F. Finite Element Models.

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F.1 General

Two types of bridge models have been developed: type I with arch end foundations and type II without foundations, see Figure F1.1.

(a) without displaying of ballast mass points

(b) with displaying of ballast mass points

(c) with displaying of thickness of shell element and profile of beam

Type I: Bridge models **with** solid elements in the two foundations

Type II: Bridge models **without** solid elements in the two foundations

Figure F1.1 The global view of bridge models

The advantage of the former model is that it is closer to the real bridge structure, and the predicted results from it should/could be more reliable and closer to the 'real results', but the disadvantage is that the problem size increases sharply, as shown in Table F.1.

<table>
<thead>
<tr>
<th></th>
<th>Type I including foundations</th>
<th>Type II without foundations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of elements</td>
<td>93,910</td>
<td>47,438</td>
</tr>
<tr>
<td>Number of nodes</td>
<td>102,457</td>
<td>48,171</td>
</tr>
<tr>
<td>Total number of variables</td>
<td>443,800</td>
<td>282,808</td>
</tr>
</tbody>
</table>

Hence, the bridge model Type I, including the arch end foundations, is only used as a **Modal Benchmark model**, and a simplified model without arch end foundations, Type II, is introduced. This simplified bridge model should predicate almost the same modal frequencies.
and modal shapes. Only such a simplified bridge model can be used to predict the dynamic responses under moving loads in a **Dynamic Benchmark model**.

Both types of models are mainly developed with shell elements except that 16 circular columns were constructed with beam elements and two bridge foundations were constructed with solid elements if/when the foundations were included in the model. The track was modelled with point mass/inertia elements and flexible boundary conditions with spring elements (only for the type II model defined below (Note: HERE 'Piers' only refer to the Piers located on the arch end foundations), as shown in Figure F1.1. (Type I model will also be named as *Modal Benchmark model*, and type II as *Dynamic Benchmark model*).

**Beam elements:** 16 circular columns, of which 8 are located in the main span, and the other 8 are located in side spans.

**Solid elements:** The two arch end foundations in model I.

**Point mass/Inertia elements:** the track including ballast were modeled with equivalent mass points attached to the top surface of the bridge decks.

**Spring elements:** For model II without solid element foundations, the translational and rotational boundary conditions as well as rotational springs as extra flexible rotational boundary conditions on the bottoms of arch ends and pier bottoms.

**Shell elements:** All other parts of the bridge structure except the 16 circular columns and the two arch end foundations

The concrete Young’s modulus was set as 40GPa. The density of concrete was 2500 kg/m³, and of the ballast 2000 kg/m³. The longitudinal steel rebars embedded in the concrete was modelled using the embedded Abaqus function 'rebar layer' of shell element, although the reinforcement rebars have very small effect on the results for the modal analysis and dynamic responses in linear elastic stage of the bridge structure.

Simplifications and assumptions should be as few as possible, and the model structure should be equivalent to the original structure (at least mass equivalent, sectional area equivalent and moment inertia equivalent, etc.). The simplifications and assumptions brought into the FE bridge model in Brigade/Abaqus can be summarized as follows:

(i) To simplify the cantilever parts, which is shaped as trapezoids, the cross section of the bridge deck is modelled with rectangular shapes ensuring that the equivalent moment of inertia of the cross section of the bridge deck remains the same after the simplification, see Figure F1.2.

![Standard cross section of bridge deck before simplification](image1.png)

![Standard cross section of bridge deck after simplification in Abaqus/Brigade model](image2.png)
(ii) To assume the discrete reinforcement rebars as a continuous steel layer and to embed these steel layers into the bridge deck, arch and piers.

(iii) Boundary conditions and connections between main span and side spans of the bridge, see section F1.5 below.

The global coordinate system of the Abaqus/Brigade bridge model is defined in the following way:
- X-direction (1-direction): along the bridge direction
- Y-direction (2-direction): vertical direction (gravity direction)
- Z-direction (3-direction): transverse the bridge direction
- X-rotation direction (4-direction): rotation around X-direction using right hand rule
- Y-rotation direction (5-direction): rotation around Y-direction using right hand rule
- Z-rotation direction (6-direction): rotation around Z-direction using right hand rule

The intersections/connections between the different parts of the bridge were simulated as 'fixed connections', where the nodes have the same translations and rotations in all 6 degrees of freedom, if the connection parts are cast-in-site concrete. Hence, all the connections were constructed as fixed connections except the bottoms of circular columns of the side spans and/or the piers (which are located on the top of the arch end foundations), the ends of arch, and the boundaries of the main span and side spans of the bridge. The fixed connections of these cast-in-site parts were realized with the embedded function 'Merge' in Brigade/Abaqus.

