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Capacity Design in bridges

Cost analysis of a project

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Abstract

Following the new European Regulations and the new earthquake safety philosophy, structural design will undergo regulatory changes in ductility criterions. This is due mostly to the concept of Capacity Design and its mandatory application for medium or high seismic zones, like Portugal.

This work will focus on the costs of this new rules change, comparing possible alternatives to the project with the introduction of new regulations, which mainly reflect the columns and foundations, as these are structural elements of a bridge that normally resist horizontal actions. Also there will be a comparative analysis with previous regulation, to evaluate whether this rules change brought significant cost changes. This will be done with case studies based on a real project.

Keywords:

Bridge, Capacity Design; Structure; regulations; Eurocode 8; Costs

Introduction

Following the new European Regulations and the new earthquake safety philosophy, structural design will undergo regulatory changes in ductility criterions. This is due mostly to the concept of Capacity Design and its mandatory application for medium or high seismic zones.

The Capacity Design is a philosophy for the design of ductile structures subjected to strong earthquakes and it was firstly implemented in New Zealand. This philosophy advocates a hierarchy in failure modes, giving priority to the more ductile (that allow larges deformations and more energy dissipation) and avoiding the occurrence of brittle failure model. This leads to endorsement of plastic hinges in the critical zones of structural elements and avoids brittle failure modes such as shear (Paulay & Priestley, 1992).

The new European regulation of the seismic actions on bridges (*EN 1998-2*) defines in its Article § 4.1.6, in particular at Table 4.1, several plafond for the behaviour coefficient to choose during the design. The behaviour coefficient depends on the type of structure, and whose choice will have implications in terms of initial project cost.

The implementation of a seismic resistance structure also involves an increase in the dimensions of resistant elements, the amount of reinforcing steel and possibly also the cost of labour resulting from the execution of some complex reinforcing details.

With regard to the cost estimation of the damage, one must keep in mind that the more ductile the structure is, the greater safety and lower the repair costs will be, for seismic actions higher the seismic project. However, in the event of an earthquake equal to or less than the seismic design, repair costs will be higher (*coord. Lopes, 2009*). This can be summarized by the following figure, where the vertical axis represents the repair costs and the horizontal axis represents the intensity of the earthquake, depending on a scalar value (γ) which is multiplied by the earthquake design of EC8.

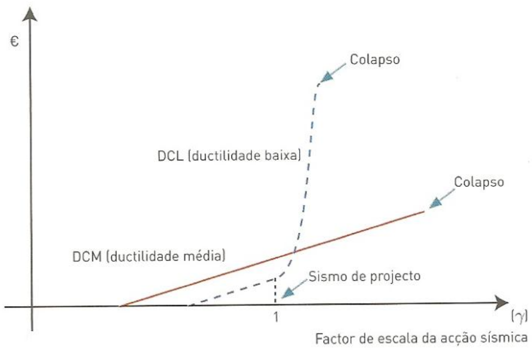

Figure 1 - Qualitative variation of repair costs (*coord. Lopes, 2009*)

In order to estimate the costs of implementing the principle of Capacity Design in the design of structures, two case studies were analysed. Those case studies were based on an existing project. The cases differ on their foundations: in the first one footing foundation was adopted, whereas deep foundations were adopted on the second case.

The case study

The superstructure consists on a prestressed concrete ribbed deck slab. Three leaks ribs were considered at 1/5 span, thus reducing the self-weight. The leaks are 0.8 meters in diameter. The slab is supported by short concrete columns.

The bridge consists of 14 spans, with lengths described in the table below:

Table 1 - Length of the spans

id_{span}	1	2	3	4	5	6	7	8	9	10	11	12	13	14
L_{span} [m]	21,40	28,00	35,00	35,00	35,00	35,00	35,00	35,00	35,00	35,00	35,00	35,00	35,00	25,35

The total deck length is 459.75m. Two 1,60m diameter columns support the deck at each alignment, as it can be seen in Figure 2 and Figure 3.

