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Limitations for a ductile design of bridges

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ABSTRACT: Most bridges all over the world have been designed for earthquakes using quasi-static approaches. In many cases simple design procedures are sufficient, but criteria and guidelines determining the limits of applicability are necessary. The work of BVFA-Structural Dynamics and Technical University Graz strongly concentrates on this point.

Large prestressed bridges are typical for Alpine regions of Central Europe. For low and medium sized bridges often the ductile behaviour of piers is taken into account. However, considering the serviceability limit state, the structure should not undergo severe nonlinear deformations. Ductile concepts for large bridges are considered to be efficient only if the plastic hinges can be repaired after the event. Hence, it is concluded that the behaviour should be nearly elastic. In EC8/part II a behaviour factor $q < 1{,}5$ is suggested for bridges with low ductility.

For Lavant bridge a simplified FE-model was elaborated. Response spectra calculations were carried out using spectra for Central Europe. The maximum acceleration was calibrated to 0,4 g which could be the acceleration of the maximum credible earthquake in Austria. Considering the maximum bending moments, piers were reinforced in that way that yielding will start under the maximum bending moment. Then, using the classical formulas, displacement ductility was estimated. It was shown, that the available displacement ductility is within $1{,}82 < \mu_s < 4{,}5$. Although more research work is necessary on the ductility of large piers with hollow rectangular cross sections, it is concluded that a behaviour factor of $q = 1{,}5$ is realistic in the case of Lavant bridge, providing a maximum earthquake capacity without severe damages up to the maximum acceleration of $a_{max} = 0{,}6$ g.

1. INTRODUCTION

Most bridges all over the world (except special structures like suspension bridges and cable stayed bridges) have been designed for earthquakes using quasi-static approaches [10]. In many cases simple design procedures are sufficient, but criteria and guidelines determining the limits of applicability are necessary. The work of BVFA/Structural Dynamics and Technical University Graz strongly concentrates on this point. Results of the investigations are given in [5-8]. Large prestressed bridges are typical for Alpine regions of Central Europe. In our opinion the main open problems are:

- which method of calculation must be used for a certain project. Which method gives the most realistic results, a high degree of safety with an acceptable amount of design work?
- in which situation travelling wave effects have to be considered?

It is assumed that in many cases response spectra calculations will be the adequate design method for large bridges. Criteria for the limits of applicability will be provided by TU-Graz in forthcoming papers.

Considering the serviceability limit state, the structure should not undergo severe nonlinear deformations. On the other hand, in regions of moderate to high seismicity, it is often uneconomic to design bridges for severe earthquakes without providing reliable means to dissipate significant amounts of seismic energy. This can be achieved e.g. by a ductile design of piers or by special energy dissipating devices.

The main aspects of displacement ductility of bridge piers are discussed in what follows:

* there is no clear evidence from real earthquakes about the behaviour of the potential plastic hinge zones of a bridge pier.
* work was done on curvature ducility of different pier sections, but a realistic estimation of the length of the plastic hinge is difficult. The length can be approximated by well known

classical formulas (theoretical formulas or experimentally obtained empirical formulas for relatively short cantilever piers, e.g. [1.4]) but more research work is necessary especially for high piers with large hollow-rectangular cross sections.

* in addition to the problems mentioned above, a considerable part of the total displacement at the top of the pier can be due to deformations of components which remain elastic after the formation of plastic hinges. Such elastic contributions result from rotations of the foundation and deformation of elastic bearings. Further, especially in the case of large bridges the deformation of the pier will be different to the deformation of a cantilever beam with a concentrated mass at the top, which is the classical assumption for the calculation of the plastic hinge length.

* in general, the appropriateness of ductility concepts is debatable especially for high piers. For bridges, ductility is considered to be efficient only if the plastic hinges can be repaired after the event. Hence, it is concluded that the behaviour should be nearly elastic.