For simplicity, the bridge models could be divided into three sub-structures and two features:
- Sub-structure(s) I: Side spans of Bridge model:
- Sub-structure II: Main span of Bridge model:
- Sub-structure III: Arch end foundations (if included in the bridge model) and Piers
- Connection gaps between main span and side spans of bridge model
- Boundary conditions

The detailed developing process of the Abaqus/Brigade bridge model is presented below. Figures are shown displaying the thickness of the shell elements and the profile of the beam elements to facilitate the understanding of the bridge models.

**F1.1 Side Spans (Substructure I)**

Only the left-side span of the bridge model is shown in Figure F1.3, as the right-side span is symmetrical. Strictly speaking, the right-side span is not absolutely symmetrical, since the heights of the circular columns are not the same. It should be pointed out that the heights of these circular columns of side spans are defined according to their real values in our Abaqus/Brigade bridge models. These side span sub-structures can be divided furthermore into even smaller parts; and the developing process (steps) of side span is shown in Figure F1.4.
Figure 1.3 Sub-structure(s) I: Left side span(s) of bridge model

(a) Without displaying shell element thickness and beam element profile

(b) Displaying shell element thickness and beam element profile

(a) Step I: Construct circular columns
(b) Step II: Construct the transverse beams under the bridge deck of the side span

(c) Step III: Construct the longitudinal beams under the bridge deck of the side span

(d) Step IV: Construct the bridge deck of the side span

Figure F1.4 Steps for the left side span of the bridge structure
F1.2 Main Span (Substructure II)
The main span of the bridge model is also composed of several smaller parts, as shown in Figure F1.5

(a) Global view: without displaying shell element thickness and beam element profile

(b) Global view: displaying shell element thickness and beam element profile

(c) The left part of main span of bridge model (The right part is symmetrical)

Figure F1.5 Sub-structure II: Main span of the bridge model
The developing process of the main span is full of challenges and much more complicated than that of the side span. The steps are shown in Figure F1.6.

(a) Step I: Construct the lower flange of the arch. The flange thickness varies, the width also varies.

(b) Step II: Construct shear walls with holes inside the arch.

(c) Step III: Construct circular columns on the top of the shear walls.

(c) Step IV: Construct three arch webs on the top surface of the lower flange of the arch. The web thickness varies, which is not easy to notice in the figure. The intersections between web and cross shear walls can be noticed.
(e) Step V: Construct the top flange of the arch. (The short cantilever part of the top flange is a bit thicker than the mid part of the top flange.)

(f) Step VI: Transverse beams and shear walls on the top of the circular beams and the top flange of the arch

(g) Step VII: Longitudinal beams and solid concrete parts on the top of the circular beams and the top flange of the arch
(h) Step VIII: Construct the bridge deck of the main span

(i) Step IX: Merge/combine the two symmetrical half-main spans to form the whole main span

Figure F1.6 Construction steps for the main span of the bridge model

F1.3 Arch End Foundations and Piers (Substructure III)
The piers located on the top of the arch end foundations are also constructed with shell elements. The equivalent profile is shown in Figure F1.7. The combination of the arch end foundations (if it exists), piers, and side spans and main span are shown in Figure F1.8.
Figure F1.7 The arch end foundations and piers. Left: without displaying shell element thickness
Right: displaying shell element thickness

(a) Combination of arch end foundations, piers and (half) main span

(b) Combination of (a) and side span

Figure F1.8 Combination of arch end foundations, piers, main span and side spans
F1.4 Connection Gaps between Main Span and Side Spans
The detailed profiles of the connection gaps between the main span and the side spans of the bridge model are shown in Figure F1.9. The gap is 70 mm wide.

(a) Without displaying shell element thickness

(b) Displaying shell element thickness

Figure F1.9 Connection gaps (70 mm) between the main span and the inner spans of the bridge model

F1.5 Boundary Conditions
The boundary conditions of the decks of the main span and the side spans, which are located on the top of piers or abutments, were constructed as pinned supports or roller supports in Brigade/Abaqus models strictly according to the real bridge conditions.