The following materials were used:

- Concrete on deck: C35/45.
- Concrete on columns: C30/37
- Reinforcing steel on columns: A400
- Concrete on foundations: C20/25

The columns-deck joint uses elastomeric bearings. The support devices consist in a locking drive in the transversal direction and in an elastic drive on the longitudinal direction. This solution minimizes the effects of temperature variations and shrinkage. The bearers are 900 mm in diameter and 250 mm high. The foundation soil consists on a Miocene subtract that does not change for the 2 case studies.

The footing foundations for the first case study follow the geometrical configuration on the picture below:

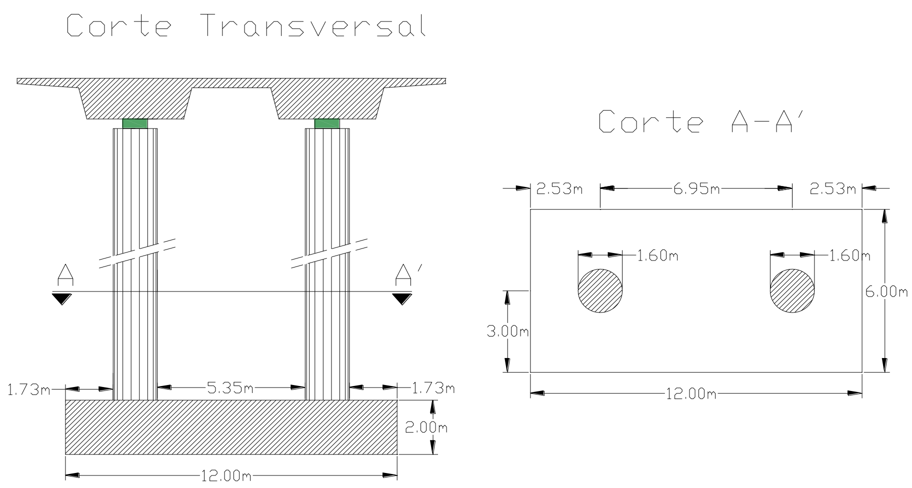

Figure 2 - Footing foundation

For the second case of analysis with piles foundations were defined the following geometrical configuration:

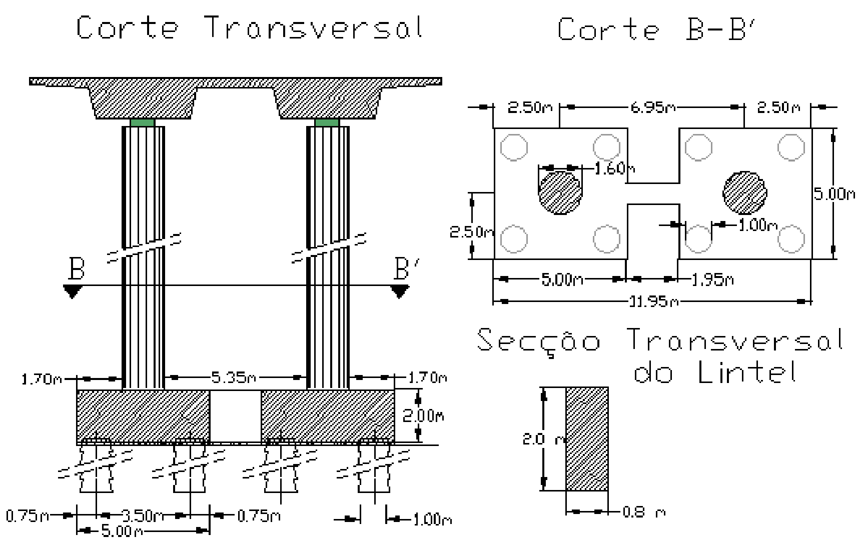

Figure 3 - Foundation with piles

This foundation is composed of concrete bored piles linked by a pile cap, also in reinforced concrete. The set of columns + pile cap + piles are connected together monolithically. Each pile cap allows connection of a pillar to four foundation piles.

