Structural response evaluation of two-blade bridge piers subjected to a localized deterioration

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ABSTRACT: Two-blade bridge piers represent today an innovative solution to optimize the structural behavior of long viaducts subjected to seismic actions. In fact, a proper design of the structure provides a correct transmission of vertical loads to the foundation structures and the possibility to allow horizontal displacements without the use of kinematics mechanisms.
Due to the particular configuration of the pier (a rigid box section in the bottom part and two flexible blades at the top connected with the viaduct deck), the structural behavior can be achieved only using refined analysis formulations and effective numerical tools. In particular, during the nonlinear response, the shear effects are very important and can govern the structural behavior. Nonlinear models have to be able to consider the shear damaging with sufficient accuracy otherwise, the numerical response would be affected by heavy errors. In addition, uncertainties on material characteristics can affect the numerical response. To handle these important uncertainties involved in the problem, the use of probabilistic or fuzzy approaches can be suitable.
In this paper, to show the influence of the degradation on the structural response a new and a damaged two-blade pier will be analyzed. Probabilistic evaluation of the material properties are considered to improve the reliability of the numerical analyses. The aim of the paper is to show the importance of a reliable modeling of the damaged zone.

1 INTRODUCTION

In the Conceptual Design of bridges, in areas of moderate to high seismicity, the selection of ductile behavior is a general strategy inside the so-called Integral Bridge Design (Biondini 2001, Calzona & Bontempi 2001). Its implementation, either by providing the formation of a dependable plastic mechanism or by using base isolation and energy dissipating devices, must be decided. When a ductile behavior is selected, the following main points should be considered.

a) The number of supporting elements (piers and abutments) that will be used to resist the seismic forces in the longitudinal and in the transverse direction must be decided. In general continuous structures behave better under earthquake conditions than bridges having many movement joints. The optimum post-elastic seismic behavior is achieved if plastic hinges develop approximately simultaneously in as many piers as possible. However, the number of earthquake resisting piers may have to be reduced, by using flexible mountings between deck and piers in one or in both directions, to avoid either high reactions due to restrained deformations or an undesirable distribution of the seismic actions and/or of the capacity design effects.

b) A balance should be maintained between the strength and the flexibility requirements of the horizontal supports. High flexibility reduces the level of the design seismic action but increases the movement at the joints and moveable bearings and may lead to high second order effects.

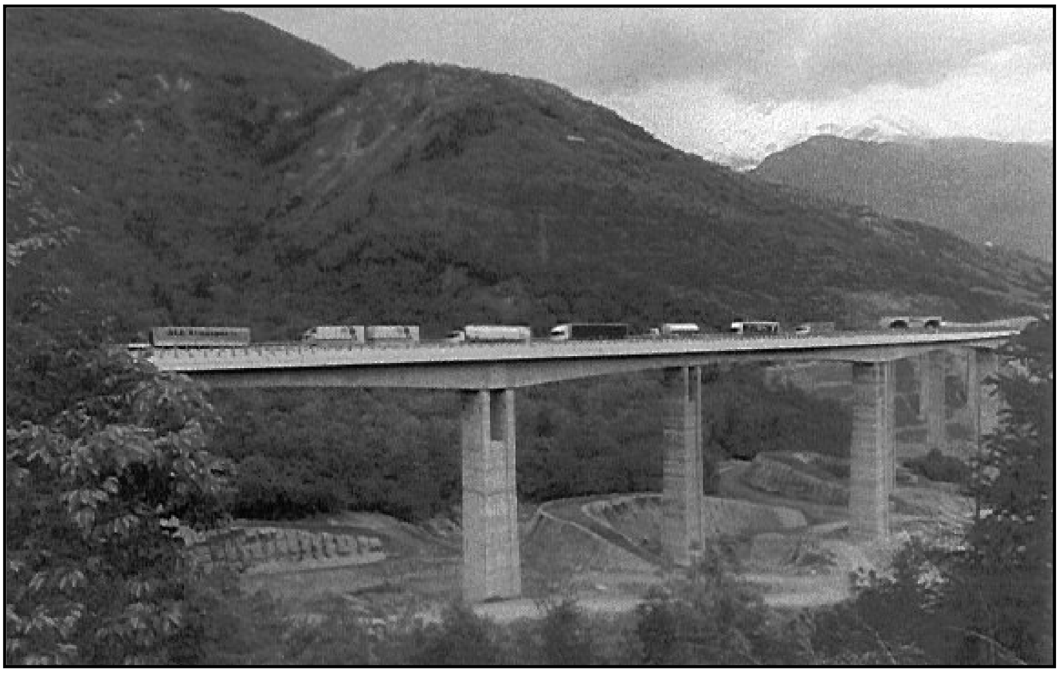

Figure 1. An image of the two-blade pier (Mancini et al. 1998).

