Seismic bridge design according to Eurocode 8 and SIA 160

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ABSTRACT: A case study of comparative seismic design of a reinforced concrete multi-span box girder bridge is presented. The bridge has a floating support system in the longitudinal direction. The seismic action effects are calculated by the equivalent lateral force method and by the multi-modal response spectrum analysis method. Three different design cases of the bridge are compared: „Limited ductility“ and „Ductile behaviour“ according to Eurocode 8, as well as „Nominal ductility“ according to Swiss Standard SIA 160. The study shows that the requirements of Eurocode 8 are more stringent than current Swiss Standards, causing a considerable increase in the seismic action. Even in zones of moderate seismicity, the seismic design situation according to Eurocode 8 will govern the dimensioning of bridge columns.

1 INTRODUCTION

A comparative seismic design of a reinforced concrete multi-span box girder bridge is presented as a contribution to the ENV trial phase of Eurocode 8 (EC 8). The study is part of a joint project between France, Germany and Switzerland. Each of the three countries analyses the same bridge according to EC 8 together with the respective National Application Document and according to the national code. The purpose is to identify major factors contributing to differences between national designs even according to Eurocodes and to show the consequences of the forthcoming transition from national codes to EC 8.

The seismic region selected for the joint project is the southern end of the Rhine Graben, a zone of moderate seismicity next to the city of Basel, where the three countries France, Germany and Switzerland meet. For the design, each country is assuming the highway bridge to be located on its side of the border. In the following, the Swiss contribution to the investigation is summarized. A similar comparative design study for an office building is reported in Wenk & Bachmann (1996).

2 DESCRIPTION OF BRIDGE

The Jules Verne Viaduct, recently built in the Somme Valley in northern France, was utilized as an example bridge structure for the study. The Jules Verne Viaduct crosses over the Somme river and several railway lines. It is a continuous reinforced concrete box girder bridge with 19 spans, normally 50.5 m in length (Fig. 1). The bridge is nearly straight in plan and has a total length of 943 m. The box girder has a constant height of 3.20 m and is post-tensioned in the longitudinal direction. The bridge deck is 10.4 m wide (Fig. 1).

The girder is supported by 18 reinforced concrete bents (Fig. 2). Two columns separated by 3.60 m in the longitudinal direction of the bridge are connected at their tops by a reinforced concrete table to form a bent (Fig. 2). The cross section of the columns is I-shaped with cutoff corners
for reason of aesthetics (Fig. 2). Both columns of a bent are resting on a common shallow foundation where soil conditions allow for. Otherwise pile foundations are provided. The height of the bents varies between 8 m and 22 m. Bent No. 8, investigated further in the chapter stress resultants, is 14.15 m tall. In the transverse direction, the girder is stabilized by fixed bearings on every bent. In the longitudinal direction, a floating support system is realized with fixed bearings on four neighboring bents in the middle of the bridge (bents No. 8 to No. 11) and longitudinally moveable bearings on all other bents (Fig. 7).

The bridge belongs to the importance category „average“ of EC 8, Part 2, Bridges, with an importance factor $I = 1.0$ (EC 8 1994b). Similarly, the bridge is placed into structural class II according to Swiss Standard SIA 160 (SIA 160 1989), the intermediate class of a total of three structural classes.

![Figure 1. Typical cross section of the girder of the Jules Verne Viaduct](image)

![Figure 2. Longitudinal view of a typical bent (left) and cross section B - B through columns of bent (right) of the Jules Verne Viaduct](image)
3 SEISMIC HAZARD

In the Middle Ages, the southern end of the Rhine Graben was hit by strong earthquakes, e.g. the Basel earthquake of 1356 with an epicentral MSK-Intensity of IX to X and a recurrence period of about 1000 years. Currently, the region belongs to the higher hazard zones in the French, German, and Swiss seismic codes. Unfortunately, a direct comparison of the ground accelerations prescribed by the three nations for this area is not possible because to date only Switzerland has published the National Application Document to EC 8 (SIA 460 1997).