The bridge model with boundary conditions is shown in Figure F1.10 and F1.11.

Note: In Abaqus/Brigade Brown marks represent the boundary constraints in translational directions, and Blue marks represent the boundary constraints in rotational directions.

For the two types of bridge models, the boundary conditions are the same except those related to arch ends and pier ends. This will be shown in the following.
Figure F1.10 The bridge model I including arch end foundations with boundary conditions.
(b) Without displaying the ballast mass points

Figure F1.11 The bridge model II without arch end foundations with boundary conditions

F1.5.1 Circular columns in side spans and the outer ends of decks of side spans

Note: the outer ends of side spans refer to the ends of the side spans located on the embankments, see Figure F1.12.

The bottom ends of the circular columns are constrained in X-, Y-, Z-directions, X-rotation and Y-rotation. Only the Z-rotation is free.

The outer ends of side spans are constrained in Y-, Z-directions, X-rotation and Y-rotation. Only the X-direction and the Z-rotation are free.

Figure F1.12 Circular columns in side spans and the outer ends of decks of side spans with boundary conditions.
F1.5.2 The inner ends of the decks of side spans and the ends of bridge deck of main span

The boundary conditions of these ends are achieved by ‘coupling’ function, which is an embedded function in Abaqus/Brigade.

In these bridge models, the inner ends of side spans are coupled with the top sides of the Piers in X-, Y-, Z-directions, X-rotation and Y-rotation but the Z-rotation is free; the ends of the main span are coupled with the top sides of the Piers in Y-, Z-directions, X-rotation and Y-rotation while the X-direction and the Z-rotation are free, see Figure F1.13.

![Figure F1.13](image)

(a) Without shell thickness

(b) With shell thickness

Figure F1.13 The connection part between the inner ends of the decks of side spans and the ends of bridge deck of main span

F1.5.3 Arch ends and Pier ends

Case I: The arch end foundations are included in the bridge model, as shown in Figure F1.14.

The bottom surfaces of arch end foundations are constrained in all the three translation directions, i.e. X-, Y-, and Z-directions.
Case II: The arch end foundations are not included in the bridge model, as in Figure 4.15.

For the arch end surfaces: Set a Reference Point (say RP1, the lower purple point in Figure F1.15) coupling the arch end surface in all the 6 degrees of freedom, and then constrain RP1 in X-, Y- and Z-directions. At the same time set springs attached to RP1 in X-rotation, Y-rotation, and Z-rotation directions with different rotational stiffness values. (The rotational stiffness values of these springs are 2.5e11, 3.0e11 and 1.4e11 Nm/rad, respectively.)

For the pier end surfaces: Set a Reference Point (say RP2, the upper purple point in Figure F1.15) coupling the pier bottom surface in all the 6 degrees of freedom, and then constrain RP2 in X-, Y- and Z-directions and Y-rotation direction. At the same time set springs to RP2 in X-rotation and Z-rotation directions with different rotational stiffness values. (The rotational stiffness values of these springs are 1.2e11 and 1.0e7 Nm/rad, respectively.)
F.2 Comparison of Modal Benchmark Model and Field Measurements

The modal frequencies and modal shapes from the field measurements in September, 2011, see Appendix E, are given in Figure F2.1 together with predicted values from the Abaqus/Brigade model I (Modal Benchmark model). The first results from the FE model (No1 and 2) are related to the swaying motion along the bridge direction, which were not measured in the field measurements, so these two modes could not be compared and only modes No.3 to No.9 of the predicated modal frequencies and mode shapes are listed in Figure F2.1. A very good agreement between measurements and predications can be seen except for the second mode (Vertical-1). This second mode (Vertical-1) has a very small participation factor in the FE model, and indeed in the measured signals this mode is coupled very closely to the third mode (Transverse-2).