Tab.1

Modal frequencies:

Nr	Hertz						
1	0.317	13	2.090	25	3.594	37	5.842
2	0.420	14	2.186	26	3.716	38	6.267
3	0.498	15	2.222	27	3.864	39	6.304
4	0.635	16	2.310	28	4.947	40	6.398
5	1.076	17	2.326	29	4.963	41	6.847
6	1.195	18	2.518	30	5.061	42	6.918
7	1.236	19	2.620	31	5.174	43	6.973
8	1.270	20	2.643	32	5.202	44	6.997
9	1.408	21	2.862	33	5.258	45	7.015
10	1.589	22	3.190	34	5.538		
11	1.878	23	3.394	35	5.603		
12	2.038	24	3.513	36	5.808		

Tab. 2

Pier	Point	Transvers. excitation			Longitud. excitation			Longitud. excitation without el. 117 and 118		
		M_{max} [MNm]	V_{max} [MN]	d_{max} [mm]	M_{max} [MNm]	V_{max} [MN]	d_{max} [mm]	M_{max} [MNm]	V_{max} [MN]	d_{max} [mm]
1	3	26,5	8,8	41	193,2	8,2	65	192,3	8,2	64
	4	204,3	9,4	13	25,0	8,5	28	24,5	8,5	28
	5	433,6	9,4	0	226,5	8,5	0	225,4	8,5	0
2	13	20,6	9,4	71	169,8	7,7	65	164,2	7,2	65
	14	463,2	9,4	46	243,4	7,7	46	236,2	7,2	45
	15	269,2	16,8	23	211,3	12,0	22	202,7	11,4	21
	16	649,0	16,8	0	429,0	12,0	0	409,9	11,4	0
3	24	14,8	8,2	102	173,6	7,9	66	211,3	8,5	66
	25	406,0	8,2	59	250,0	7,9	46	263,9	8,5	49
	26	274,4	14,7	24	215,4	12,3	22	192,0	11,5	24
	27	624,0	14,7	0	438,8	12,3	0	442,0	11,5	0