In footing foundations, the allowable stress of the soil is 0,5 MPa, while in the piles foundations was considered a tip resistance of 3500 kN.

In each alignment corresponds to the following columns height of the columns and the piles:

Table 2 - Height of the columns

Column	h_{column} (m)
P1	10,06
P2	11,01
P3	11,10
P4	11,20
P5	9,26
P6	11,12
P7	9,59
P8	9,06
P9	11,09
P10	13,12
P11	14,21
P12	14,26
P13	14,40

Table 3 - Height of piles

Column	h_{column} (m)
P1	10,50
P2	10,00
P3	11,50
P4	13,00
P5	16,50
P6	16,00
P7	15,00
P8	15,00
P9	12,00
P10	9,00
P11	8,50
P12	8,50
P13	8,50

Modelation of structure

The determination of design efforts was performed using a linear analysis by response spectra introduced into a model suitable for the purpose. Thus, for the analysis of the effects of the seismic action was created a model in *SAP2000* Software. It was considered frames elements to simulate the behavior of the columns and also the deck.

To modeling the foundations of the columns it was considered that the columns are fixed at their base, thus ignoring the effect of the deformability of the foundations.

The abutments were modeled by mobile pinned supports that constrain only the vertical and the horizontal axis that is transverse to the development of the deck.

The structural bearings were simulated using bar elements with the suitable geometrical and mechanical characteristics

Costs of design with *EN 1998*

With the purpose of making a cost analysis of alternatives to design the bridges, a case study analysis was made in accordance with a ductile design defined by the new *European regulations EN 1998-2*, which defined a high behavior factor (q = 3,5) which is a higher value than that one used in the former rules.

It was made a limited ductility design in the structure, i.e. without exploiting the ductility of the reinforced concrete (EN 1998-2 defines a behavior factor q ≤ 1.5), for two cases of study: $q = 1.5$ (ductile) and $q = 1.0$ (elastic).

It was also necessary to change the dimensions of the structure. In the ductile design (q = 1.5), the columns have a diameter of 2.0 meters and the piles have 1.1 meters of diameter. The footings have a dimension of 13,0x7,0x2,0 meters and the piles cap have 5,5x5,5x2,0 meters, with the distance of 4.0 meters between the piles axis. With elastic design (q = 1,0), the columns have a diameter of 2.3 meters of diameter and the piles have 1.2 meters of diameter. The footings have a dimension of 16.0 x8.0x2.0 meters, and only one massive pile cap with 13.0 x8.0x2.0 meters with 8 piles.

By quantities list was possible to verify the following decrease in the amount of steel, while making ductile design:

Table 4 - Quantities analysis of steel for the ductile design and limited ductility design [kg]

		Quantities of steel [kg]		
		Columns + Foundations		
Structure with footing	*Limited ductile (q=1,5)*	1.307.058,60	*Variation – Δ*	
	Ductile (q=3,5)	755.084,38	-551.974,22	-42,23%
Structure with piles	*Limited ductile (q=1,5)*	1.902.712,71	*Variation – Δ*	
	Ductile (q=3,5)	1.034.760,74	-867.951,97	-45,62%

Table 4 reveals that the saving of the total amount of steel in the set *foundation+columns* between limited ductile design and ductile design achieved is very significant in both cases of foundation, and is even more significant in relation to foundation with piles (the structure savings of the with footing foundation is about 42.23% of the total steel of limited ductility design and for foundation with piles the decrease is about 45.62% of the total steel of limited ductility design). With these data it is observable that the new rules ensure cost savings, through reducing the amount of steel, for both cases of foundation. In terms of absolute values, the savings made in foundation with piles is a higher value than those obtained in the footing foundation (savings of about 552 tons in the case of footing foundations, and approximately 868 tons in deep foundations case).

