DETERMINATION OF THE OVERSTRENGTH FACTORS R FOR THE CONSTRUCTION OF SEISMIC DESIGN SPECTRA IN BRIDGES

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Abstract

After the earthquakes of September 2017 in Mexico, is necessary to have less vulnerable earthquake resistant structures; so, regarding vehicular bridges, they must be design them with a higher level of safety, which implies proposing seismic design actions representative of the earthquakes that have occurred in recent years, so, to generate better and more reliable designs, it is essential to obtain more precise seismic design spectra that cover a higher level of uncertainties.

Hence, in the present work an evaluation of the overstrength factors R is carried out, for the construction of seismic design spectra in bridges designed according to the criteria and requirements established by the Manual of civil structures design of the (CFE-2015) [1], as well as other regulations, such as the AASHTO code [2] and the Caltrans recommendations [3].

The aim of this research is to evaluate and refining the overstrength R factors, as currently, today there is uncertainty about the values given by the Manual (CFE-2015), bringing with it the uncertainty of the real behavior of the bridges before a seismic event.

To reach this, the method used in this work, is the nonlinear method proposed by the CFE-2015 manual, also known as "progressive plasticization or pushover", which consider once they have exceeded their elastic capacity, they provide a more realistic measure of the behavior when it is required to estimate the demands on the level of response close to the collapse.

The overstrength values are obtained for different concrete bridge structures, which will be used in the analysis and design, for the high seismic regions of the country, through numerical simulations, for different scenarios with the help of analysis programs that consider non-linear behavior.

Finally, it is observed that the configuration of the damage depends on the location and distribution of the plastic joints that are presented in the structure, which are a function, of the excitation, such as: frequencies and the origin of the earthquakes, making it incursion quickly in the inelastic or non-linear interval. Therefore, it is observed that the overstrength factors R, proposed by the CFE-2015 [1] manual, compared with the values obtained from the non-linear analyses, shows some disagreement with the numerical values obtained in these study, with a variation among less 5% to the greater than 15%, approximately.

Keywords: Seismic design spectra; seismic design of bridges; concrete and steel bridges; overstrength factors R.

1. Introduction

In this research, an evaluation of the overstrength factors R is carried out, for the construction of seismic design spectra in bridges designed according to the criteria and requirements established in by seismic manual CFE-2015, as well as other regulations such as: the AASHTO code and the Caltrans [1, 2, 3].
Based on the seismic design recommendations of common bridges, the basic philosophy is to avoid collapse during severe earthquakes, for which two alternative approaches used in the design are proposed.

• The first is a conventional force-based approach where the adjustment factor Q for the assessment of ductility and risk, or the modifying factor of the response R, is applied to the elastic forces obtained from an analysis of spectra of response or equivalent static analysis.

• The second approach is a more recent approach based on displacement where displacement is an important consideration in the design.

In the United States, prior to the San Fernando earthquake of 1971, the seismic design of road bridges was partially based on lateral force requirements for buildings. After the Loma Prieta earthquake in 1989, the design of bridges has faced three essential challenges such as:

• It must be ensured that the seismic risks posed by new construction are acceptable.
• Identify and correct unacceptable conditions of seismic safety in existing structures.
• Develop and implement a quick, effective and economical response mechanism to recover structural integrity after earthquakes (*resilient criteria*).

1.1 Seismic loads

Based on the above, this research is focus to determine the overstrength R factor, for seismic design of concrete bridges by use of the spectra of design according to the requirements set in the seismic design manual CFE-2015 [1], given that is the regulation used to determine the seismic forces for the all country. Thus, for this research this manual was used for the seismic design of concrete bridges.

Analysis Methods

Based on the seismic design recommendations for bridges [2, 4, 3, 6], different analysis methods were used: for the dimensioning of structural elements under different load conditions, and for seismic analysis and design: a. the static seismic method, b. the spectral modal method, c. the "push-over" method and d. the time history method.

1.2 Overstrength **R**

The overstrength effect obtained when designing the concrete elements is due to the greater resistance that the reinforcing steel has because of the hardening by deformation and the value of the reinforcement of creep. Additionally, the concrete has a greater resistance than the one used in the design, due to the confinement, the increase of the resistance with the age and the effect of the application rate of the dynamic loads. When considering these effects, the overstrength can reach values greater than 50% of the design resistance [5], so that the manual CFE-2015 [1] recommend a factor R = 1.5, Caltrans [3], also proposes to reduce the spectrum of agreement to table 1.

1.2 Ductility factor

The objective of seismic design is to ensure that all structural components have sufficient strength and develop a ductility, appropriate to avoid collapse, thus, a limit state where additional deformation could make that a bridge loses its stability during a maximum credible earthquake, and collapse could inevitably occur, which is generally is characterized by a structural failure of the material and / or instability in one or more components. Thus, ductility is defined as the ratio of ultimate deformation to the deformation at first yield $\mu = \delta_{max}/\delta_y$, that is the predominant measure of structural ability to dissipate energy. Caltrans takes advantage of ductility and post-elastic strength and does not design ordinary bridges to remain elastic during design earthquakes because of economic constraints and the uncertainties in predicting future seismic demands. Seismic deformation demands should not exceed structural deformation capacity or energy-dissipating capacity.