Measurements

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency (Hz)</th>
<th>Participation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No.3</td>
<td>1.7819</td>
<td>0.7741 ± 0.06837</td>
</tr>
<tr>
<td>No.4</td>
<td>2.5528</td>
<td>1.144 ± 0.07269</td>
</tr>
<tr>
<td>No.5</td>
<td>3.1697</td>
<td>1.039 ± 0.133</td>
</tr>
</tbody>
</table>

Predications

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No.3</td>
<td>1.79 ± 0.001969 Hz</td>
</tr>
<tr>
<td>No.4</td>
<td>3.062 ± 0.0158 Hz</td>
</tr>
<tr>
<td>No.5</td>
<td>3.184 ± 0.004814 Hz</td>
</tr>
</tbody>
</table>

(a1) Symmetrical, Transverse-1

(a2) Symmetrical, Transverse-1

(a2) Asymmetrical, Vertical-1

(b2) Asymmetrical, Vertical-1

(c1) Asymmetrical, Transverse-2

(c2) Asymmetrical, Transverse-2
It is hard to compare the values above with the ones in the Assessment in section E2 as no characteristics are given there. However, the frequencies for the first five modes in section E2, 2.2 – 7 Hz, are of the same magnitude as the ones above.
F.3 Comparison of Modal and Dynamic Benchmark models (Type I & II)

The first 30 mode shapes and frequencies of the Modal Benchmark model (I) and those of the Dynamic Benchmark model (II) are presented in Figure F3.1 and Table F3.1. For the integrity and completeness of the mode shapes and frequencies of these two bridge models, some modes with very small generalized masses are also listed in Figure F3.1, for instance, modes No. 14-17, and No. 26-27.

Based on Figure F3.1, a very good agreement between the mode shapes and frequencies from the two models can be seen, except that three modes (No. 19, 20 and 21) are shifted (i.e. 19, 20, and 21 of model I correspond to 21,19, and 20 of model II). Although there is mode shift for three modes, the simplified model II without the solid foundation can be regarded as a very good simplification of the Modal Benchmark model I including solid foundation.

Table F3.1. Comparison between measured and predicted Eigen frequencies (Hz)

<table>
<thead>
<tr>
<th>Mode no</th>
<th>Measured values Sept 2011</th>
<th>Type I (with foundation)</th>
<th>Type II (without foundation)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Swaying, Axial-1</td>
<td>0.34185</td>
<td>0.34465</td>
<td></td>
</tr>
<tr>
<td>2. Swaying, Axial-2</td>
<td>0.34767</td>
<td>0.35042</td>
<td></td>
</tr>
<tr>
<td>3. Symmetric, Transverse-1</td>
<td>1.790±0.002</td>
<td>1.7819</td>
<td>1.7771</td>
</tr>
<tr>
<td>4. Asymmetric, Vertical-1</td>
<td>3.062±0.016</td>
<td>2.5528</td>
<td>2.5563</td>
</tr>
<tr>
<td>5. Asymmetric, Transverse-2</td>
<td>3.184±0.005</td>
<td>3.1697</td>
<td>3.2274</td>
</tr>
<tr>
<td>6. Symmetric, Transverse-3</td>
<td>3.436±0.248</td>
<td>3.5023</td>
<td>3.5131</td>
</tr>
<tr>
<td>7. Symmetric, Vertical-2</td>
<td>4.158±0.024</td>
<td>4.2924</td>
<td>4.3991</td>
</tr>
<tr>
<td>8. Asymmetric, Transverse-4</td>
<td>5.015±0.038</td>
<td>5.0425</td>
<td>5.0228</td>
</tr>
<tr>
<td>9. Symmetric, Vertical-3</td>
<td>5.964±0.022</td>
<td>5.8469</td>
<td>5.9792</td>
</tr>
<tr>
<td>10. Sym., Torsion</td>
<td>6.4085</td>
<td>6.4282</td>
<td></td>
</tr>
<tr>
<td>11. Sym., Torsion</td>
<td>7.8407</td>
<td>7.7634</td>
<td></td>
</tr>
<tr>
<td>12. Sym., Torsion</td>
<td>8.1548</td>
<td>8.2255</td>
<td></td>
</tr>
<tr>
<td>13. Asym., Torsion</td>
<td>8.3095</td>
<td>8.3038</td>
<td></td>
</tr>
<tr>
<td>17. Asym., Columns</td>
<td>10.738</td>
<td>10.738</td>
<td></td>
</tr>
<tr>
<td>18. Sym., Torsion</td>
<td>11.133</td>
<td>11.038</td>
<td></td>
</tr>
<tr>
<td>19. Asym., Torsion</td>
<td>11.142</td>
<td>11.349</td>
<td></td>
</tr>
<tr>
<td>20. Sym., Torsion</td>
<td>11.300</td>
<td>11.430</td>
<td></td>
</tr>
<tr>
<td>22. Asym., Vertical</td>
<td>11.811</td>
<td>11.861</td>
<td></td>
</tr>
<tr>
<td>23. Asym., Vertical</td>
<td>11.921</td>
<td>11.943</td>
<td></td>
</tr>
<tr>
<td>27. Asym., Vertical- top beam</td>
<td>13.044</td>
<td>13.044</td>
<td></td>
</tr>
<tr>
<td>No.</td>
<td>Mode shapes and frequencies (Hz)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>-----</td>
<td>----------------------------------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Swaying Axial 1, 0.34185 Hz</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Bridge model with solid arch end foundation
Modal Benchmark model (Original model)