A structural scheme, which can furnish the right structural restoring, refers to a two-blade pier: a structural element often used in bridge design (Figure 1). The two-blade pier has a double aim. The first one is the transmission of vertical loads to the foundation structures; the second one is the possibility to allow horizontal displacements (thermal effects, concrete shrinkage, etc.) without the use of cinematic mechanisms. The ratio α between the length of the top flexible two-blade cross-section (h_1) and the total pier height ($h=h_1+h_2$) is used to characterize the structural behavior, with respect to the horizontal loading system:

$$\alpha = \frac{h_1}{h_1 + h_2} \quad (h_1 = \text{top part height},\ h_2 = \text{bottom part height}) \tag{1}$$

If the two-blade section has a great height as to the total pier height, the behavior will have great ductility but little strength. Instead, if the rigid box section is dominant, both stiffness and strength increase, as to the horizontal loads, but the ductility of the pier decreases. The choice of the relative height of the rigid box section has then great importance since it influences heavily the global structural behavior.

A material nonlinear analysis is developed in order to know the global behavior of this structure. This will be made using a solid (three-dimensional) finite element modeling (Sgambi 2001, Sgambi & Bontempi 2002). In this way, shear behavior and diffusive effect will be directly considered into the model. In particular, the maximum horizontal displacement and the maximum level of the horizontal load at the top of the pier are considered in order to summarize the nonlinear structural behavior.

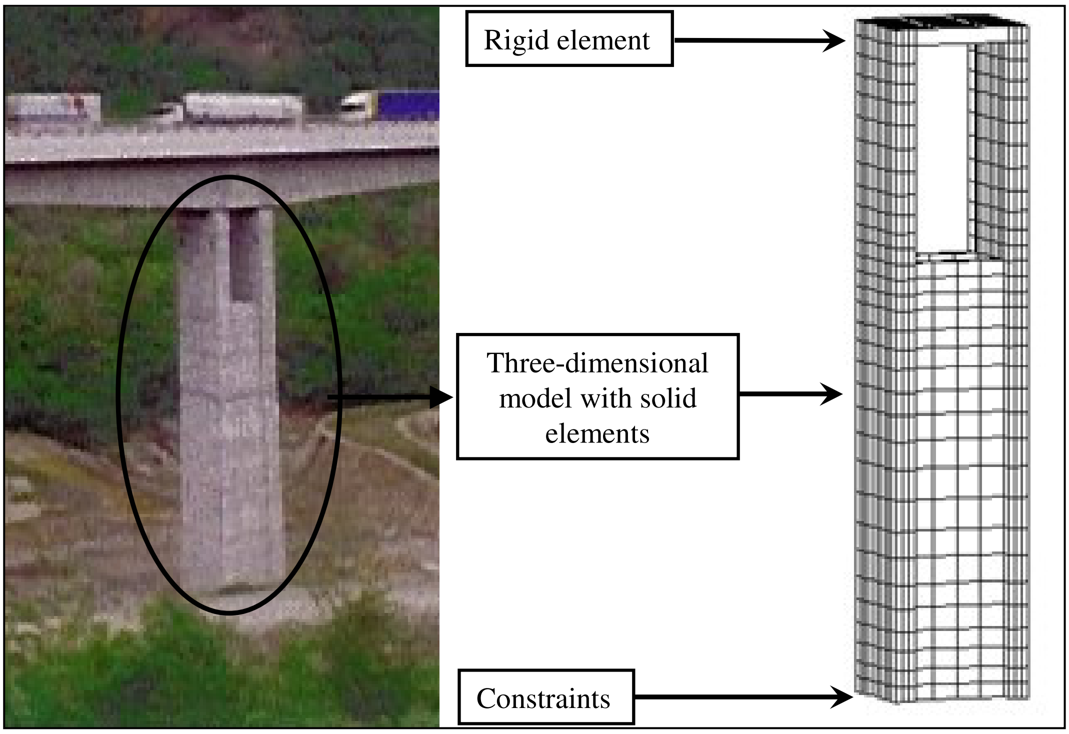

Figure 2. The three-dimensional model used in the analyses.