In the case of structures with footing foundations, the decrease in quantity of steel is distributed as follows:

Table 5 - Differential amount of steel analysis, Δ, between the two types of sizing *EN 1998-2*, in structure with footing foundation [kg]

	Columns	*Foot. found.*
Amount of steel [kg]	-314.425,48	-237.548,74
%	56,96%	43,04%

The quantity of steel reduction, when moving from a limited ductility design to a ductile design, occurs between the two types of structural elements considered (columns and footing foundation).

In the structures with piles foundation, the variation between the amount of steel in ductile design (q = 3,5) and limited ductility design (q = 1.5) is:

Table 6 - Differential analysis of the steel, Δ, between the two types of sizing EN 1998-2 in structure with piles foundation [kg]

	Columns	*Pile Caps*	*Piles*
Amount of steel [kg]	-314.425,48	-202.336,23	-351.190,26
%	36,23%	23,31%	40,46%

The foundations correspond to considerable savings of steel, particularly in the case of piles. There is an increase in steel saving with increasing dimension of the structure.

In the following table it is clearly visible a considerable reduction of approximately half of each column's steel content while going from limited ductile design to ductile design:

Table 7 - Difference of steel content analysis by column between the ductile design and limited ductility [kg]

Column	h_{pile} (m)	Limited ductile q=1,5	Ductile q=3,5	Δ	Δ (%)
P1	10,06	22.893,79	11.893,53	-11.000,26	-48,0%
P2	11,01	24.207,55	12.479,82	-11.727,73	-48,4%
P3	11,10	24.311,99	12.523,94	-11.788,05	-48,5%
P4	11,20	24.458,23	12.597,06	-11.861,17	-48,5%
P5	9,26	21.814,45	11.404,89	-10.409,55	-47,7%
P6	11,12	24.335,21	12.533,74	-11.801,47	-48,5%
P7	9,59	22.257,78	11.590,79	-10.666,99	-47,9%
P8	9,06	21.552,16	11.282,74	-10.269,42	-47,6%
P9	11,09	24.300,38	12.519,03	-11.781,36	-48,5%
P10	13,12	27.078,79	13.755,32	-13.323,47	-49,2%
P11	14,21	28.555,05	14.434,33	-14.120,71	-49,5%
P12	14,26	28.613,07	14.458,85	-14.154,22	-49,5%
P13	14,40	28.859,93	14.551,60	-14.308,33	-49,6%
			Total	**-157.212,74**	

On the considered foundations, although the difference between efforts on both designs is attenuated by the overstrenght factor ductile design, γ_0, there is savings in steel in the same order of magnitude as observed in the columns. On footing foundation case, steel saving is:

Table 8 - Analysis of the amount of steel on the footing foundation of an element ductile and limited ductility [kg]

Ductile Limited q=1,5	Ductile q=3,5	Δ	Δ (%)
25.406,99	13.680,74	-11.726,25	-46,15%

In pile foundations there is a smaller saving in the quantity of steel, with values of about 43-44%:

Table 9 - Amount of steel analysis per alignment of columns with pile foundations in a ductile element and a limited ductile element [kg]

Column	h_{pile} (m)	Ductile Limited q=1,5	Ductile q=3,5	Δ	Δ (%)
P1	10,50	47.011,19	26.385,81	-20.625,38	-43,9%
P2	10,00	46.404,06	26.089,05	-20.315,01	-43,8%
P3	11,50	48.225,44	26.979,48	-21.245,97	-44,1%
P4	13,00	50.046,69	27.869,91	-22.176,78	-44,3%
P5	16,50	54.296,30	29.947,60	-24.348,70	-44,8%
P6	16,00	53.689,04	29.650,76	-24.038,27	-44,8%
P7	15,00	52.475,06	29.057,17	-23.417,89	-44,6%
P8	15,00	52.475,06	29.057,17	-23.417,89	-44,6%
P9	12,00	48.832,43	27.276,31	-21.556,12	-44,1%
P10	9,00	44.412,55	25.122,70	-19.289,85	-43,4%
P11	8,50	43.416,73	24.639,60	-18.777,13	-43,2%
P12	8,50	43.416,73	24.639,60	-18.777,13	-43,2%
P13	8,50	43.416,73	24.639,60	-18.777,13	-43,2%
			Total	**-276.763,24**	

In piles cap the savings of steel was 34.61%.