According to SIA 160, the area is part of earthquake hazard zone 2 with an effective peak ground acceleration $a_g = 1.0 \text{ m/s}^2$. The existing zoning map of SIA 160 together with the corresponding accelerations has been taken over unchanged by the National Application Document to EC 8. Consequently, a value of $a_g = 1.0 \text{ m/s}^2$ will be assumed for the bridge designs according to EC 8 and SIA 160. The recurrence period of the seismic design event is about 400 years.

4 SEISMIC INPUT

Based on the effective peak ground acceleration $a_g = 1.0 \text{ m/s}^2$ for EC 8 and SIA 160 at the bridge location, horizontal and vertical spectra are graphically represented for various reduction factors in Figures 3-6. It is assumed that all piers and abutments are supported on the same soil condition, namely subsoil class B according to EC 8 Part 1-1 (deep deposits of medium dense sand or gravel with a shear wave velocity of at least 400 m/s). This soil condition is designated as „medium stiff ground“ in SIA 160.

4.1 Horizontal elastic response spectra

The horizontal elastic response spectra of EC 8 and SIA 160 for subsoil class B (medium stiff ground) and for an effective peak ground acceleration $a_g = 1.0 \text{ m/s}^2$ are shown in Figure 3. Both codes assume a reference value of 5% viscous damping for the elastic response spectra. The EC 8 spectrum is calculated using the unchanged boxed values of the spectral parameters given in tables 4.1 and 4.2 of EC 8 Part 1-1 as it was originally prescribed in an earlier draft of the Swiss National Application Document.

![Figure 3. Comparison of horizontal elastic response spectra of EC 8 and SIA 160 for subsoil class B (medium stiff ground) and $a_g = 1.0 \text{ m/s}^2$.](image-url)
At the fundamental vibration period $T_1 = 1.9$ s of the bridge (Fig. 7), the spectral value of EC 8 reaches more than twice the value of SIA 160 (Fig. 3). This is partially due to the exponent $k_{d1} = 2/3$ governing the descending branch of the EC 8 spectrum for vibration periods between 0.6 s and 3.0 s. Instead of declining with constant pseudovelocity, i.e. proportional to the vibration period $T^{-1}$ as the SIA 160 spectrum, the EC 8 spectrum descends somewhat slower proportional to $T^{-2/3}$.

4.2 Horizontal design spectra

The horizontal design spectra (Fig. 4) are obtained by dividing the elastic spectra (Fig. 3) through reduction factors considering mainly ductility in EC 8 and ductility as well as overstrength in SIA 160. EC 8 provides two ductility classes for reinforced concrete bridge piers: „Limited Ductility“ and „Ductile Behaviour“. For „Limited Ductility“, a conventional design for earthquake action is sufficient and the behaviour factor $q$ has to be taken equal to 1.5. For „Ductile Behaviour“, capacity design rules including special constructive detailing have to be observed. Consequently, the behaviour factor can be increased to $q = 3.0$.

As a special feature in SIA 160, overstrength is taken into account over the whole period domain of the spectrum by the design coefficient $C_d = 0.65$ (Bachmann 1995). The design coefficient compensates the difference between the design resistance determined with 2% respectively 5% fractile values of material properties including a safety factor and the probable resistance calculated with average material properties. The total reduction factor $1/C_K = K/C_d$ of SIA 160 has to be compared with the behaviour factor $q$ of EC 8 (Wenk & Bachmann 1995). For reinforced concrete piers and structural class II, the ductility factor $K$ is equal to 2.5 and consequently the total reduction factor becomes $K/C_d = 3.8$.

