Highway Bridge Pier Capacity Design

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Abstract: - To prevent brittle failure under strong earthquakes, ductile reinforced concrete bridge columns are designed following the capacity design philosophy, which requires evaluating their overstrength to ensure safe plastic hinge formation. This study presents a comparative capacity analysis of RC bridge columns based on the Albanian code, Eurocode 2, and AASHTO LRFD. Ultimate axial force and bending moment capacities are calculated analytically using selected overstrength parameters, and representative case studies illustrate how normative differences influence column design and seismic performance.

Key-Words: - bridge, pier bridges, capacity design, Eurocode 2, Aashto, moment curvature section

Received: April 19, 2025. Revised: July 11, 2025. Accepted: August 18, 2025. Published: December 31, 2025.

1 Introduction

This study evaluates and compares pier capacity using a section-based capacity assessment approach for bridges designed in accordance with AASHTO, Eurocode, and the Albanian code. Bridges are critical components of transportation networks, and their failure can result in severe structural, economic, and social consequences [1]. The evaluation of existing bridges provides essential information to support decision-making processes related to bridge maintenance, rehabilitation, and load management [2], [3], [4], [5], [6], [7], [8].

The pier assessment is conducted on a two-span, simply supported longitudinal bridge section, as illustrated in Figure 1. Each span has a length of 15 m, and the pier is supported by piles with a rectangular cross-section measuring 350 mm × 220 mm.

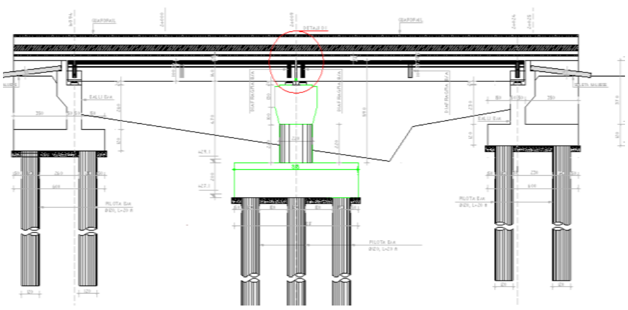

Fig. 1: Longitudinal bridge section [created by the author]

2 AASHTO pier capacity assessment

AASHTO provides two fundamental design processes for reinforced concrete: load factor and service loads. Column design must be based on capacity, according to the standards provided under service loads design. According to the code, service loads and moments cannot be greater than 35% of the column section's capacity when φ=1. As the axial load strength drops from 0.1f'cAg or Pb, whichever is smaller, to pure bending with no axial load, the AASHTO load factor design parameters for the transition from the column φ value to the flexural value of φ=0.9 for pure bending are as follows [9], [10], and [11]. AASHTO LRFD relies on the relative effective stiffness of the piers.

According to the AASHTO design guidelines, biaxial bending and direct load should be examined using the following approximations or, alternatively, stress and strain compatibility as shown on formulas 1, 2, and 3:

$$\frac{1}{P_{nxy}} = \frac{1}{P_{nx}} + \frac{1}{P_{ny}} - \frac{1}{P_0} \qquad (1)$$

where,

$$Pu > 0.10f'cAg \qquad (2)$$

$$\frac{M_{ux}}{\phi M_x} + \frac{M_{uy}}{\phi M_y} \leq 1 \qquad (3)$$

where

P_U- ultimate axial load

M_{ux} - ultimate moment (x-x)

M_{uy} - ultimate moment (y-y)

The basic criteria for load factor design involve factoring the design loads and then comparing the results with the ultimate capacity.

$$\gamma[\beta_D(DL) + \beta_{LL}(L + I) \le P_u \tag{4}$$

γ –load factor

β- coefficient

β_D=0.75 when checking members for minimum axial load and maximum moment or maximum eccentricity for column design.

β_D =1 when checking members for maximum axial load and minimum moment for column design.

3 Eurocode 2 (EN 1992-1-1) Procedure for RC Pier Capacity

In Eurocode 2, the capacity of a reinforced concrete pier is evaluated at the Ultimate Limit State (ULS) by determining the design axial force–bending moment resistance (N_{Rd}, M_{Rd}). The procedure is based on strain compatibility, equilibrium of internal forces, and design material strengths. Eurocode 2 uses design values obtained by applying partial safety factors [12], [13], [14], [15], [16]:

For concrete;

$$f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_c} \tag{5}$$

where:

f_{ck} = characteristic compressive strength

γ_c=1.50

α_{cc}=1.0 (recommended value)

For Reinforcing steel;

$$f_{yd} = \frac{f_{yk}}{\gamma_s} \tag{6}$$

where:

f_{yk}= characteristic yield strength;

γs=1.15.