Aim

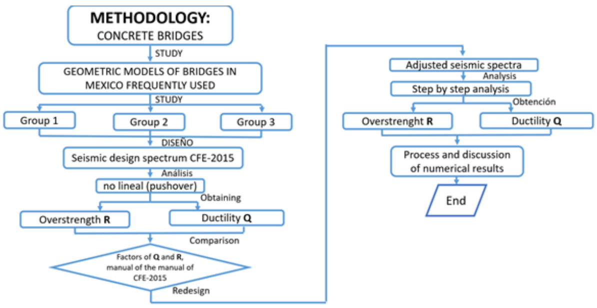
The determination and refinement of the factors of overstrength R and ductility Q, are the main objective, since it has caused controversy regarding its validity, being a very important factor related to the real capacity of energy dissipation of the structures. So, there is currently uncertainty about the values given by the Manual CFE-2015 [1], which brings with it a lack of confidence in the non-linear behavior of the bridges before a seismic event.

2. Methodology used

2.1 Seismic Design

Based on all the above and considering the aim of this research this methodology has been proposed (see Figure 1).

To elaborate this methodology, the following were considered:



1. The type of bridge, the selection of components, the dimensions of the elements should be investigated to reduce seismic demands to the greatest extent possible.
2. Simplified analysis models should be used for the initial assessment of structural behavior. The results of more sophisticated models should be verified to see if they are consistent with the results obtained from the simplistic models. The translational rigidity of the superstructure, the abutments and the foundations, modeled and analyzed seismically, must be compatible with its structural and geotechnical capacity.
3. The estimated displacement demands under a design earthquake should not exceed the global displacement capacity of the structure and the local displacement capacity of any of its individual components.
4. For concrete bridges, structural components must be proportioned to address inelastic damage to columns, piers and abutments. The superstructure must have enough strength and rigidity to remain essentially elastic if the columns reach their most likely plastic moment capacity.

Fig. 1 – Methodology proposed

2.2 Proposed study structures

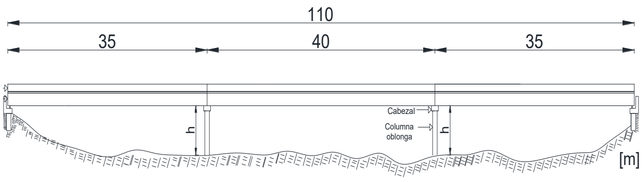


Fig. 2.a – Longitudinal section of the bridge.

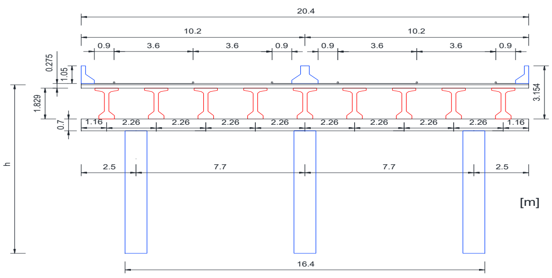


Fig. 2.b – Transverse bent with prestressed beams type AASHTO VI.

Structured bridges were proposed with a deck by precast concrete girders AASHTO type VI, with four lanes, and a continuous deck, with a road width, of 20.40m; and 3 spans (35, 40, 30 m), with a total length of 110m, with two intermediate supports, formed by a transverse bent with 3 columns and a cap head, of concrete. The foundation supports are considered embedded at the base on a rock soil (see figures 2.a, 2.b). The location of each diaphragm is about 10m respectively. The mechanical a geometric characteristic are show in tables 2 and 3. Three groups (I, II and III) of structures are analyzed, with variable column heights, as shown in Table 4. The structures are in an area of high seismicity, on the Pacific coast, in Acapulco, Gro.

Table 1 – Overstrength R

Comparison of the Factors		
Regulation		**Overstrength R**
CFE-2015	Bridge	1.5
		Critical Essential
Caltrans, 1990 Substructure	Vertical pile only	1.5 - 2
	Concrete pile bents	1.5 - 2
	Single column	1.5 – 2
	Multiple-column bents	1.5 – 3.5

Geometric and mechanical properties

The geometric and mechanical properties are shown in tables 2 and 3.

Table 2 – Mechanical characteristics of the materials; Table 3 – Geometric characteristics of the bridge

Material	Strength Mpa (kg/cm^2)
Concrete columns	f´c=35 (350)
Concrete precast, I beam AASHTO TYPE VI	f´c=40 (400)
Concrete slab	f´c=25 (250)
Concrete footing	f´c=25 (250)
Concrete diaphragm	f´c=30 (300)
Concrete header beam	f´c=30 (300)
Structural steel	fy=420 (4200)
Material	Weight γ (ton/m^3)
Concrete	2.4
Asphalt	2.2
Material	Poisson ratio
ν_c (concrete)	0.2
ν_s (steel)	0.3

Concept	Dimensions
Track width	3.60 m
Dimensioning	0.90 m
Number of lanes	4
AASHTO beam type VI	9.00 pzas
Road width	7.20 m
Ground load capacity	Q=40 T/m^2

Table 4 – Groups I, II and III of bridges with varying heights of columns.