Bridge model without solid arch end foundation
Dynamic Benchmark model (Simplified model)
Swaying, Axial 2  0.34767 Hz

Transverse 1, Symmetric  1.7819 Hz
Vertical 1 Assymmetric 2.5528 Hz

2.5563 Hz

Transverse 2, Assymmetric 3.1697 Hz

3.2274 Hz
Transverse 3, Symmetric  3.5023 Hz

Vertical 2, Symmetric  4.2924 Hz
Transverse 4, Asymmetric  5.0425 Hz

Vertical 3, Symmetric  5.8469 Hz
<table>
<thead>
<tr>
<th></th>
<th>Torsion, Symmetric</th>
<th>Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td></td>
<td>6.4085 Hz</td>
</tr>
<tr>
<td>11</td>
<td></td>
<td>7.8407 Hz</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>Torsion, Symmetric</th>
<th>Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>6.4282 Hz</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7.7634 Hz</td>
</tr>
</tbody>
</table>

F25
<table>
<thead>
<tr>
<th>Torsion, Symmetric</th>
<th>8.1548 Hz</th>
<th>8.2255 Hz</th>
</tr>
</thead>
<tbody>
<tr>
<td>Torsion, Asymmetric</td>
<td>8.3095 Hz</td>
<td>8.3038 Hz</td>
</tr>
</tbody>
</table>
Columns, Asymmetric  9.9038 Hz

9.9055 Hz

Columns, Asymmetric  9.9605 Hz

9.9605 Hz
Columns, Asymmetric 10.619 Hz

Columns, Asymmetric 10.738 Hz

10.625 Hz

10.738 Hz
Torsion, Symmetric  11.133 Hz

Torsion, Asymmetric  11.142 Hz
Torsion, Symmetric 11.300 Hz

Torsion, Symmetric 11.333 Hz
Transverse, Asymmetric, – Side span 12.233 Hz

Transverse, Asymmetric, – Side span 12.264 Hz
Vertical, Asymmetric – Top beam 12.647 Hz

Vertical, Asymmetric – Top beam 12.649 Hz

Vertical, Asymmetric – Top beam 13.044 Hz

Vertical, Asymmetric – Top beam 13.044 Hz
Vertical, Asymmetric – Top beam 13.073 Hz
13.083 Hz

Vertical, Asymmetric – Top beam 13.114 Hz
13.124 Hz
Figure F3.1  Comparison between the mode shapes and frequencies of the original model (I) and those of the simplified model (II)
F.4 Comparison of strains in a longitudinal beam

The dynamic strain is an important factor for the dynamic performance of a bridge structure under train passage. Here four types of moving load will be studied, compare Appendix E:

1. Only one locomotive T43ra240; (180 kN axle load, 720 kN total load)
2. One locomotive and several wagons:
   - Locomotive + 2 wagons, Locomotive + 7 wagons, and Locomotive + 13 wagons;
3. BV3 train, and for simplicity, assume 20 BV3 wagons; (250 kN axle load, 80 kN/m)
4. BV4 train, for the same reason, assume 20 BV4 wagons. (300 kN axle load, 100 kN/m)

A summary of measured and predicted strain envelop values is listed in Table F.4.1. It should be noted that all these strain-envelop values are obtained from the steel strain gauge embedded in the bottom of the observed part of longitudinal beam of Långforsen Bridge, see also Figure F.4.2, which is in '6e-7e' section and marked by red circle.