The following three design cases of the bridge are compared. For each of these the design spectra are given in Figure 4:
- „Limited Ductility“ according to EC 8, Part 2, Bridges with a behaviour factor $q = 1.5$
- „Ductile Behaviour“ according to EC 8, Part 2, Bridges with a behaviour factor $q = 3.0$
- „Nominal ductility“ according to Swiss Standard SIA 160 with a reduction factor $1/C_K = 3.8$

![Figure 4. Comparison of horizontal design spectra of EC 8 and SIA 160 for subsoil class B (medium stiff ground) and $a_g = 1.0$ m/s$^2$.](image)

4.3 Vertical design spectra

In EC 8, the vertical component of the seismic action is defined by the response spectrum for horizontal action multiplied by a factor of 0.5 for vibration periods greater than 0.5 s and by a factor of 0.7 for vibration periods smaller than 0.15 s. Between 0.15 s and 0.5 s a linear interpolation is used. In SIA 160 this factor is $2/3$ for all vibration periods.
Due to the predominantly fragile behaviour of reinforced concrete structures under normal forces, an additional reduction of the vertical spectra by ductility factors cannot be justified (i.e. $q = 1.0$ respectively $K = 1$). However, overstrength is to be considered in SIA 160 by the design coefficient $C_d = 0.65$ for all vibration periods identically to the horizontal design spectrum. For all three design cases the vertical spectra are plotted in Figure 5. The vertical design spectra of the two EC 8 design cases „Limited Ductility“ and „Ductile Behaviour“ coincide as $q = 1.0$ for both cases. It is interesting to see that the variation of the multiplication factor for the vertical spectra independent of the subsoil produces a pronounced peak at $T = 0.15$ s in the present case of subsoil class B.

![Figure 5. Comparison of vertical design spectra of EC 8 and SIA 160 for subsoil class B (medium stiff ground) and $a_g = 1.0$ m/s².](image1)

3.4 Horizontal displacement spectra

The horizontal displacement spectra are not directly defined in the codes. They are calculated by multiplication of the elastic response spectra with the square of the vibration period $d_e = S_e T^2/4\pi^2$ (Fig. 6). Applying the principle of equal displacements, the elastic displacement spectra determined in this way match the design spectra. Subsequently, the displacement spectra for the two EC 8 design
cases „Limited Ductility“ and „Ductile Behaviour“ coincide. Taking the boxed values in table 4.2 of EC 8 Part 1-1 (EC 8 1994a) for the exponents $k_{d1}$ and $k_{d2}$ ($k_{d1} = 2/3; k_{d2} = 5/3$), the ordinates of the displacement spectrum unrealistically tend toward infinity for large vibration periods (Fig. 6).

With the envisaged exponents $k_{d1} = 1$ and $k_{d2} = 2$ in the Swiss National Application Document, the EC 8 displacement spectrum would reach a maximum value of 110 mm for vibration periods greater than 3 s.

5 ANALYSIS METHODS

The three design cases of the bridge, as explained in chapter 3.3, are analyzed by the following two methods:
- equivalent lateral force method
- multi-modal response spectrum analysis

The equivalent lateral force method serves as a simple check for the more complicated three-dimensional multi-modal response spectrum analysis. Both methods assume linear elastic behaviour. The nonlinear material behaviour is approximately taken into account by reducing the elastic seismic action by global reduction factors. The resulting inelastic spectra, called design spectra (Figs 4-5), are then utilized as seismic input for the linear analysis methods.

In EC 8, the simplified analysis method is called „fundamental mode method“. Depending on the characteristics of the bridge, three different approaches are prescribed for this method:
- rigid deck model
- flexible deck model
- individual pier model.

The rigid deck model was applied for longitudinal and transverse seismic action, even though the EC 8 conditions of applicability were not strictly satisfied for transverse seismic action. Actually, the flexible deck model should have been used. But it would require complicated calculations compared to the much simpler rigid deck model not to mention that the flexible deck model formulas 4.12 and 4.13 in EC 8 Part 2 are erroneous. To facilitate the comparison of results, the vibration period for the rigid deck model is taken over from the multi-modal response spectrum analysis, namely $T_1 = 1.9$ s for longitudinal and $T_2 = 0.68$ s for transverse seismic action (Fig. 7).

The multi-modal response spectrum analysis is carried out by a three-dimensional finite element model. Bridge girder and bents are discretized by beam elements as can be seen in Figures 7-8. Rigid boundary conditions are assumed at foundation level of the bents. Further details of the finite element modeling can be found in Hausammann (1996, 1997). The seismic input consists of the design spectra for the three principal directions of the model. The resulting action effects are determined by combining the modal contributions from all three dimensions according to the square root of the sum of the squares rule.