Elevation, longitudinal direction of the bridge

	Heights h_1 and h_2 (m)		
Case	Group I	Group II	Group III
1	7.5 – 4.5	8.5 – 4.5	10 – 4.5
2	7.5 – 5.5	8.5 – 5.5	10 – 5.5
3	7.5 – 6.5	8.5 – 6.5	10 – 6.5
4	7.5 – 7.5	8.5 – 7.5	10 – 7.5
5	7.5 – 8.5	8.5 – 8.5	10 – 8.5
6	7.5 – 10	8.5 – 10	10 – 10

3. Structural Analysis

The structural analysis of all the bridges is performed numerically with the help of the CSI-Bridge analysis program, their results were validated with analytical methods.

3.1 Column dimensioning

The sizing of the columns was determined under a level of displacement or an objective drift, the goal of this approach is to ensure that the structural system and its individual components have enough capacity to withstand the deformation imposed by the design earthquake, Tolentino [7]. The use of displacements instead of forces as a measure of earthquake damage allows a structure to perform the required functions. Thus, for this investigation, the columns were sized with an objective drift. By using the design spectrum see figure 9

(*Transparent seismic spectrum, in Acapulco, Gro.),* given by CFE-2015 [1], Prodisis program. The columns were dimensioned and calculated, considering that seismic spectrum. The structures are analyzed with the help of the CSI-Bridge program and the most unfavorable results of the columns are obtained: such as the maximum distortions and the mechanical elements. The characteristics of weight of the structures are shown in table 5.

Table 5 – Total weight of the bridge structure

a) Total weight of bridge with constant height of 10m in height.

Components of the bridge	Weight (kN) (9.81kN=1Ton)
Superstructure (Deck)	402.62
Substructure	61.54
Total	464.16

b) Total weight of bridge with variable height of 10-6.5m Tall

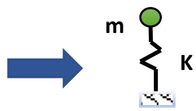
Components of the bridge	Weight (kN) (9.81kN=1Ton)
Superstructure (Deck)	402.62
Substructure	49.92
Total	452.54

3.2 Modal analysis

Given the characteristics of concrete bridges studied for Mexico, which are classified as common bridges, with a regular geometry, and a superstructure of constant width, continuous between the substructure and the superstructure, forming frames in both directions (longitudinal and transversal), of constant mass, placed in rock soil and located in an area of high seismicity of the Mexican Pacific. Firstly, simplified models are used for the evaluation of dynamic behavior, with translational stiffness of substructure and abutments, the period of vibration and that later, serve to verify the results of more sophisticated numerical models are consistent (see figure 4).

concrete bridge model

Fig. 4 – Simplified concrete bridge model



one degree of freedom model, 1dof

Thus, this structure is idealized as 1dof model, taking into a count the translational stiffness of superstructure, giving the equation of motion of a system with one degree of freedom [8], see the equation 1.

$$\ddot{y}(x) + \omega^2 y(x) = 0 \qquad (1)$$

were: $w = \sqrt{\frac{K}{m}}$ *frequency*; $T = \frac{2\pi}{w}$ *period*

3.3 Drifts of the columns of bridges

The distortions of the bridges columns studied were dimensioned, considering levels of objective drift less than (δ / H ≤ 0.006), [7], since these bridges are characterized by having short periods of vibration less than (Ts <1s), so that with the sections of the columns defined, drift levels of 0.004 were reached, see figure 5.a. Based on these levels of drifts, the first sections of columns had the following dimensions, see figure 5.b.

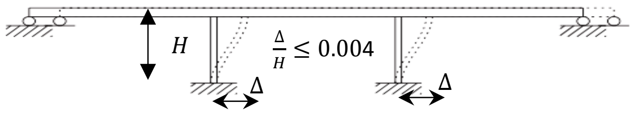


Fig. 5.a – Bridge drift profile.

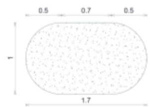


Fig. 5.b – Geometry of the columns.

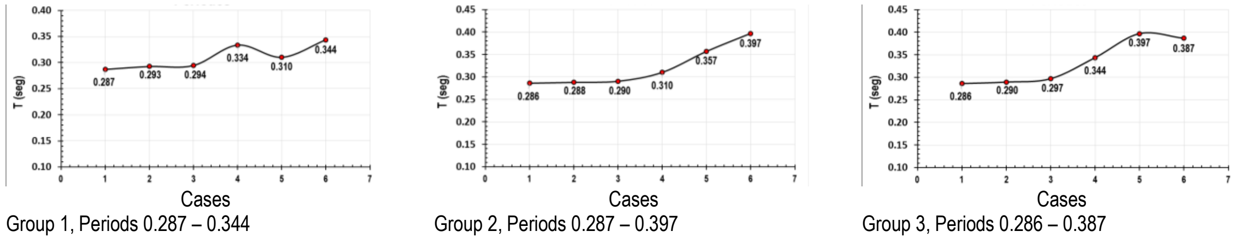
Table 6 summarizes the periods and frequencies of the bridge, and figure 6 shows modal form associated to the period in **X** direction.