Internal Force Equilibrium

Axial force equilibrium:

$$N_{Rd}=Cc+Cs-Ts \tag{7}$$

Moment equilibrium (about the section centroid):

$$M_{Rd}=\sum Fi\cdot zi \tag{8}$$

where: Cc= concrete compressive force

Cs,Ts= compressive and tensile steel forces

zi= internal lever arms.

4 Albanian design code

Design of reinforced concrete section on ultimate limit state according Albanian code in case of the maximum eccentricity for column design [8]. The entire limit state is evaluated using a ϕ -based resistance format, similar to AASHTO, but with nationally defined material parameters as shown in Figure 2. The design capacities Pu and Mu are obtained by applying the prescribed reduction factors to the nominal section resistance.

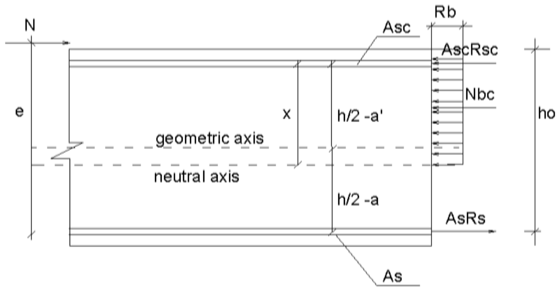

Fig. 2: Intire limit state of R/F section [17], [18]

The following equation shows the section balance condition:

$$N\cdot e \le b\cdot x\cdot R_b(ho-0.5x)+R_{sc}\cdot A_{sc}(ho-a') \tag{9}$$

$$\text{and } N\le b\cdot x\cdot R_b+A_{sc}\cdot R_{sc}-As\cdot Rs \tag{10}$$

The moment due to reinforcing the compression block, Asc: Msc = Rsc·Asc(ho-a')

The concrete moment on the compression block:

$$M_{bc} = N\cdot e-M_{sc}=Ao,b\cdot b\cdot ho^{2}\cdot Rb \tag{11}$$

The Albanian code defines coefficients for combining loads (permanent, temporary, seismic) when determining design action.

Table 1 summarizes design rules according to KTP-N.2-89 [17], [18].

Table 1: Albanian code KTP-N.2-89 load type combination coefficient [17], [18]

Load Type	Coefficient values on group combination
Dead Loads	0.9
Live load with long temporary action	0.8
Live load with short temporary action	0.8
Seismic loads	1

5 Case Study of Bridge Capacity Pier

Figure 1 illustrates a bridge composed of two continuous simply supported concrete girder spans. As measuring 15 m. The superstructure consists of six prefabricated beams supporting a deck with a width of 9 m. Intermediate support is provided by single-column bents, while abutments are located at both ends of the bridge. The reinforced concrete (RC) bridge columns have a rectangular cross-section of 350 × 220 mm and are reinforced with steel having a yield strength f_y of 420 MPa. The characteristic compressive strength of the concrete is f_{ck}=30 MPa. The material properties of Concrete class C30/37 are summarized in Table 2, [12], [15], [19].

Table 2: Concrete C30/37 Properties- EN 206/ Eurocode 2 [12]

Concrete C30/37 Properties	
Characteristic Compressive Strength f_{ck}	f_{ck}=30Mpa
Strain at peak stress ε_{C1}	ε_{CU1}=0.0021
Ultimate compressive strain ε_{CU1}	ε_{CU1}=0.0035
Steel Reinforcement Properties	
Ultimate tensile strength, f_u	$f_u \geq$ 620MPa
Yield Stress	ε_S=0.005
Yield strength	fy=420 Mpa
Maximum of elasticity E_s	Es=200 GPa

5.1 Moment and Force Capacity Assessment Procedure

The structural capacity of the pier is verified in accordance with the AASHTO LRFD Bridge Design Specifications using a nonlinear cross-sectional analysis approach. The verification is performed at the Strength Limit State, considering combined axial load and biaxial bending, which governs the pier response.

The pier section is subjected to combined axial force and bending about both principal axes. To capture the governing behavior under biaxial bending, the axial force–moment interaction surface is developed by rotating the neutral axis through all possible orientations and computing the corresponding section capacities. The resulting interaction surface is used to verify that the factored design demands lie within the factored resistance envelope.