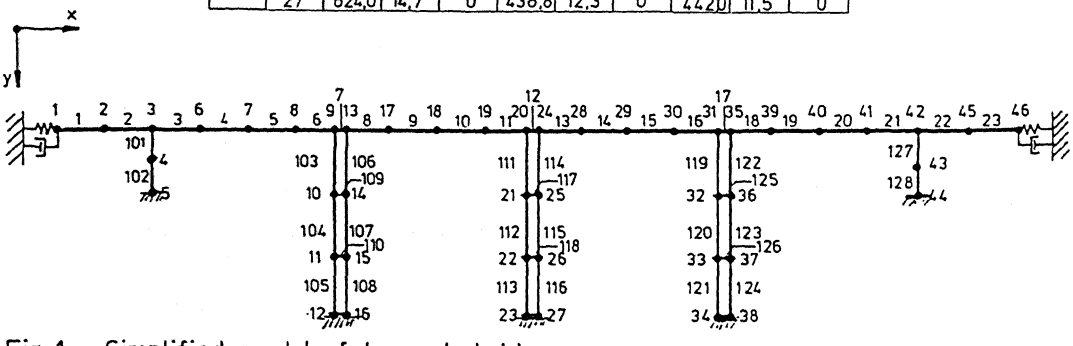


Fig. 1 Simplified model of Lavant-bridge

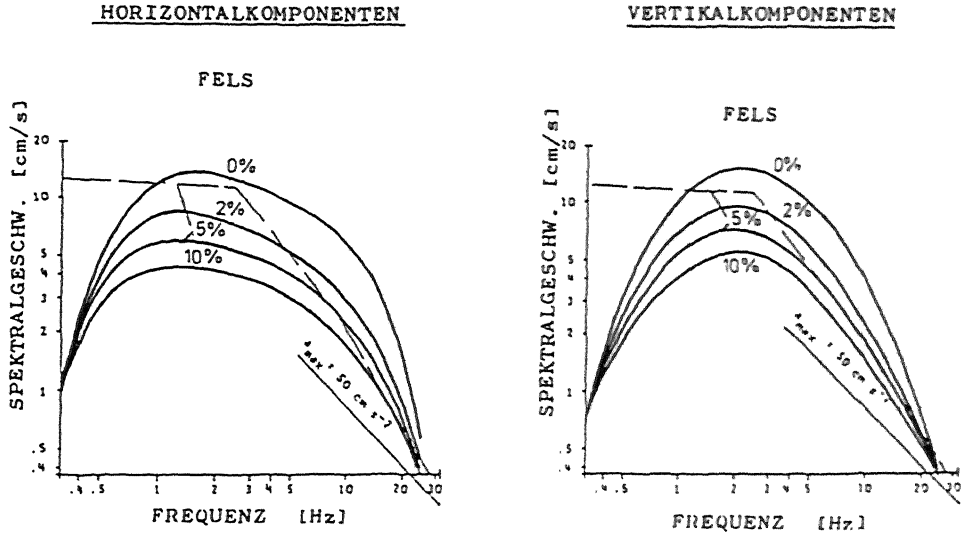


Fig.2 *response spectra from [9]*

To get a first idea, whether a low behaviour factor (q= 1,5) is sufficient for large bridges, a simplified model of Lavant bridge was used. First results were also given in [10, 11]. Using the classical formulas, an attempt was made to estimate the available displacement ductility for pier 2 and 3.

2.FUNDAMENTAL CONCEPT FOR THE EARTHQUAKE RESISTANT DESIGN

2.1. Philosophy

The seismic design philosophy of bridges is based on the general requirement that communication should be maintained with appropriate reliability, after the design seismic event. The fundamental requirements are defined in EC 8/part II-bridges:

- non collapse requirement (ultimate limit state)
- serviceability limit state. A sufficient capacity for seismic- and other loads must be attainable by repair work after the design earthquake.

The repair of plastic hinges of large piers could become difficult, hence nearly elastic behaviour (q=1,5) is required.

The concept of distribution of earthquake loads to the piers and abutements is shown in fig. 1. All piers are framed to the girders. At the abutements, the girders are assumed to be fixed in transverse direction, while spring-damper devices are acting in longitudinal direction. A spring constant equivalent to the bending resistance of pier 3 was selected for the devices.

2.2. Codes

The design is based on the following codes:

(1) EC no.8, especially part II - bridges (draft 1990)
(2) ÖNORM B 4040 (Austrian Code) general principles on reliability for structures
(3) ÖNORM B 4015, part 1+2 Austrian Seismic Code, draft 1990

2.3. Modelling

2.3.1. Model of exicitation

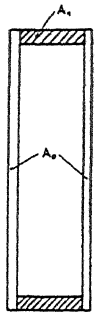
For the calculations, response spectra elaborated for Central Europe (by Grossmayer [9]) were used. The spectra (for rock, for vertical and horizontal excitation) are given in fig. 2. These spectra are in close agreement with a simplified response spectrum used in ÖNORM B 4015 and with the spectrum of EC 8.

According to EC 8/part II a viscous damping ratio ξ = 5 % was used for all modes.

The spectra were calibrated to a maximum acceleration of 0,4g which could be the acceleration of the maximum credible earthquake in Austria (the maximum design acceleration beeing 0,12g for seismic zone no. 4).

2.3.2. Model of structure

The original FE-Model of Lavant bridge [5, 6, 7] was



available ductility

(model: I-cross section)

most important factors: γ, n

$A_1 = 2{,}4 \times 0{,}5 = 1{,}2\,m^2$

$A_0 = 2 \times 0{,}3 \times 10 = 6{,}0\,m^2$

$\gamma = \frac{A_1}{A_0} = \frac{1{,}2}{6{,}0} = 0{,}2$

$A_b = A_0 + 2A_1 = 6{,}0 + 2{,}4 = \underline{8{,}4\,m^2}$

$f_{ck} = 30$ MPa

$N_d = 7{,}1 \cdot 10^7$ N

$n = \frac{N_d}{f_{ck} \cdot A_b} = \frac{7{,}1 \cdot 10^7}{3 \cdot 10^7\, 8{,}4} = \underline{0{,}28}$

from Fig. 8: $n = 0{,}28$, $\gamma = 0{,}2 \longrightarrow \mu_\varphi = 4$

$L_p = (0{,}08 \cdot 150 + 6 \cdot 0{,}032)(0{,}5 + 1{,}67 \cdot 0{,}28)$

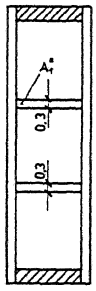
$= 11{,}8\,m$

$\mu_\delta = 1 + (\mu_\varphi - 1)\frac{3 L_p (L - 0{,}5 L_p)}{L^2}$

$= 1 + 3\,\frac{3 \cdot 11{,}8\,(150 - 5{,}9)}{150^2} = \underline{1{,}68}$

Fig. 3 Estimation of ductility for y-direction

improvement cross-section



$A_b = A_0 + 2A_1 + 2A_1^* = 8{,}4 + 2 \times 2{,}4 \times 0{,}3 = \underline{9{,}84\,m^2}$