Figure 6 – Modal form associated to the period in *X* direction

Table 6 – Comparison of the Periods and Frequencies of the bridges

Bridge Characteristics	Direction	Comparison Analytical		CSi Bridge		
		ω	T (s)	ω	T_n (s)	T_n/T
Constant height of 10 m in height (Group III)	**X**	15.7383	0.3992	14.7134	0.3865	0.968
	Y	22.2158	0.2829	24.4958	0.2674	0.945
Variable height of 10-6.5m. (Group III)	**X**	23.7979	0.2640	21.1519	0.2971	1.125
	Y	25.7358	0.2441	27.8453	0.2257	0.9246
Constant height of 7.5 m in height (Group I)	**X**	17.9562	0.3499	17.3359	0.334	0.954
	Y	25.5296	0.2140	24.4958	0.220	1.028

In figures 7.a, 7.b and 7.c, the vibration periods of all the studied structures are shown, which include the three group of bridges (I, II, II), and in table 7 shows distortions of group II.

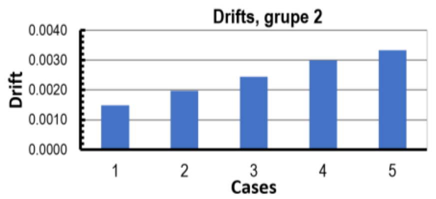


a. – For the 6 cases of group I   b. – For the 6 cases of group II   c. – For the 6 cases of group III

Fig. 7 – Vibration periods T (s)

Table 7 – Distortions of group II

Case	h_1 (m)	h_2 (m)	Deck displacement δx (cm)	δx (cm)	Drifts $\delta x/h_1$	$\delta x/h_2$
1	8.5	4.5	1.264	1.508	0.0015	0.0034
2	8.5	5.5	1.677	1.537	0.0020	0.0028
3	8.5	6.5	2.078	1.625	0.0024	0.0025
4	8.5	8.5	2.545	1.255	0.0030	0.0015
5	8.5	10.0	2.834	1.852	0.0033	0.0019



4. Linear Analysis

4.1 3D Numerical modeling

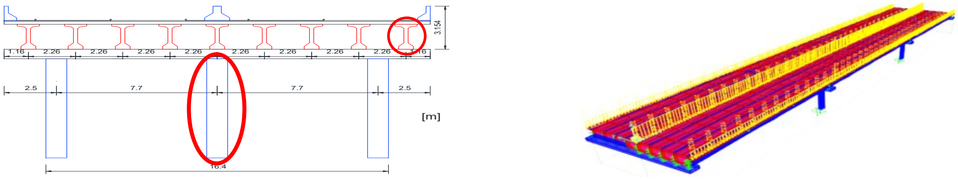
The bridge structures were analyzed using 3D three-dimensional numerical models, by beam elements and plate elements, see figure 8. They were analyzed and designed a typical highway bridges, which are in Acapulco coast, Guerrero, in the rocky subduction zone of the Pacific. The bridge has a length of 110.00 m, with a deck width of 20.40 m, consisting of 4 lanes (see figures 3.a, 3.b). The supports of the columns are embedded, and the abutments can slide only in the longitudinal direction (***$u_x \neq 0$ and u_z=u_y=rot_x=rot =rot_z=0***).

4.2 Modeling

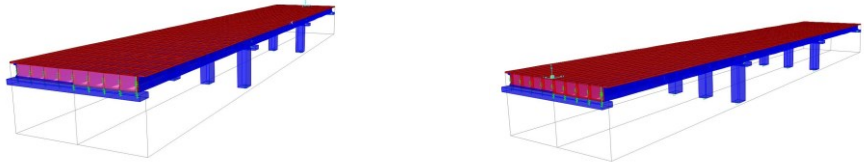
They were designed with the most unfavorable loads and combinations, according to the recommendations of the AASHTO code, 2002, IMT, 2007, Caltrans, [2, 4, 3] which are used in these types of bridges.

Design of structural elements

Based on the recommendations of the design codes, in the structuring of the studied bridges, seismic design methodologies for bridges are used, considering the displacement approach, so in these concrete bridges, the structural components are provided to address the inelastic damage to the columns and abutments and the superstructure presents a sufficient resistance to remain essentially elastic, while the columns reach their capacity of plastic moment. Also, all connections and joints should ideally be designed to remain essentially elastic. Figure 8 shows the linear analysis model that was used to design the elements of the bridge, under gravitational load and earthquake. NTC-RC-2017, ACI-318 and AASHTO-LRFD, [9,10,1], were used to design the precast concrete beams, slab, head cap, columns and abutments. The seismic actions were obtained from the selection of the seismic design spectra provided by the CFE-2015 and the IMT standards [1, 4], for the Mexican Pacific region.



a. Transversal bent. b. 3D view of bridge



c. 3D longitudinal view of bridge

Fig. 8 – Numerical models

4.3 Seismic design spectrum

MDOC-DS, CFE-2015. Using the "Prodisis program", the CFE [1] seismic design spectrum for that area is shown in figure 9, and according to this manual [1], the overstrength is proposed to reduce the spectrum with a factor **R**= 0.5 (see table 8).