It is assumed that the governing response occurs below the balanced failure point of the interaction surface, consistent with reinforced concrete column behavior under combined loading.

At the balanced strain condition, defined by a concrete compressive strain of 0.003 and a reinforcing steel strain of 0.00207, the section capacity is calculated based on the assumed linear strain distribution. The strain in each reinforcing bar is determined from its geometric location relative to the neutral axis, with reference to the section centroid after the reinforcement layout has been defined.

The stress in the reinforcing steel is computed by multiplying the calculated strain by the modulus of elasticity, Es = 200 GPa, and is limited to the specified yield strength of 420 MPa. The axial force in each reinforcing bar is obtained by multiplying the steel stress by the corresponding bar area. The moment contribution of each bar is then calculated by multiplying the bar force by its lever arm measured from the section centroid.

The concrete compressive force is evaluated using an equivalent rectangular stress block. The depth of the compression block is defined using the coefficient β_1 , taken as 0.85 in accordance with AASHTO, 0.80 according to Eurocode, and 1.00 as specified by the Albanian code. The resultant concrete force is assumed to act at the centroid of the compression block, and its moment contribution is calculated accordingly.

The axial force–moment capacity values (P–M) are also determined for a tension-controlled strain condition, in which the reinforcing steel strain reaches 0.005 while the concrete compressive strain remains equal to 0.003. The same computational procedure is applied to generate the corresponding capacity points, which together define the biaxial interaction surface of the section.

The analysis demonstrates that the pier possesses adequate axial and flexural strength under combined axial load and biaxial bending at the Strength Limit State. The factored demand-to-capacity comparison confirms compliance with AASHTO LRFD requirements for reinforced concrete bridge piers.

Table 3 presents the factored axial force and biaxial bending moment capacities of the pier section as determined using the capacity reduction provisions of AASHTO LRFD, Eurocode, and the Albanian code. The results indicate systematic differences in predicted capacities arising from the distinct resistance factors and stress block parameters adopted by each design standard. In general, the AASHTO LRFD provisions yield comparatively conservative capacity estimates, reflecting lower resistance factors and a reduced equivalent concrete compression block depth. The Eurocode-based capacities are moderately higher, attributable to differences in partial safety factors and the adopted value of the concrete stress block coefficient. The Albanian code produces the highest capacity values, primarily due to the use of a larger effective compression block and less conservative reduction factors. Despite these quantitative differences, all three codes exhibit consistent trends in the relative contribution of axial force and biaxial bending moments, and the governing failure mode remains controlled by combined axial–flexural behavior. The comparison demonstrates that while code-specific assumptions significantly influence the numerical capacity values, the overall structural response and interaction behavior of the pier section remain qualitatively similar across the three design frameworks.

Table 3: Factored axial force and bending moment capacities corresponding to the capacity reduction provisions of AASHTO LRFD, Eurocode, and the Albanian code. [created by the authors]

Force and moments capacity according to AASHTO			**Force and Moments capacity according to Eurocode 8**		**Force and Moments capacity according Albanian Code**	
Φ factor	P ton	M ton m)	P ton	M ton m	P ton	M ton m
Φ=1	4675	6291	4115	5102	6460	6416
Φ=0.9	4207	5662				
Φ=0.7	3272	4403				

Figure 3 and Figure 4 illustrate the calculated axial force–bending moment capacity values, emphasizing the governing scenarios identified from the ultimate load and moment results.

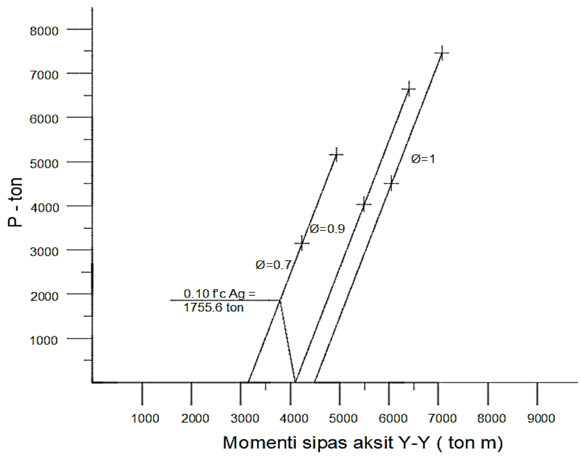

Fig. 3: Pier section Analysis: Axis Y-Y [created by the authors]

