
• Griffith Bridge

• Modal analysis
  – $f_1 = 17.3 \text{ Hz}$
  – $f_2 = 19.2 \text{ Hz}$
  – $f_3 = 20.7 \text{ Hz}$

• Transient analysis
  – Rayleigh damping 5% at $f_1$ and $f_2$
  – Integration time step = $1/20f$
    $f = \text{max frequency of interest}$
    $\text{ITS} = 1/(20 \times 35) = 0.0014 \text{ s}$
Griffith Bridge

\[ \varphi'_{\text{dyn}} = \max \left( \frac{\gamma_{\text{dyn}}}{\gamma_{\text{stat}}} \right) - 1 \]

Figure 1.5.12 Dynamic factors for Griffith Bridge
### Greenfield Bridge

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<table>
<thead>
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<tr>
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<td>Poisson's ratio</td>
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<tr>
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<table>
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<tr>
<td>Angle of dilatancy [deg]</td>
<td>44.43</td>
</tr>
</tbody>
</table>
- **Greenfield Bridge**
  - **Modal analysis**
    - $f_1 = 20.3$ Hz
    - $f_2 = 26.8$ Hz
  - **Transient analysis**
    - Rayleigh damping 5% at $f_1$ and $f_2$
    - Integration time step = $1/45f$
    - $f = \text{max frequency of interest}$
    - $\text{ITS} = 1/(20*45) = 0.0011$ s

Rayleigh damping

- $f_1 = 20.3$ Hz
- $f_2 = 26.8$ Hz
\( \varphi'_{\text{dyn}} = \max \left( \frac{\gamma_{\text{dyn}}}{\gamma_{\text{stat}}} \right) - 1 \)

- Greenfield

Figure 1.5.14 Dynamic factors for Greenfield Bridge
Guidance Notes

- UIC 776-1R extremely conservative for MAB’s
- Do not condemn bridges based on UIC dynamic amplification factors applied to static loads
- Test data (limited) indicates little/no dynamic attenuation
- Fundamental principles of dynamics apply
  - Loads need to be moving quickly enough to excite bridge
- 2 bridge simulations indicate a value of 0.1 for $\phi'$ would still be conservative
Appendix G contains advice an alternative method to that given in UIC 776-1R for the determination of $\varphi'$ the proportion of factor $\varphi$ for a track in perfect condition.

- Follows fundamentals of structural dynamics
  - Loads need to be moving quickly enough to excite bridge
  - Requires knowledge of frequency of bridge
    - Determine by test or by simulation

- Do not condemn bridges based on UIC dynamic amplification factors applied to static loads.
Appendix G - Alternative method for the determination of $\varphi'$

Appendix G contains advice on an alternative method to that given in UIC 776-1R for the determination of $\varphi'$, the proportion of factor $p$ for a track in perfect condition.

The method is considered suitable for short single span bridges with track in good condition loaded by a passing sequence of bogies. The axle loads used for determining static response should be factored to reflect the track design and train-track interaction (as would be the case for a train running on track where there is no bridge).

The dynamic magnification factor (DMF) is defined as the ratio of dynamic to static response. The DMF for a Single Degree of Freedom (SDOF) system subjected to a load increasing linearly in magnitude to a finite value and reducing again to zero (i.e. a triangular load variation in time) is plotted against the ratio of the total load duration to period of vibration (i.e. period of the natural frequency) in Fig. 52.

A maximum DMF of 1.5 occurs when the ratio of the load duration to the period of vibration is 1.0. As this ratio increases above 1.0 (i.e. the rate of loading becomes slower relative to the natural frequency) the DMF reduces and is 1.0 once the load duration is greater than or equal to twice the period of vibration. As the ratio of the load duration to period of vibration reduces below 1.0 the DMF also reduces. This reduction occurs even though the rate of loading is fast as the load is not on the system long enough for it to respond.

![Graph of DMF against Ratio of Load Duration to Period of Vibration](image)

Fig. 52 - DMF against Ratio of Load Duration to Period of Vibration

The DMF for a SDOF system subjected to a harmonic load is given in Fig. 53 on page 168. The parameter $\beta$ is the ratio of the excitation frequency to the natural frequency. The dynamic magnification factor is insensitive to $\xi$ (the percentage of critical damping, when $\beta$ is not close to 1.0).

When the frequency of harmonic loading is low relative to the natural frequency (i.e. $\beta \rightarrow 0$) the DMF approaches 1.0. When $\beta \approx 1.0$ resonance occurs and there is a large attenuation of response. For $\beta > 1.0$ the DMF reduces.

In the context of a train traversing a bridge, there are at least two scenarios to consider. Individual axles/bogies traverse the bridge. For an individual axle/bogie the load on the
bridge rises from zero and back to zero again (akin to a triangular load distribution) over a period of time dictated by the speed of the train.

Additionally, the bridge is subjected to harmonic loading due to the arrival of successive bogies (potentially axles if the bridge span is short enough).

![DMF for a Single Degree of Freedom (SDOF)](image)

For example, consider an 8 m span masonry arch bridge with a natural frequency of 15 Hz. It is assumed it will be traversed by carriages supported on double axle bogies, arranged as per Fig. 54. The speed to be considered is up to 200km/h (55.6m/s).

![Sample Axle/Bogie Pattern](image)

The crossing time for an axle/bogie is thus 8/55.6 ~ 0.14 seconds. The load on the bridge thus rises from zero to the load applied by the axle/bogie and back to zero in 0.14 seconds. The period of the bridge is 0.067 seconds. The ratio of the load duration to natural frequency is thus 0.14/0.067 ~ 2.09, which is greater than 2.0 so there is no dynamic attenuation – the DMF is taken as 1.0, using Fig. 52 - page 107.

The passage of the train over the bridge results in harmonic loading as successive bogies arrive on it. The arrival time for bogies at the bridge will be 11/55.6 ~ 0.2 seconds. The frequency of arrival is thus 5 Hz. The ratio of excitation frequency to natural frequency, \( \beta \), is thus 5/15 ~ 0.33. The resulting DMF, from Fig. 52 is 1.12.

Hence given a DMF of 1.12 the resulting \( \phi^* \) factor is 0.12 in this case.
Conclusions

- UIC 776-1R extremely conservative for MAB’s
  - Do not condemn bridges based on UIC dynamic amplification factors applied to static loads
- Appendix G is designed to address this
- The most robust solution is a full dynamic analysis
- Substantive further research is required here.