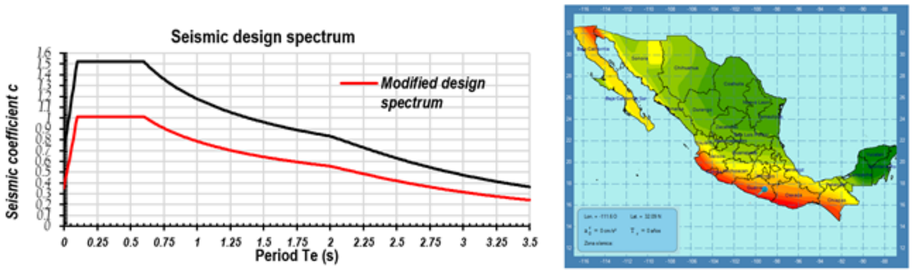


Fig. 9 – Transparent seismic spectrum (black line) and modified design spectrum (red line) with Q=1 and R =1.5 given by CFE-2015 Prodisis program, (Acapulco, Gro).

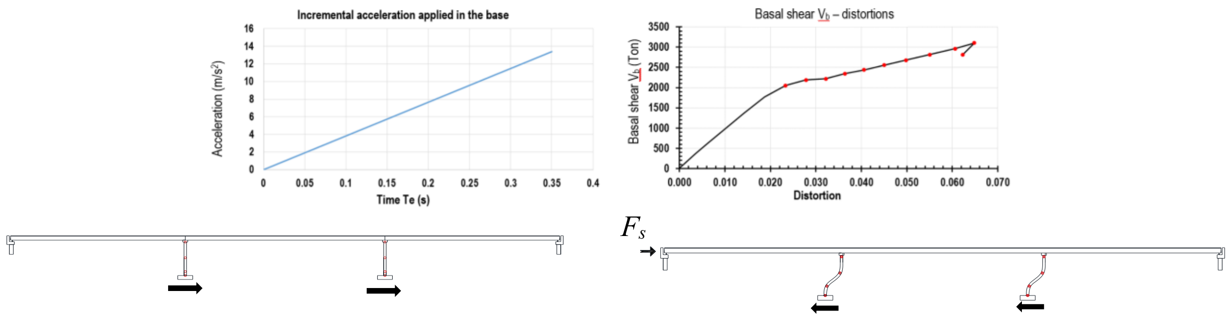
Table – 8 Ductility and overstrength, factors

Regulation:	Ductility Q	Overstrength R
CFE-2015	1	1.5

5. Nonlinear Analysis

5.1 Non-linear analysis (Pushover)

This method consists of a succession of incremental analyzes that together define the non-linear response of the structure. The lateral load applied to the bridge is gradually increased, until a maximum demand is reached (see figure 10). The non-linear analysis was carried out by DRAIN-2DX program, which it is a 2D no-linear analysis program, through which the analysis is achieved, considering the variation of stiffness and resistance of the structural elements, beams and columns, with a model of bilinear hysteresis. The detailed design of the columns, head and precast concrete girder, the (dimensioning and assembly), is shown in [11].



Excitation Lateral displacement under an incremental load F_s

Fig. 10 – Excitation and Response Curve V_b *vs* distortion

5.2 Non-linear analysis and bridge capacity curves

Comparison and discussion of results obtained from typical bridges. This section shows the results of the non-linear analysis for the groups I and III of bridges studied, mentioned in table 4, as well as comparison and discussion of the results, with the overstrength factor R given by [1] with values obtained here. The "pushover" or progressive plasticization analysis of the bridge structure is carried out by means of a model in 2D with DRAIN-2DX program. The images 11.a and 11.b, show the numerical models of analysis developed to carry out the non-linear analyzes, for the geometries of the bridges with constant height and variable height, respectively.

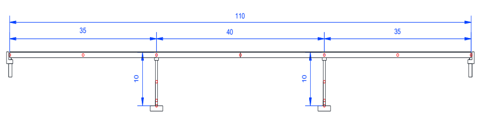


Figure 11.a – Discretization of bridge with constant height of h_1=h_2=10m

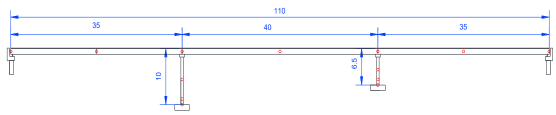
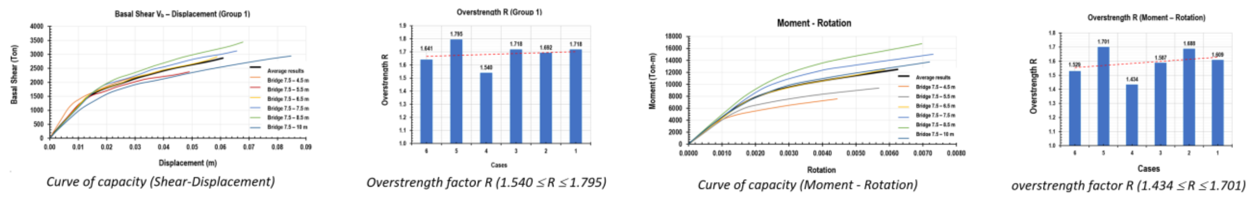