In each pile there was a saving of steel was slightly greater than 50.0% of the respective steel:

Table 10 - Amount of steel analysis for the piles [kg]

Pilar	h_{pile} (m)	Dutile limited q=1,5	Ductile q=3,5	Δ	Δ (%)
P1	10,50	24.525,67	11.682,45	-12.843,22	-52,4%
P2	10,00	23.918,54	11.385,69	-12.532,85	-52,4%
P3	11,50	25.739,93	12.276,12	-13.463,80	-52,3%
P4	13,00	27.561,17	13.166,55	-14.394,62	-52,2%
P5	16,50	31.810,79	15.244,25	-16.566,54	-52,1%
P6	16,00	31.203,52	14.947,41	-16.256,11	-52,1%
P7	15,00	29.989,54	14.353,82	-15.635,73	-52,1%
P8	15,00	29.989,54	14.353,82	-15.635,73	-52,1%
P9	12,00	26.346,91	12.572,96	-13.773,96	-52,3%
P10	9,00	21.927,03	10.419,34	-11.507,69	-52,5%
P11	8,50	20.931,21	9.936,25	-10.994,96	-52,5%
P12	8,50	20.931,21	9.936,25	-10.994,96	-52,5%
P13	8,50	20.931,21	9.936,25	-10.994,96	-52,5%
			Total	**- 175.595,13**	

As the geometric characteristics of the structures are different from each other, there is also a difference in cost of the respective formwork, concrete and excavation quantities considered. With the purpose of assess the costs of each case and quantify the savings made with ductile design, several map measurements were made:

Table 11 - Cost analysis of two types of design recommended in EN 1998-2 [€]

		Columns + Foundations		
Structure with footing	*Limited ductile (q=1,5)*	2.601.161,66	*Variation - Δ*	
	ductile (q=3,5)	1.745.631,52	-855.530,14	-32,89%
Structure with piles	*Limited ductile (q=1,5)*	3.591.267,01	*Variation - Δ*	
	ductile (q=3,5)	2.452.900,79	-1.138.366,22	-31,70%

Analyzing only the cost of the set *columns + foundations*, it can be seen that the savings achieved with the design ductile corresponds to 32.89% of the total cost of the columns+ footing foundations case study with limited ductile design. In pile foundations that portion of saving decreases to approximately 31.70% of its total cost of limited ductile. Quantifying in absolute values, there is a saving in the order of 856,000€'s, while that in the case study of pile foundations this saving is about 1,138,000€'s.

In both cases, we speak of considerable cost, with a structure that will cost a few million euros, thus making the design ductile very competitive for designers.

We must also say that the fact that we did not consider the difference in costs of structural bearings, including the guides that have to resist shear forces, and that if they chose different solutions, the solution ductile become even more competitive (less shear in the guides of structural bearings).

Regarding the of elastic design of structures ($q = 1{,}0$) analyzing the respective maps measurements allowed to quantify the differences over the ductile design ($q = 3{,}5$):

Table 12 - Analysis of costs between ductile and elastic design (EN 1998-2) [€]

		Columns + Foundations		
Structure with footing	*Limited ductile (q=1,5)*	3.581.292,05	*Variation - Δ*	
	ductile (q=3,5)	1.745.631,52	-1.835.660,53	-51,26%
Structure with piles	*Limited ductile (q=1,5)*	4.594.727,93	*Variation - Δ*	
	ductile (q=3,5)	2.452.900,79	-2.141.827,14	-46,61%

Analyzing the costs of all their foundations + columns, savings verified by design respecting the principles of CD corresponds to a very high value, approximately 51.26% of the total cost of footing foundations + columns sized through design not ductile. In the case of foundations with piles this portion rises to a saving of 46.61% of their total cost.