<table>
<thead>
<tr>
<th>Train type</th>
<th>Speed (km/h)</th>
<th>Strain (µε)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Measurement</td>
<td>Prediction</td>
</tr>
<tr>
<td>Locomotive T43ra240</td>
<td>Approx. 50</td>
<td>-8.3 ~ 23</td>
</tr>
<tr>
<td>Locomotive + 2 wagons</td>
<td>58.5</td>
<td>-11.7 ~ 33.4</td>
</tr>
<tr>
<td>Locomotive + 7 wagons</td>
<td>41.2</td>
<td>-9.6 ~ 20.4</td>
</tr>
<tr>
<td>Locomotive + 13 wagons</td>
<td>33.1</td>
<td>-11.2 ~ 25.6</td>
</tr>
<tr>
<td>BV3: 20 wagons</td>
<td>70 ~ 150</td>
<td>None</td>
</tr>
<tr>
<td></td>
<td>with interval 5 km/h</td>
<td>(See also Figure F4.1)</td>
</tr>
<tr>
<td>BV4: 20 wagons</td>
<td>70</td>
<td>None</td>
</tr>
</tbody>
</table>

The envelop of maximum and minimum strain of observed part of longitudinal beam of Långforsen Bridge is shown in Figure F.4.2, where both the tensile and compressive strains are expressed in positive numbers. It is clear that the strain envelopes are sensitive to the BV3 train with a running speed from 110 to 120 km/h and the extreme maximum/minimum strain envelop occur at speed 115 km/h.

Actually, not only dynamic strains but other dynamic responses of Långforsen Bridge are sensitive to the BV3 with this speed. Moreover, it should be noted that only dynamic responses of Långforsen Bridge induced by BV3 (20 wagons) with speeds from 70 to 150 km/h is calculated in this report, so a BV3 train with speeds higher than 150 km/h, of which dynamic strain/dynamic responses increase dramatically, are also possible.

The maximum steel stress for a strain of $\varepsilon = 50 \cdot 10^{-6}$ will be $\sigma = E \varepsilon \approx 200 \cdot 10^9 \cdot 50 \cdot 10^{-6}$ Pa = 10 MPa.
According to Table F4.1, the predicted strain time histories are close to the measurements and only small differences are present. The reason can be that the FE bridge model is built in the elastic range, and although the real bridge is in a very good condition, it will not be in a perfectly elastic condition, and minor/fine cracks are unavoidable.

The measured and predicted strain time histories of the observed part are shown in Figure 7.3. The calculated acceleration time history in Figure 7.3 (a) and (b) can predict the real acceleration very well.
(a) Measured and predicated strain time history induced by locomotive T43ra240

(b) Measured and predicated strain time history induced by locomotive and 2 wagons

(c) Measured and predicted strain time history induced by locomotive and 7 wagons
Figure F.4.3. Measured (red) and predicted (blue) strain time history of an observed part in a longitudinal beam of the Långforsen Bridge.
F.5 Comparison of measured and predicted accelerations and displacements

In the following sections, the measured and predicted accelerations and displacements of Långforsen Bridge for four load cases in Table F.4.1 are investigated below. Based on the acceleration and displacement analysis in Section F.5.1 and F.5.2, it is shown that the model can predict the real accelerations and displacements very well, so this FE model can be used to predict the dynamic responses of the BV3 train and the BV4 train, as shown in Sections F.5.3 and F.5.4.

The maximum deflection below varies from 1.5 to 5 mm. This is less than half the value 14 mm assessed in section D2 and much less than the allowable deflection 89.5/800 m = 112 mm.

F.5.1 Load Case I: Locomotive (type is T43ra240) at circa 50 km/h

**Note:**

1. The dynamic responses at 'midpoint' and 'quarter point' of the main span of the bridge, shown as two green points, always attract more concern, at which the accelerations and displacements are calculated although the accelerometers are not always put at these two points, as shown in Figure F.5.1.
2. Accelerometers A5 and A6 are always fixed at the two controlling points, and A1, A2, A3 and A4 are the movable accelerometers, which are expressed by two blue points and four red points, respectively, as shown in Figure F.5.2. Test Setup3 captured the measured acceleration of the midpoint of the main span very well for Load Case I, so Setup3 is selected for the analysis for this load case.

![Figure F.5.1 Midpoint and quarter point of main span of Långforsen Bridge](image)

![Figure F.5.2 Test Setup3 of Långforsen Bridge in September 2011](image)
The predicted accelerations at the midpoint and the quarter point of the main span of the Långforsen Bridge are shown in Figure F.5.3. The peak value of predicted accelerations at the midpoint and the quarter point are very close to each other. The predicted displacements are shown in Figure F.5.4.