Fig. 11.b – Bridge discretization with variable height of h_1=10m and h_2=6.5m

5.3 Group I. Bridges variable height keeping in height 7.5m for the 6 cases studied.



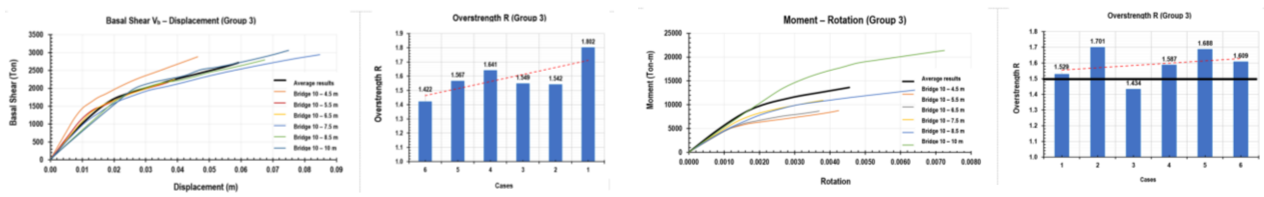
a. Curve of capacity (Shear-Displacement) and Overstrength factor R ($1.540 \leq R \leq 1.795$)

b. Curve of capacity (Moment - Rotation) and Overstrength factor R ($1.434 \leq R \leq 1.701$)

Fig. 12 – Graphs of the group I for the 6 cases studied

The results obtained from the bridges of group I mentioned in table 4, are shown firstly in the figure 12.a, the overstrength factors R and their curves of capacity shear *vs* displacement, and in the figure 12.b shown the graphs of capacity, moment *vs* rotation, like so their values of the overstrength factor R.

5.4 Group III. Bridges variable height keeping in height 10 m for the 6 cases studied.



a. Curve of capacity (Shear-Displacement) and Overstrength factor R ($1.540 \leq R \leq 1.795$)

b. Curve of capacity (Moment - Rotation) and Overstrength factor R ($1.434 \leq R \leq 1.701$)

Fig. – 13 Graphs of the group III for the 6 cases studied

The results obtained from group III are described in figure 13.a such as, the overstrength factors R and the curve capacity shear *vs* displacement. The figure 13.b shows the graphs of capacity curves, moment *vs* rotation and their values of overstrength factors R.

5.5 Damage configuration

The damage configuration (figure 14) shows the location of the plastic joints, generated by the excitation, as well as the evolution of the damage. This evolution of the damage was numbered sequentially, in the way in which the loss of capacity of the structural elements (columns) is presented, at the moment in which the fibers of the cross-section of the element yield, thus generating the damage in that cross section and producing the failure of that element, until the collapse. Figure 16 show the configuration and sequence of damages and the curves for the non-linear push-over analysis of the bridge. The evolution of the displacement *vs* basal shear is observed, the capacity curve of the substructure under the action of the incremental load to the fault. From this figure, it can be seen, that for all cases studied, the curves acquire the same pattern of behavior, and that they have different values of the resistance **R** that go, from 1,573 to 1,584. These **R** values are compared with the factor recommended by the CFE manual [1] (**R = 1.5**), which shows that this given factor is considered close in most of the bridge structures with the characteristics studied.

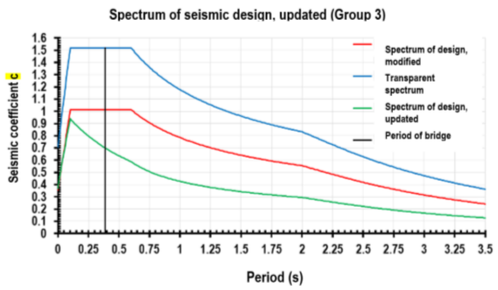
Fig. 14 – Bridge with variable height of h_1 = 10m and h_2 = 6.5m, (group III, case 3.), and damage setup and sequence.

5.6 Non-linear analysis (step by step)

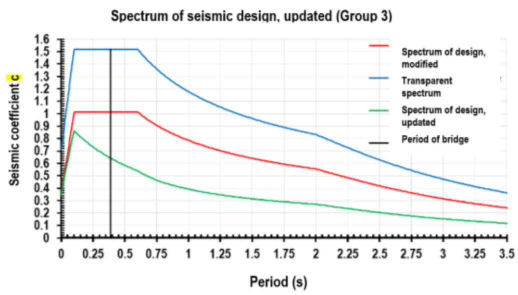
To perform nonlinear analysis time (step by step), it proceeded as follows:

- The seismic design spectra are updated, readjusting them with the new values of the overstrength factors **R** and a seismic behavior factor **Q = 2,**
- By the modified seismic spectra, the structural elements were redesigned,
- Finally, step by step non-linear analyzes are carried out with the real and simulated accelerograms corresponding to the seismic study region.

The purpose is to study the behavior and response of the structures and the influence of the overstrength factor **R** in the seismic response of these.