$n = \frac{7{,}1 \cdot 10^7}{3 \cdot 10^7 \times 9{,}84} = \underline{0{,}24}$

$n = 0{,}24$, $\gamma = 0{,}2$ (or greater due to A_1^*)

$\longrightarrow \mu_\varphi = 5$

$L_p = 12{,}19 \cdot (0{,}5 + 1{,}67 \cdot 0{,}24) = 11{,}0\,m$

$\mu_\delta = 1 + 4\,\frac{3 \cdot 11(150 - 10{,}5)}{150^2} = \underline{1{,}82}$

Fig. 4 Estimation of ductility for y-direction / improved cross section

simplified. The FE-mesh is shown in fig. 1. Using programm ARS/GENF (in PC-version) the bridge was modelled as a space frame with 46 nodal points, 23 beam elements for the superstructure and 28 beam elements for the piers. The cross section properties for the beams and piers were taken from the original model. Further, the moduli of elasticity obtained by global fitting of the original model to the results of dynamic insitu tests were used. As it is assumed that the bridge should be constructed using the cantilevering method, piers 2-4 are designed as twin piers, with deep coupling beams (see fig. 1). Spring-damper devices acting in longitudinal direction are provided in points 1 and 46. Fixed end conditions are assumed in points 5, 12, 16, 23, 27, 34, 38 and 44.

Hence, foundation and soil influence were disregarded in the FE-model.

2.4. Material properties

concrete for piers: B 500, cube strength $f_c = 50$ MPa

$E = 4{,}96 \cdot 10^7\,kN/m^2$ (from model-fitting to test results)

$= 2{,}5\,t/m^3$

concrete for girders: B600, cube strength $f_c = 60$ MPa

$E = 5{,}53 \cdot 10^7\,kN/m^2$ (from model-fitting to test results)

$\gamma = 2{,}5\,t/m^3$

steel: BSt 550 $f_y = 550$ MPa

Material properties are considered to be characteristic values. It is assumed, that an overstrength factor $\gamma^*_{oH} = 1{,}15$ and a material safety factor of $\gamma^*_S = 1{,}15$ cancel each other out, hence steel will really yield at 550 MPa.

2.5. Method of analysis

The bridge was modelled using program ARS/GENF in the PC-version. Then, with module DYNA, 45 eigenfrequencies and modeshapes were calculated. The eigenfrequencies are given in tab. 1. Using the response spectra described in chapter 2.3.1. the modal responses were obtained for modal damping ratios $\xi = 5$ %. The modal responses were combined using the SRSS combination rule.

3. ANALYSIS

3.1 Program of the investigations

Starting from the structural model of Lavant bridge the simplified FE model given in fig. 1 was developed.

Using response spectra, the maximum bending moments, shears and displacements were calculated, assuming excitation in transverse as well as longitudinal direction. More details of the calculations are given in chapter 2. The results are presented in tab. 2. Further, it was assumed that during a severe earthquake the joints between the

coupling beams (elements 117 and 118) and twin pier 3 could faile. Hence a variant of the model without element no 117 and 118 was investigated. The results are also given in tab. 2.

3.2. Results for pier 2 and 3

3.2.1. Estimation of curvature ductility

The maximum bending moment for transverse excitation occured at pier 2 and 4. It was found to be 649 MNm for the elements 105 and 108.

154 bars $\phi 32$ were selected for the tension zone. Yielding of the longitudinal reinforcement will start under the maximum bending moment.