There is a saving of around the value of 2 M€'s in both types of foundation, which is a very considerable value and that categorically demonstrates how little is recommended to design without leverage the capabilities of energy dissipation of the elements of resistant reinforced concrete, i.e. without exploiting the ductility of concrete.

With the values obtained for initial costs, it is clear that the new European regulation aims to make economically appealing the ductile design, which will certainly be a considerable incentive for Portuguese designers adopt the new philosophy of design with the Capacity Design. The new regulation defines a small range of values in the design not ductile, $q\epsilon[1{,}0; 1{,}5]$ giving little margin for the designer, and that from what we saw in this study, is economically uncompetitive compared to ductile design.

Comparison with the Portuguese regulation (RSA)

Comparing with the previous design ductile regulation applicable in Portugal, was obtained following difference of quantity of steel:

Table 13 - Analysis of the amounts of ductile steel obtained by design (EN 1998) and improved ductility (RSA):

		Columns + Foundations		
Structure with footing	*RSA*	517.893,87	*Variation - Δ*	
	EC8	755.084,38	237.190,51	45,80%
Structure with piles	*RSA*	865.899,15	*Variation - Δ*	
	EC8	1.034.760,74	168.861,59	19,50%

It is verified that the difference of the steel resulting in the set columns + foundation between the ductile design of both regulations is significant for both cases of foundation, and even more significant in relative terms in the case of footing foundations. In terms of absolute numbers, the increase of quantity of steel is about 237 tonnes in structures with footing foundations and about 169 tonnes in structure with piles foundations.

Making a cost analysis, the following values were obtained:

Table 14 - Cost analysis of ductile design advocated by the old and current regulation [€]

		Columns + Foundations		
Structure with footing	*RSA ($\eta = 3{,}0$)*	1.492.043,76	*Variation - Δ*	
	EC8 ($q = 3{,}5$)	1.745.631,52	253.587,76	17,00%
Structure with piles	*RSA ($\eta = 3{,}0$)*	2.240.552,10	*Variation - Δ*	
	EC8 ($q = 3{,}5$)	2.452.900,79	212.348,69	9,48%

There is a small increase in costs in both situations of foundation. In footing foundations is observed an increase in costs of 253,000 €'s, and piles foundations in this increase is slightly higher, around 212000 €'s.

The introduction of the Capacity Design in the design of bridges in Portugal will require increased costs. However, it certain that they will not represent significant initial cost of the bridge. On the long-term, and for actions to lower than the design seism, repair costs of plastic hinges are not much higher in the case of European regulation because the chosen behavior factor is not much higher than the chosen for the improved ductility design of the RSA. Besides the control of the local energy dissipation hinges can prevent future repairs in places not easily accessible (eg in foundations), cannot be said at the outset of the regulations which have lower repair costs. For the situation of seismic action above the design the advantage is evident to European regulations, because the fragile breakage is more unlikely than in the case of structures dimensioned with RSA. The brittle collapse as those associated with shear ruptures involve very lengthy and costly repairs, as they may lead to loss of the ability to support vertical loads. Besides the repair part itself be very deep because it may require the restoration of the damaged column, ie, its demolition and reconstruction (*coord. Lopes, 2009*). Furthermore, the hinges of the pillars have the advantage that their repair itself be simple and inexpensive, generally not associated with a loss of vertical load capacity (enabling the functioning of the bridge in the aftermath of earthquake), and dispense the use of temporary structures for their repair (*coord. Lopes, 2009*).

The increase in the value of the maximum behavior factor (moved from η = 3.0 to a maximum coefficient of q = 3.5) does not make the new design more competitive in their initial costs, but only mitigates the higher cost associated with the introduction of the Design Capacity Principals.

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