Modified design spectrum, Q=2 and R=1.422 for bridge with constant height h=10m (group III)



Modified design spectrum, Q=2 and R=1.549 for bridge with variable height of columns h_1=10 and h_2=6.5m (group III)

Fig. – 15 Spectra tuned up with an equal ductility 2 and a value of overstrength factor R

5.7 Analysis and re-design of structural elements of the bridge

A new calculation is made that involves affecting the design spectrum CFE-2015 [1] with a ductility of **Q** equal to 2 in all cases and with an overstrength factor **R** obtained in the previous calculation, then the spectra obtained after of the adjustment (see figure 15). For the all cases of the three groups (I, II and III) studied, all spectrum was tuned up in the same way.

5.8 Seismic records selected to carry out the time history analysis

Table – 9 Data of the seismic records

Name	**Δt (s)**	**Number of points**	**Time (s)**
Prodisis	0.01	13643	136.42
Prodisis 10%	0.01	13643	136.42
Unión (real seismic record)	0.01	6232	62.29

The real seismic record considered is that of the Union of the earthquake of 1985, and the second simulated record was obtained with the program Prodisis v4.1 of [1], given that these present dominant short periods, characteristic of firm soil, and that become more unfavorable for these structures, see table 9. In figures 16, the 3 seismic records and their respective response spectrum are shown.

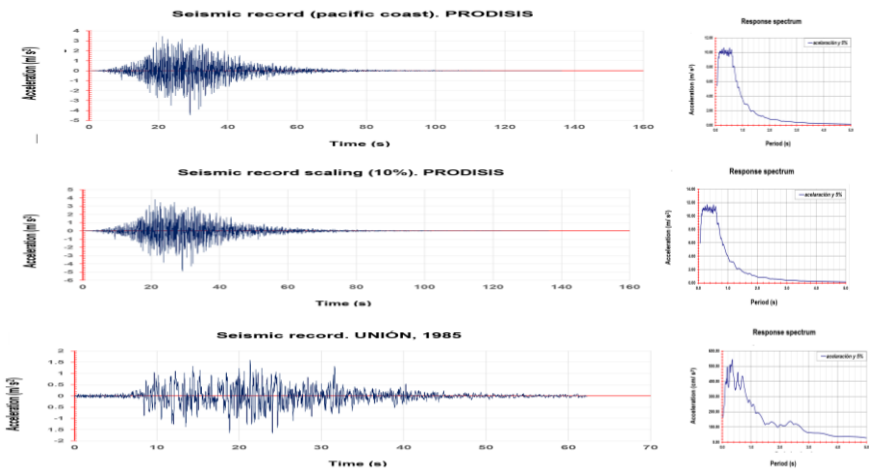
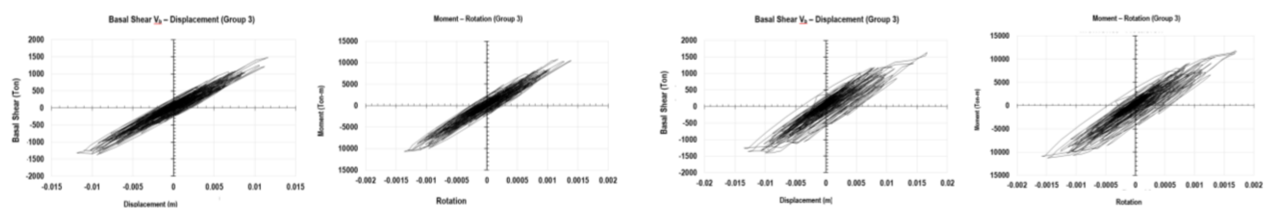


Fig. – 16 Seismic records and response spectra used in the analyzes.

5.9 Seismic response of the redesigned bridges

In order to know the behavior and the seismic response of the redesigned bridges with the factors of ductility Q and overstrength R, the hysteresis curves, shear *vs* distortion and moment vs rotation, derived from the step-by-step analyzes are presented below. It is important to mention that the analysis was carried out in the longitudinal **X** direction, given that it is the most vulnerable direction before seismic actions. The results for 3 cases of group III are shown below (see figures 17, 18, 19 and 20).

Hysteretic cycles (Group III, case 6)



a) **Basal shear *vs* displacement**; ductility Q=1.212 and overstrength R=1.226

b) **Moment *vs* Rotation**; ductility Q=1.117 and overstrength R=1.050

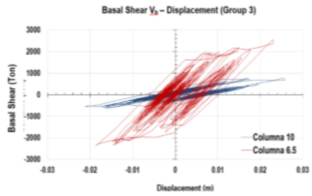
a) **Basal shear *vs* displacement**; ductility Q=1.438 and overstrength R = 1.281

b) **Moment *vs* Rotation**; ductility Q=1.404 and overstrength R=1.087

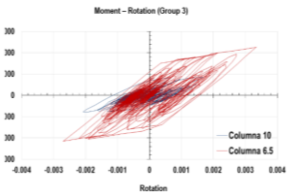
Fig. – 17 Hysteretic cycles, h_1=h_2=10m, bridge with Prodisis accelerogram.

Fig. – 18 Hysteretic cycles, h_1=h_2=10m, bridge with Prodisis accelerogram with 10%.