The available curvature ductility of I-shaped shear walls was investigated by Keintzel [1]. Parameters equivalent to [1] were defined in fig. 3. With N_d = $7{,}1.10^7$ N (from permanent load) and $f_{ck} = 50/\gamma_c \sim 30$ MPa ($\gamma_c = 1{,}5$; material safety factor for concrete) the factor n = 0,28 was obtained. It is known from [1] that curvature ductility is mainly influenced by , n and ε_s while the reinforcement ratio has only a minor influence. Hence, with n = 0,28 and $\gamma = 0{,}2$ from [1] the available curvature ductility was found to be $\mu_q = 4$. In a second step, the cross section was improved by adding two stiffening walls (see fig. 4). The stiffening was considered necessary especially to improve the ductile behaviour in longitudinal direction. A curvature ductility of at least $\mu_q = 5$ is estimated for the improved cross section.

The maximum bending moment for longitudinal excitation occured at pier 3 and was found to be 442 MNm for the elements 113 and 116. 370 bars $\phi 32$ were selected for the tension zone. Yielding will start under the maximum bending moment.

With n = 0,24 and $\gamma = 0{,}5$ an available curvature ductility of $\mu_q = 18$ was found from [1].

3.2.2. Estimation of plastic hinge length

Several formulas for the calculation of the plastic hinge length are given in literature [1, 2, 3, 4] and in EC 8/part II. According to the comparison of experimental and theoretical plastic hinge lengths [2, 3, 4] for n < 0,3, l_p is given by:

$$l_p = (0{,}08L + 6d_b)\,(0{,}5 + 1{,}67.n) \qquad (1)$$

with L ... height of the cantilever beam
d_b ... diameter of longitudinal reinforcing bars
n ... normalized normal force

For the improved cross section (fig. 4) a plastic hinge length of $l_p = 11$m was obtained from equ. (1).

3.2.3. Displacement ductility

For a cantilever pier with the height l the displacement ductility μ_s can be estimated using equ. (2), (see also [1]):

$$\mu_s = 1 + (\mu_q - 1)\,\frac{3L_p(L - 0{,}5L_p)}{L^2} \qquad (2)$$

with μ_q... curvature ductility
l_p ... length of the plastic hinge

For transverse excitation, with $\mu_q = 5$ and $l_p = 11$m, from equ. (2) the displacement ductility $\mu_s = 1{,}82$ is obtained (fig. 4). For longitudinal excitation, with $\mu_q = 18$ and $l_p = 11$m, the displacement ductility $\mu_s = 4{,}5$ is obtained.

3.2.4. Discussion of the results

According to EC 8/part II, clause 4.4, a bridge designed after the EC 2 provision will behave as a low ductile structure. Without any additional design or special details a behaviour factor of 1,5 can be assumed.

In our example the structure was designed to start yielding under the given earthquake load (a_{max} = 0,4g).

The displacement ductility was obtained using the classical formulas, which is a highly simplified approach. There is no practical evidence of an adequate ductile behaviour of large piers with large hollow-rectangular cross sections.

The available displacement ductility evaluated in chapter 3.2.3. was within $1{,}82 \le \mu_s \le 4{,}5$. Hence, it is concluded, that a behaviour factor $q = 1{,}5$ is realistic for both directions, providing a maximum earthquake capacity (without severe damages) up to the maximum acceleration of $a_{max} = 0{,}6g$.

For both directions (without areas A_s^* in fig. 4), altogether 524 bars $\phi 32$ (geometrical ratio $\rho_L = 4{,}3$ %) were used for the longitudinal reinforcement.

Transversal reinforcement was designed after ÖNORM B 4200/part 8: links $\phi 14$ with two legs at 100 mm distance.

P-Δ effects can be disregarded because of the relatively small maximum displacement $d_{max} = 0{,}1$m.

4. CONCLUSIONS

Recent investigations of TU-Graz indicate that the ductility-estimation procedure used in chapter 3.2.1 could be applicable.

It was shown, that in principle large bridges could be designed to remain within the elastic range also

under a high maximum acceleration. In our case the bridge will behave elastically up to a maximum acceleration $a_{max} = 0{,}4\ g$, having a maximum earthquake capacity (without severe damages) up to $a_{max} = 0{,}6g$ which is very conservative for the Alpine Region.

According to EC8/part II a bridge designed after EC2 provisions will behave as a low ductile structure. Without any additional design or special detailling a behaviour factor of 1,5 can be assumed.

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