6 STRESS RESULTANTS

The critical structural elements are bents No. 8 to 11, the four bents with fixed bearings in the middle of the bridge. They are affected by transverse and longitudinal seismic action. Among them, the critical sections with the highest stress resultants are at the bottom ends of the columns of bent No. 8. As typical examples of results, bending moments, shear forces, and normal forces are presented in the following for one critical column section of bent No. 8 at foundation level. The first four eigenmodes of the response spectrum model are also shown to illustrate the dynamic behaviour of the bridge.

The other bents with moveable bearings in the longitudinal direction have similar stress resultants as bent No. 8 from transverse but none from longitudinal seismic action. In the bridge girder, the seismic action effects are small and can be ignored.
6.1 Eigenmodes

The first four eigenmodes are shown in Figures 7-8. The fundamental mode with a period $T_1 = 1.9$ s (0.54 Hz) is a vibration of the whole bridge girder in the longitudinal direction (x-direction). The upper part of the four bents with fixed bearings in the middle of the bridge move longitudinally together with the girder as can be seen in Figure 7.

The second mode with a period $T_2 = 0.68$ s (1.5 Hz) is the first mode of vibration in the transverse direction (y-direction) of the bridge girder (Fig. 7). The second mode mainly affects the taller bents close to the abutment in the upper part of Figure 7. The next five modes are higher modes of vibration in the transverse direction. Mode 3 ($T_3 = 0.64$ s) and mode 4 ($T_4 = 0.61$ s) are shown in Figure 8 as examples.

![Figure 7. Mode 1 of the viaduct, $T_1 = 1.9$ s (left); and mode 2 of the viaduct, $T_2 = 0.68$ s (right)](image)

![Figure 8. Mode 3 of the viaduct, $T_3 = 0.64$ s (left); and mode 4 of the viaduct, $T_4 = 0.61$ s (right)](image)

Cumulative curves of the participating mass as a function of the number of modes are plotted in Figure 8 for the three principal directions of the bridge. The first mode, the fundamental vibration in the longitudinal direction, exhibits a participating mass of 67% of the total mass. The 67% approximately correspond to the mass of the girder, the remainder of the mass in the finite element model belonging to the bents.

6.2 Equivalent lateral force method

As example for the action effects of the seismic design situation at a critical cross section, the stress resultants at the bottom of the columns of bent No. 8 are given in Table 1 for longitudinal seismic action and accompanying gravity loads. Also indicated in Table 1 are the spectral accelerations for the fundamental period of vibration $T_1 = 1.9$ s (Fig. 4). For the design case "Ductile behaviour" with $q = 3.5$, the capacity design effects are already included in the shear force $V_x$ by an overstrength factor $\gamma_o = 0.7 + 0.2q = 1.4$, as specified in EC 8.
Figure 9. Participating mass in % for the three principal directions of the bridge as a function of the number of modes considered.

Table 1. Typical stress resultants at bottom of column for longitudinal seismic action

<table>
<thead>
<tr>
<th>Design case</th>
<th>Spectral acceleration [m/s²]</th>
<th>Normal force [MN]</th>
<th>Shear force $V_x$ [MN]</th>
<th>Moment $M_y$ [MNm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>EC 8: Limited ductility</td>
<td>0.79</td>
<td>7.7</td>
<td>1.8</td>
<td>14.2</td>
</tr>
<tr>
<td>EC 8: Ductile behaviour</td>
<td>0.39</td>
<td>6.9</td>
<td>1.4</td>
<td>7.3</td>
</tr>
<tr>
<td>SIA 160</td>
<td>0.15</td>
<td>6.4</td>
<td>0.47</td>
<td>3.0</td>
</tr>
</tbody>
</table>

Table 2. Typical stress resultants at bottom of column for transverse seismic action