2 NONLINEAR MODELING

A three-dimensional generalization of the membrane constitutive law (CFT, MCFT) has been implemented to study the material nonlinear behavior (Vecchio & Collins 1988, Vecchio 1989, Hsu 1998). In these models, the concrete stiffness matrix is assumed diagonal, in the principal reference system, and its terms are computed using uniaxial laws. The material law is based on a rotating crack model with non-local damage regularization (Sgambi 2001). The uniaxial laws for the compression concrete are reported in the following equations.

$$\sigma_d = \zeta \cdot f'_c \left[2\left(\frac{\varepsilon_d}{\zeta \cdot \varepsilon_0}\right) - \left(\frac{\varepsilon_d}{\zeta \cdot \varepsilon_0}\right)^2 \right] \text{ if } \left(\varepsilon_d \leq \zeta \cdot \varepsilon_0\right) \tag{2}$$

$$\sigma_d = \zeta \cdot f'_c \left[1 - \left(\frac{\frac{\varepsilon_d}{\zeta \varepsilon_0} - 1}{\frac{2}{\zeta} - 1} \right)^2 \right] \text{ if } \left(\varepsilon_d > \zeta \cdot \varepsilon_0\right) \tag{3}$$

$$\zeta = \frac{1}{\sqrt{1 + 400 \cdot \varepsilon_{eq}}} \tag{4}$$

where, the softening parameter ζ is used to reproduce the compression strength in presence of a transversal tensile strain. The equivalent strain is assumed equal to:

$$\varepsilon_{eq} = \sqrt{\langle \varepsilon_1 \rangle^2 + \langle \varepsilon_2 \rangle^2 + \langle \varepsilon_3 \rangle^2} \tag{5}$$

where:

$$\langle \varepsilon_i \rangle = \begin{cases} \varepsilon_i & \varepsilon_i \geq 0 \\ 0 & \varepsilon_i < 0 \end{cases} \tag{6}$$

The constitutive law for the tensile concrete is:

$$\sigma_r = E_c \cdot \varepsilon_r \quad \text{if } (\varepsilon_r \leq \varepsilon_{cr}) \tag{7}$$

$$\sigma_r = f_{cr} \left(\frac{\varepsilon_{cr}}{\varepsilon_r} \right)^{0.4} \quad \text{if } (\varepsilon_r > \varepsilon_{cr}) \tag{8}$$

where E_c is the initial elastic modulus and f_{cr} is the cracking stress:

$$E_c = 9500 \cdot \sqrt[3]{f_c} \tag{9}$$

$$f_{cr} = 0.25 \cdot \sqrt[3]{f_c^2} \tag{10}$$

The shear modulus is assumed equal to:

$$G_{ij} = \frac{E_i \cdot E_j}{E_i + E_j} \tag{11}$$

The stiffness matrix of the material $\underline{\underline{D}}$ is evaluated, in the principal directions system, by means of the uniaxial laws Equations 2-11. The matrix $\underline{\underline{D}}_{xyz}$, in the global reference system is:

$$\underline{\underline{D}}_{xyz} = \underline{\underline{T}}^T \cdot \underline{\underline{D}} \cdot \underline{\underline{T}} \tag{12}$$

where $\underline{\underline{T}}$ is a rotation matrix.

$$\underline{\underline{T}} = \begin{bmatrix} l_1^2 & m_1^2 & n_1^2 & l_1 \cdot m_1 & m_1 \cdot n_1 & l_1 \cdot n_1 \\ l_2^2 & m_2^2 & n_2^2 & l_2 \cdot m_2 & m_2 \cdot n_2 & l_2 \cdot n_2 \\ l_3^2 & m_3^2 & n_3^2 & l_3 \cdot m_3 & m_3 \cdot n_3 & l_3 \cdot n_3 \\ 2 \cdot l_1 \cdot l_2 & 2 \cdot m_1 \cdot m_2 & 2 \cdot n_1 \cdot n_2 & l_1 \cdot m_2 + l_2 \cdot m_1 & m_1 \cdot n_2 + m_2 \cdot n_1 & n_1 \cdot l_2 + n_2 \cdot l_1 \\ 2 \cdot l_2 \cdot l_3 & 2 \cdot m_2 \cdot m_3 & 2 \cdot n_2 \cdot n_3 & l_2 \cdot m_3 + l_3 \cdot m_2 & m_2 \cdot n_3 + m_3 \cdot n_2 & n_2 \cdot l_3 + n_3 \cdot l_2 \\ 2 \cdot l_3 \cdot l_1 & 2 \cdot m_3 \cdot m_1 & 2 \cdot n_3 \cdot n_1 & l_3 \cdot m_1 + l_1 \cdot m_3 & m_3 \cdot n_1 + m_1 \cdot n_3 & n_3 \cdot l_1 + n_1 \cdot l_3 \end{bmatrix}$$