![Graph showing predicted accelerations under Load Case I](image)

**Fig. F.5.3** Predicted accelerations under Load Case I

The comparison of measured and predicted accelerations at the middle point (Position of accelerometer A1 in this setup) and the inner quarter point (Position of accelerometer A2, A3, or A6 in this setup) of main span is shown in Figure F.5.5. It is clear that the first peak of predicted acceleration curve arrives a bit earlier than the measurement. This is logical because this FE bridge model is a theoretical elastic model and the responses of bridge deck induced by locomotive load can be approximately predicted with the influence line/plane method; when the equivalent locomotive load entered the main span, the midpoint and quarter point of the main span make responses immediately. But it is not true for the real Långforsen Bridge, which is not elastic, and is actually with minor nonlinearity including fine cracks and small damages in its structure after more than 50 years of service. Moreover, the character of ballast on the bridge deck is complicated and it could play a role for delaying the first acceleration peak. In general, according to Figure F.5.5, the predicted accelerations from FE bridge model agree with the measurements very well.

*Note: Only some representative measured results are compared with the corresponding predictions, and plotted here. (The same below)*
Note: Inner quarter point is the accelerometer A2, A3, or A6 point in this test setup, see Figure F.5.2

Fig. F.5.5 Comparison of measured and predicted accelerations at middle point and inner quarter point of the main span under Load Case I

F.5.2 Load Case II: Trains running during test in September 2011

F.5.2.1 A locomotive + 2 wagons

Note:
1. The positions of A5 and A6 are always fixed as before, and for example, in Setup5 the accelerometers A1, A2, A3 and A4 are put on the connection points of column feet and the top flange of the arch, as shown in Figure F.5.6. When Setup5 worked, the locomotive with 2 wagons was running through the Långforsen Bridge with a speed of about 58 km/h, so Setup5 is selected for the analysis for this load case.

The predicated accelerations at the midpoint and the quarter point of the main span for this load case are shown in Figure F.5.7. The peak value of predicted acceleration of the quarter point is larger than that of the midpoint. The predicted displacements are shown in Figure F.5.8.
The comparison of measured and predicted accelerations at the inner quarter point (Position of accelerometer A6) and accelerometer A2 point of main span are shown in Figure F.5.9.

The acceleration curves of the inner quarter point of the main span show that the prediction can capture the maximum value of the measurement, but cannot capture the minimum. The acceleration curves of accelerometer A2 show that the prediction of acceleration underestimates the real acceleration magnitude, but the prediction can act as a reference of the real acceleration. Both predicted accelerations of midpoint and quarter point attenuate more slowly than the measurements.
### F.5.2.2 A locomotive + 7 wagons

**Note:**

1. The locomotive with 7 wagons was running through the Långforsen Bridge with a speed about 41 km/h while Setup7 worked, so this test setup, see Figure F.5.10, is selected for the analysis for this load case.

![Figure F.5.10. Test Setup7 of Långforsen Bridge in September 2011](image)

The predicted accelerations at the midpoint and the quarter point of the main span for this load case are shown in Figure F.5.11. The peak value of the predicted acceleration of the midpoint is larger than that of the quarter point. The predicted displacements are shown in Figure F.5.12.

![Fig. F.5.11. Predicted accelerations under Load Case II: A locomotive + 7 wagons](image)

![Fig. F.5.12. Predicted displacements under Load Case II: A locomotive + 7 wagons](image)

The comparison of measured and predicted accelerations at accelerometer A5 point and A2 point of the main span are shown in Figure F.5.13.
The acceleration curves of accelerometer A5 show that the prediction agrees very well with the measurements. The acceleration curves of accelerometer A2 show that the prediction overestimates the real acceleration magnitude, but the prediction can act as a very good reference of the real acceleration.

Fig. F.5.13. Comparison of measured and predicated accelerations at accelerometer A5 point and accelerometer A2 point of main span under Load Case II: A locomotive + 7 wagons

F.5.2.3 A locomotive + 13 wagons

Note:
1. The locomotive with 13 wagons was running through the Långforsen Bridge with a speed of about 28.5 km/h while Setup 4 worked, see Figure F.5.14, is selected for the analysis for this load case.