<table>
<thead>
<tr>
<th>Design case</th>
<th>Spectral acceleration [m/s²]</th>
<th>Normal force [MN]</th>
<th>Shear force $V_y$ [MN]</th>
<th>Moment $M_x$ [MNm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>EC 8: Limited ductility</td>
<td>1.7</td>
<td>6.1</td>
<td>1.0</td>
<td>17.7</td>
</tr>
<tr>
<td>EC 8: Ductile behaviour</td>
<td>0.83</td>
<td>6.1</td>
<td>0.70</td>
<td>8.9</td>
</tr>
<tr>
<td>SIA 160</td>
<td>0.44</td>
<td>6.1</td>
<td>0.27</td>
<td>4.7</td>
</tr>
</tbody>
</table>

Table 2 summarizes the analogous results for the seismic action in the transverse direction together with the spectral accelerations for the second period of vibration $T_2 = 0.68$ s (Fig. 4). The Moment $M_x$ designates strong axis bending of the column cross section shown in Figure 10, while $M_y$ produces weak axis bending.

6.3 Multi-modal response spectrum analysis

Table 3 shows the stress resultants for the same critical cross section at the bottom of the columns of bent No. 8 obtained by the multi-modal response spectrum analysis. The contributions from the seismic excitation in the three principal directions are already combined with the gravity load action effects. The shear forces of design case „Ductile behaviour“ take into account the overstrength factor $\gamma_o = 1.4$, as previously explained for Tables 1-2.
Table 3. Typical stress resultants at bottom of column for longitudinal and transverse seismic action

<table>
<thead>
<tr>
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<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>EC 8: Limited ductility</td>
<td>7.4</td>
<td>1.7</td>
<td>16.0</td>
<td>0.75</td>
<td>14.0</td>
</tr>
<tr>
<td>EC 8: Ductile behaviour</td>
<td>6.8</td>
<td>1.4</td>
<td>8.2</td>
<td>0.52</td>
<td>7.0</td>
</tr>
<tr>
<td>SIA 160</td>
<td>6.4</td>
<td>0.49</td>
<td>3.3</td>
<td>0.21</td>
<td>3.9</td>
</tr>
</tbody>
</table>

7 RESISTANCE VERIFICATION

In Figure 10 the original reinforcement of a column of the Jules Verne Viaduct is schematically shown. A verification of the resistance indicated that this cross section just satisfies the requirements for biaxial bending with normal force of design case SIA 160 (Table 3). For design case EC 8 „Ductile behaviour“, considerably more longitudinal reinforcement has to be provided together with a stronger confinement reinforcement. The vertical spacing of the hoops and crossties has to be reduced from a maximum of 15 times the diameter of the vertical bars for design case SIA 160 to 6 diameters for design case EC 8 „Ductile behaviour“.

The much larger stress resultants of EC 8 design case „Limited ductility“ (Table 3), cannot be accommodated by the given concrete section. The cross section of the columns would have to be substantially enlarged and heavily reinforced. As a consequence, the EC 8 design case „Limited ductility“ would lead to an uneconomical and less esthetic design without presenting any advantages over the design case „Ductile behaviour“.

Figure 10. Cross section through column showing original vertical reinforcement in lower half and confinement reinforcement in upper half of cross section.

8 SPAN UNSEATING

As a conceptional measure to prevent span unseating, both codes require a minimum overlap length of the supports. The girder of the Jules Verne Viaduct being continuous over all 19 spans, this
The measure has to be observed at the abutments only. Table 4 summarizes the minimum overlap length together with the peak relative girder displacement in the longitudinal direction. The difference of minimum overlap lengths of 15 cm between design cases is mainly due to the fact the EC 8 prescribes a minimum support length of 40 cm for safe transmission of vertical reaction forces, while 20 cm are sufficient for SIA 160.