(13)

The coefficients l_i, m_i, n_i, are the director cosines of the principal directions and can be derived from the strain tensor, using a Jacoby procedure.

The reinforcement steel is considered like a smeared material into the concrete and its constitutive law is assumed elastic-perfectly plastic:

$$\sigma_s = E_s \cdot \varepsilon_s \quad \text{if} \quad \varepsilon_s < \varepsilon_y \qquad \text{or} \qquad \sigma_s = f_y \quad \text{if} \quad \varepsilon_s \geq \varepsilon_y \tag{14}$$

Consequently, the composite material stiffness can be evaluated directly adding the steel stiffness matrices to the concrete stiffness matrix. Every stiffness matrix must be, obviously, referred to the global reference system; therefore, the composite material stiffness results as:

$$\underline{\underline{D}} = \underline{\underline{T}}_C^{\ T} \cdot \underline{\underline{D}}_C \cdot \underline{\underline{T}}_C + \sum_I \underline{\underline{T}}_{SI}^{\ T} \cdot \underline{\underline{D}}_{SI} \cdot \underline{\underline{T}}_{SI} \tag{15}$$

The fidelity of the concrete model implemented is proven by many comparisons made with experimental studies (Sgambi 2001).

In previous papers (Sgambi & Bontempi 2002, Sgambi 2004) the authors show that the structural behavior depends strongly from the α parameter defined in equation 1. In this paper the authors focus their attention on a pier with parameter $\alpha = 1 / 3$. Figure 3 shows the crack orientation and the damaged zone during the nonlinear analysis when the top of the pier is loaded with horizontal forces, perpendicular at the blade plane. One can observe that a shear mechanism is present into the rigid box section in the transfer zone. The shear mechanism is due to the section change and it influences very much the nonlinear pier behavior.

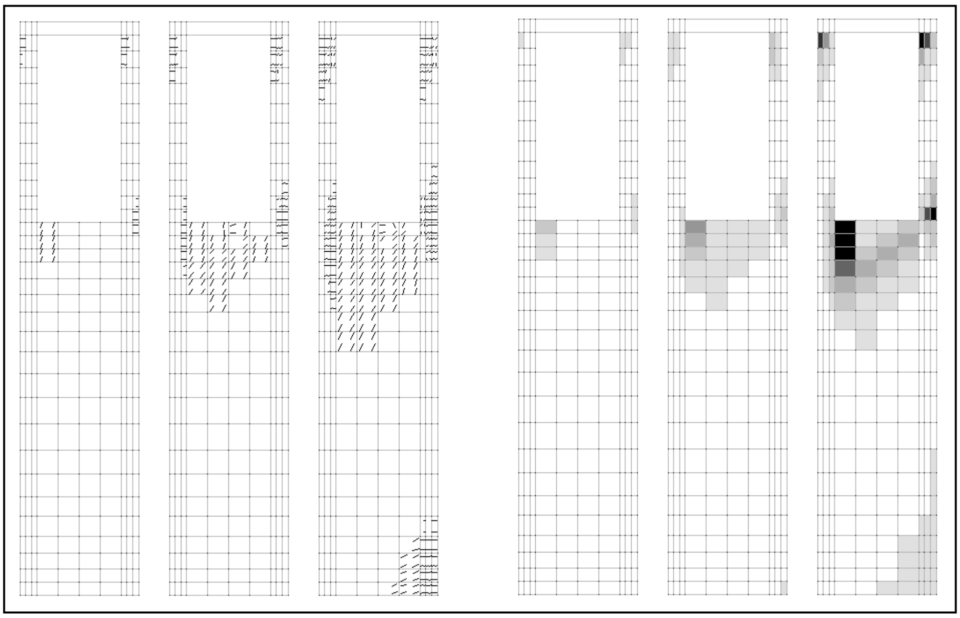

Figure 3. Cracks orientation and damage when the horizontal load is 50, 75 and 100% of the ultimate load.