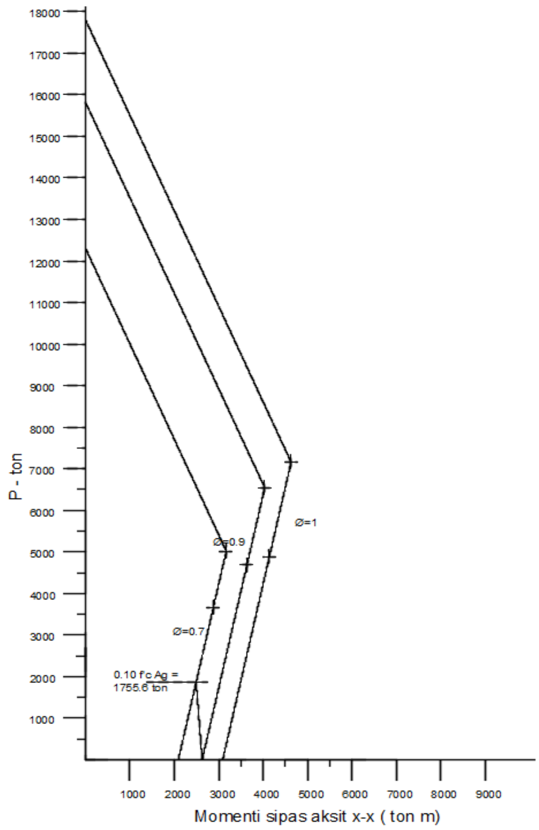

Fig. 4: Pier Capacity: Axis X-X [created by the authors]

Table 4 presents a comparison of reinforced concrete pier axial–flexural capacity as determined in accordance with Eurocode 2 (EC2), AASHTO LRFD Bridge Design Specifications, and the Albanian design code.

Table 4: EC2, AASHTO LRFD, and Albanian Code – Pier Capacity Comparison [created by the authors]

Aspect	**Eurocode 2 (EC2)**	**AASHTO LRFD**	**Albanian Code**
Safety format	Partial safety factors	Strength reduction factor	Strength reduction factor
Capacity terms	N_{Rd}, M_{Rd}	ϕP_n, ϕM_n	P_u, M_u
Limit state	Ultimate Limit State (ULS)	Strength Limit State	Ultimate Limit State
Ductility control	Implicit	Explicit (strain-based	Limited / implicit

6 Conclusions

In accordance with AASHTO LRFD Bridge Design Specifications, Eurocode 2 (EN 1992-1-1), and the Albanian technical design codes, this study presents a case-study application for evaluating the axial–flexural resistance of a reinforced concrete (RC) bridge column cross-section. Section capacity is first evaluated at the balanced strain condition, defined by a reinforcing steel strain $\varepsilon s=0.00207$ and an ultimate concrete compressive strain $\varepsilon_{cu}=0.003$.

Additional section capacity analyses are carried out for a tension-controlled (ductile) limit state, in which the reinforcing steel strain reaches $\varepsilon s=0.005$ while the concrete compressive strain remains $\varepsilon cu =0.003$, consistent with AASHTO LRFD strain limits.

For each design standard, the nominal axial resistance P_n and nominal bending resistance Mn are obtained by enforcing equilibrium of internal forces and moments. The corresponding factored design resistances, Pu and Mu, are then determined by applying the appropriate strength (resistance) reduction factors ϕ for AASHTO LRFD and Albanian codes, and partial safety factors γc and γs for EC2. Capacity results are reported at multiple reduction-factor levels to allow direct comparison between the standards.

The results show that section analysis based on Eurocode 2 material parameters and partial safety factors yields lower axial–moment resistance compared to both AASHTO LRFD and Albanian code provisions. Conversely, the Albanian code produces higher section capacity values than those obtained using Eurocode 2 and AASHTO LRFD methodologies.

Based on these findings, the study concludes that the development of updated national Albanian design parameters, harmonized with Eurocode 2 and European safety formats, is necessary to ensure consistency and reliability in bridge column design.

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Contribution of Individual Authors to the Creation of a Scientific Article (Ghostwriting Policy)

The sole author contributed to the present research, at all stages from the formulation of the problem to the final findings and solution.

Sources of Funding for Research Presented in a Scientific Article or Scientific Article Itself

No funding was received for conducting this study.

Conflict of Interest

The authors have no conflicts of interest to declare that are relevant to the content of this article.

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