Table 4. Minimum overlap length at abutments to prevent span unseating

<table>
<thead>
<tr>
<th>Design case</th>
<th>Peak girder displacement [cm]</th>
<th>Minimum overlap length [cm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>EC 8: Limited ductility</td>
<td>11</td>
<td>69</td>
</tr>
<tr>
<td>EC 8: Ductile behaviour</td>
<td>11</td>
<td>69</td>
</tr>
<tr>
<td>SIA 160</td>
<td>5</td>
<td>54</td>
</tr>
</tbody>
</table>

To satisfy the serviceability limit state requirements of EC 8, the peak girder displacement under the seismic design event (Table 4) has to be considered for the dimensioning of the expansion joints, even for bridge importance category „average“. On the other hand, SIA 160 does not require a check of the serviceability limit state for structural class II, accepting repairable damages to expansion joints.

9 COMPARISON OF RESULTS

A comparison of Tables 1-2 with Table 3 points out that the simple lateral force method is sufficient to determine the stress resultants at the critical sections and the much more complicated multi-modal response spectrum analysis is not justified in this case. The higher modes only have a noticeable influence in the transverse direction, where the stress resultants are about 25% higher using the multi-modal response spectrum analysis.

Taking design case SIA 160 as reference, the ratios of the spectral accelerations of for EC 8’s design cases „Ductile behaviour“ and „Limited ductility“ are 1 : 2.6 : 5.2 for the longitudinal and 1 : 2 : 4 for the transverse direction (Tables 1-2). These spectral accelerations determine the response in the dominant mode for the two principal directions. The influence of gravity loads and, to a lesser extent, of higher modes on the stress resultants slightly reduces these ratios to 1 : 2.4 : 4.8 for weak axis bending moments and 1 : 1.8 : 3.6 for strong axis bending moments in the columns (Table 3). In other words, EC 8 design case „Limited ductility“ results in column moments five times those of design case SIA 160. EC 8 design case „Ductile behaviour“ yields twice the column moments of design case SIA 160.

The following factors contribute to this significant increase of bridge column stress resultants:
- Neglecting overstrength, EC 8 stress resultants are a priori 50% higher than in SIA 160 where overstrength is compensated by a factor $C_d = 0.65$.
- The relatively optimistic estimation of ductility in SIA 160 yields a further increase of approximately 50%. In SIA 160, a ductility factor of 2.5 can be taken into account without any special ductility-enhancing detailing compared with 1.5 in EC 8.
- Higher amplification factors of the elastic spectrum in EC 8 result in about 50% greater spectral accelerations for the relevant vibration periods.
- For vibration periods greater than 0.6 s, the EC 8 spectrum descends somewhat slower to add additional conservatism for slow vibrating large structures. It is planned to correct this point in the Swiss National Application Document.

EC 8 offers a way to partially compensate for the general increase of the seismic action effects by selecting a higher ductility class. In most cases, the ductility class „Ductile behaviour“ will be more economical and, thanks to capacity design, will lead to safer seismic performance than using duc-
tility class „Limited Ductility“. The higher shear forces of EC 8 are not critical in this example due to the relatively large cross section of the columns.

10 CONCLUSIONS

The comparison of seismic design cases clearly shows that EC 8 requirements are more stringent for reinforced concrete bridge columns than the current Swiss Standards. As an extreme value, the column bending moments of design case EC 8 „Limited ductility“ were five times those of design case SIA 160. Even for design case EC 8 „Ductile behaviour“, the stress resultants were more than twice those of design case SIA 160.

As a consequence, the seismic design situation according to EC 8 will in general govern the dimensioning of bridge piers not only in zones of high seismicity, but also in zones of low seismicity, e.g. in the Swiss earthquake hazard zone 2 with an effective peak ground acceleration $a_g = 1.0 \text{ m/s}^2$.

The EC 8 ductility class „Limited Ductility“ closely corresponds to the SIA 160 ductility class „Nominal ductility“ for design and constructive detailing. But as a general rule, EC 8 ductility class „Limited Ductility“ cannot be recommended for design due to the high stress resultants. Selecting ductility class „Ductile behaviour“ will be more economical and lead to safer seismic performance.

However, ductility class „Ductile behaviour“ is more demanding in design and construction and therefore its application requires special education in the introductory phase.

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