The predicated accelerations at the midpoint and the quarter point of the main span of the Långforsen Bridge for this load case are shown in Figure F.5.15. The predicted displacements are shown in Figure F.5.16
The comparison of measured and predicted accelerations at A6 and A3 of the main span are shown in Figure F.5.17. The acceleration curves of accelerometer A6 show that the prediction agrees very well with the measurements. The acceleration curves of accelerometer A3 show that the prediction underestimates the real acceleration amplitude, however, the prediction can act as a very good reference for the real acceleration.
F.5.3 Load Case III: BV3 (20 wagons), speed 70 to 150 km/h, increment 10 km/h

This case includes the speed $= 115$ km/h, from which the approximate maximum dynamic responses are reached.

The summary envelop of the maximum and minimum of the midpoint and quarter point of the main span of Långforsen Bridge is shown in Figure F.5.18. It shows that the sensitive speeds for accelerations of the midpoint and the quarter point of the main span are approximately 100 km/h and 115 km/h, respectively, which agrees with the strain-sensitive speed shown in Figure F4.1. In this figure only the strain near the quarter point of the main span was investigated.

Fig. F.5.18. Summary of predicted acceleration envelop of the mid/quarter point of the main span under Load Case III: BV3 (20 wagons) with speed from 70 to 150 km/h. □ = ¼-point, ◊ = ½-point

F.5.3.1 BV3 run with speed 70 km/h

Fig. F.5.19. Predicted accelerations under Load Case III: BV3 (20 wagons) with speed 70 km/h
Fig. F.5.20. Predicted displacements under Load Case III: BV3 (20 wagons) with speed 70 km/h

F.5.3.2 BV3 run with speed 80 km/h

Fig. F.5.21. Predicted accelerations under Load Case III: BV3 (20 wagons) with speed 80 km/h

Fig. F.5.22. Predicted displacements under Load Case III: BV3 (20 wagons) with speed 80 km/h

F.5.3.3 BV3 run with speed 90 km/h

Fig. F.5.23. Predicted accelerations under Load Case III: BV3 (20 wagons) with speed 90 km/h
Fig. F.5.24. Predicted displacements under Load Case III: BV3 (20 wagons) with speed 90 km/h

**F.5.3.4 BV3 run with speed 100 km/h**

Fig. F.5.25. Predicted accelerations under Load Case III: BV3 (20 wagons) with speed 100 km/h

Fig. F.5.26. Predicted displacements under Load Case III: BV3 (20 wagons) with speed 100 km/h
F.5.3.5 BV3 run with speed 110 km/h

Fig. F.5.27. Predicated accelerations for Load Case III: BV3 (20 wagons) 110 km/h

Fig. F.5.28. Predicated displacements for Load Case III: BV3 (20 wagons) 110 km/h

F.5.3.6 BV3 run with speed 115 km/h

Fig. F.5.29. Predicated accelerations for Load Case III: BV3 (20 wagons) 115 km/h

Fig. F.5.30. Predicated displacements for Load Case III: BV3 (20 wagons) 115 km/h
**F.5.3.7 BV3 run with speed 120 km/h**

Fig. F.5.31. Predicated accelerations under Load Case III: BV3 (20 wagons) 120 km/h

Fig. F.5.32. Predicated displacements under Load Case III: BV3 (20 wagons) 120 km/h

**F.5.3.8 BV3 run with speed 130 km/h**

Fig. F.5.33. Predicated accelerations under Load Case III: BV3 (20 wagons) 130 km/h

Fig. F.5.34. Predicated displacements under Load Case III: BV3 (20 wagons) 130 km/h
**F.5.3.9 BV3 run with speed 140 km/h**

Fig. F.5.35. Predicted accelerations under Load Case III: BV3 (20 wagons) 140 km/h

![Acceleration Time History](image1)

Fig. F.5.36. Predicted displacements under Load Case III: BV3 (20 wagons) 140 km/h

![Displacement Time History](image2)

**F.5.3.10 BV3 run with speed 150 km/h**

Fig. F.5.37. Predicted accelerations under Load Case III: BV3 (20 wagons) 150 km/h

![Acceleration Time History](image3)

Fig. F.5.38. Predicted displacements under Load Case III: BV3 (20 wagons) 150 km/h

![Displacement Time History](image4)
F.5.4 Load Case IV: BV4, run with speed 70 km/h. (20 wagons)

Fig. F.5.39. Predicted accelerations under Load Case IV: BV4 (20 wagons) 70 km/h

Fig. F.5.40. Predicted displacements under Load Case IV: BV4 (20 wagons) 70 km/h