3 MATERIAL AND DETERIORATION MODELING

In the following part, the authors want investigate the structural behavior of the pier with $\alpha = 1 / 3$ when a horizontal section of the structure is damaged by a generic deterioration. Scope of this study is to put in evidence the influence of the position of the deteriorated section on the structural behavior. To reach this goal, one has introduced in the model a portion of the pier deteriorated having *4 m* of height.

The concrete constitutive laws (equation 2, 3 and 8), presented in the second chapter, are governed by the material strength f'_c and f_{cr} (compression and traction law respectively) while the steel behavior (equation 14) is governed by the yielding value f_y. To modeling the deteriorated portion, one has assigned a deterioration factor δ_c and δ_s a the material, to have these initial mechanical properties:

$$\begin{cases} \overline{f}'_c = f'_c \cdot (1-\delta_c) \\ \overline{f}_s = f_s \cdot (1-\delta_s) \end{cases} \tag{16}$$

in these investigations one has assumed $\delta_c = \delta_s = 0.3$. In order to consider the uncertainties related to the actual concrete, *f'c* is assumed having a probabilistic distribution. The distribution modeling the characteristic strength value *f'c* is a lognormal; such choice is supported by physical consideration of the quantities take in consideration and on the theoretical structure of the hazard rate related to the distribution function adopted (Garavaglia et al. 2004). Since the uncertainty connected with the *f'c* value depends on a lot of environmental aspects (human errors, manufacturing process, design procedure, etc.) a random distribution of variability along the piers height and into its sections has been assumed.

Therefore, in order to study the structural sensitivity at the deterioration, nineteen deteriorated sections at different quota were investigated. For each deteriorated quota, 30 different piers were defined having a different spatial distribution of the mechanical properties

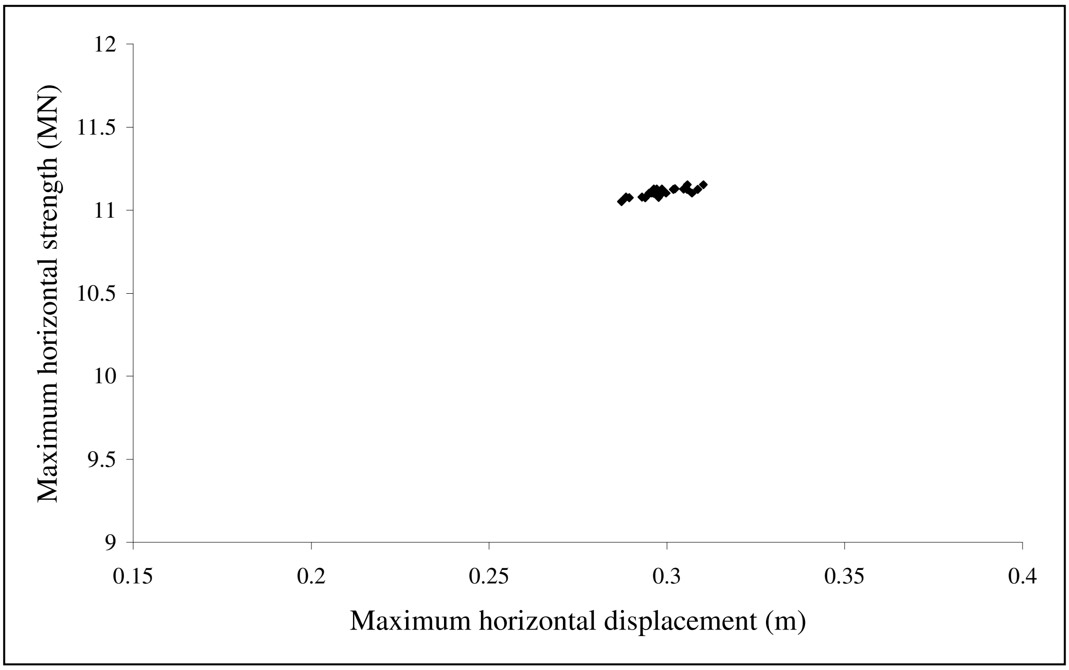

Figure 4. Characteristic points in the case of structure without initial deterioration.