Hysteretic cycles (Group III, case 3)



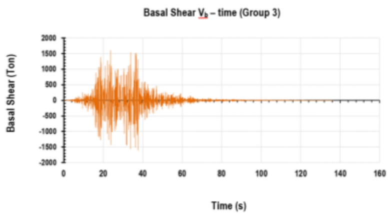
a. Basal Shear *vs* Displacement, for 2 columns



b. Moment *vs* Rotation, for 2 columns

c. Damage setup and sequence

Fig. – 19 Hysteretic cycles comparison of for case 3, with 2 columns of variable height h_1=10 and h_2=6.5 with Prodisis accelerogram with 10%.



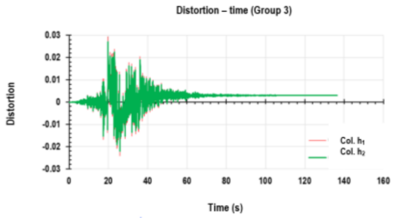


Fig. – 20 History of shear and drifts for the two columns of variable height h_1= 10 and h_2= 6.5 m with Prodisis accelerogram with 10%.

Group III, case of fixing h_1=h_2=10m constant and variable column heights (h_1 = 10, h_2 = 6.5m).

Group	Case	Heights h_1 and h_2 (m)
III	3	10 – 6.5
	6	10 – 10

5.10 Factor λ (*Lambda*)

With the purpose of comparing the demand vs the capacities, we propose a factor that we define lambda factor **λ**, which represents a quotient of the overstrength capabilities "C" vs the overstrength demands "D". If the lambda factor **λ** is greater than one (λ> 1) has a level is acceptable, otherwise you have an unacceptable level.

Factor **λ** (***Lambda***) $$\boldsymbol{\lambda = \frac{C}{D}}$$

Figures 21, 22 and 23, shown the results of the lambda factor.

Group I

Method / Case		Push - over Overstrength	Step – by step Overstrength	C/D λ
1	7.5-4.5	1.718	1.3279	1.2939
2	7.5-5.5	1.6917	1.2376	1.3669
3	7.5-6.5	1.7185	1.1930	1.4405
4	7.5-7.5	1.5400	1.1229	1.3715
5	7.5-8.5	1.7948	1.3253	1.3543
6	7.5-10	1.6406	1.3020	1.2601

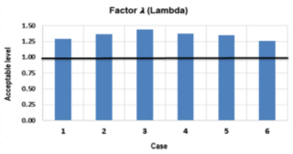


Fig. – 21 Factor **λ** Group I

Group II

Method / Case		Push - over Overstrength	Step – by step Overstrength	C/D λ
1	8.5-4.5	1.7074	1.0379	1.6451
2	8.5-5.5	1.6212	1.3691	1.1842
3	8.5-6.5	1.5841	1.2409	1.2766
4	8.5-7.5	1.7948	1.3253	1.3543
5	8.5-8.5	1.5130	1.2174	1.2428
6	8.5-10	1.5667	1.5169	1.0328

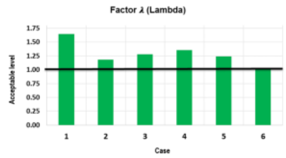
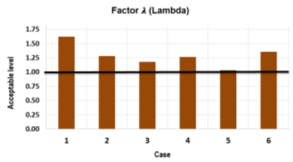


Fig. – 22 Factor **λ** Group II

Group III

Method / Case		Push - over Overstrength	Step – by step Overstrength	C/D λ
1	10-4.5	1.8016	1.1131	1.6186
2	10-5.5	1.5425	1.2076	1.2773
3	10-6.5	1.5491	1.3182	1.1752
4	10-7.5	1.6406	1.2981	1.2638
5	10-8.5	1.5667	1.5169	1.0328
6	10 10	1.4220	1.0510	1.3530



From these images it is observed that for all the cases of the three groups of structures the level is acceptable, that there is a margin of safety of the

Fig. – 23 Factor λ Group III

structures, under the maximum seismic excitation that can occur.

6. Conclusions and recommendations

In this research assessment factors on resistance R, to construct the seismic design spectra, for bridges designed in accordance with the criteria manual CFE-2015 [1], as well as the standards: AASHTO and Caltrans [2, 3].

Based on the seismic criteria of different regulations considered and considering the objectives of this research, a methodology was proposed. This methodology was applied to proposed bridge structures for the determination of overstrength R factors and their comparison with the recommendations given in the CFE-2015 manual [1]. Achieving satisfactory results.

- Bridges geometries with variable column heights were studied and it was observed that all the structures are of short period T (0.28 - 0.40 s).
- The values obtained numerically of the overstrength factor R are found, between from 1,422 to 1,8016 (group III), which shows that this factor recommended by [1] is in general, acceptable for most bridge structures studied. However, it is necessary to study these structures in other seismic regions of the country, to obtain more representative values of this type of bridges.
- About the distortions obtained numerically, it is shown that the distortion intervals for these types of bridges studied range from 0.0019 to 0.004, and they are close to the distortion proposed as target displacement at the beginning of the dimensioning of sections the columns.
- By using the pushover method to obtain the overstrength factor R, it was also possible to observe the evolution of progressive damage in the formation of the plastic hinge in the sections of the columns.

6. References

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