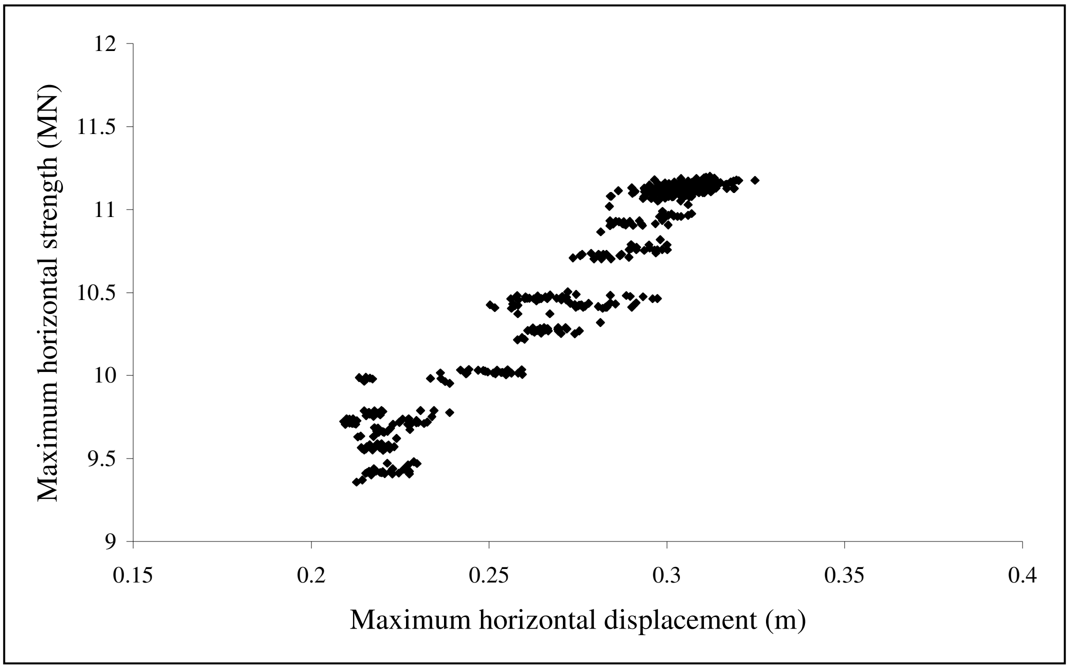

Figure 5. Characteristic points for all the analyses performed.

The piers were loaded with incremental forces at the top in the perpendicular direction of the blade planes. To characterize the ultimate behavior of the piers, one has considered the maximum horizontal force and the relative displacement just before the collapse of the structure. The Figure 4 shows the couple of these values for the group of the *30* not deteriorated piers. The expected maximum horizontal strength is *11.1 MN* while the expected maximum horizontal displacement is *0.3 m.* In the same way, Figure 5 summarizes all performed analyses. The comparison between the Figure 4 and the Figure 5 indicates that the assumed deterioration had a heavy effect in terms of ultimate strength and ultimate displacement.

It is possible to note that both the maximum displacement and the maximum strength depend from the position of the initial damaged zone. The evaluations show a maximum reduction of *15%* in terms of maximum strength and *30%* in terms of ductility. However, for different position of the degraded zone, the pier gives a different response. Clearly, close to the change of the section is the zone most sensitive to an initial damage. One can define a reduction factor like:

$$r = \frac{\langle f \rangle}{\langle f_{und} \rangle} \tag{17}$$

where $\langle f \rangle$ is the expected value of f greatness, for the *30* evaluations performed, and $\langle f_{und} \rangle$ is the expected value of f for the *30* not degraded piers. In this way, the reduction factor r is *1* if the pier is not damaged, and assumes values among *0* and *1* in the other cases. Evaluating the reduction factor for the maximum strength and for the maximum displacement to the different position of the deteriorated portion considered in the analyses, it is possible to plot the Figure 6.

In this figure, the left curve represent the reduction factor of the displacement while the right curve is the reduction factor of the strength. It is possible to note that the reduction factor increase quickly when the deteriorated portion is close to the section (*). It is possible divide the pier in two zones. The zone A) in which the structural behavior is strongly influenced from deterioration and the zone B) less sensitive.

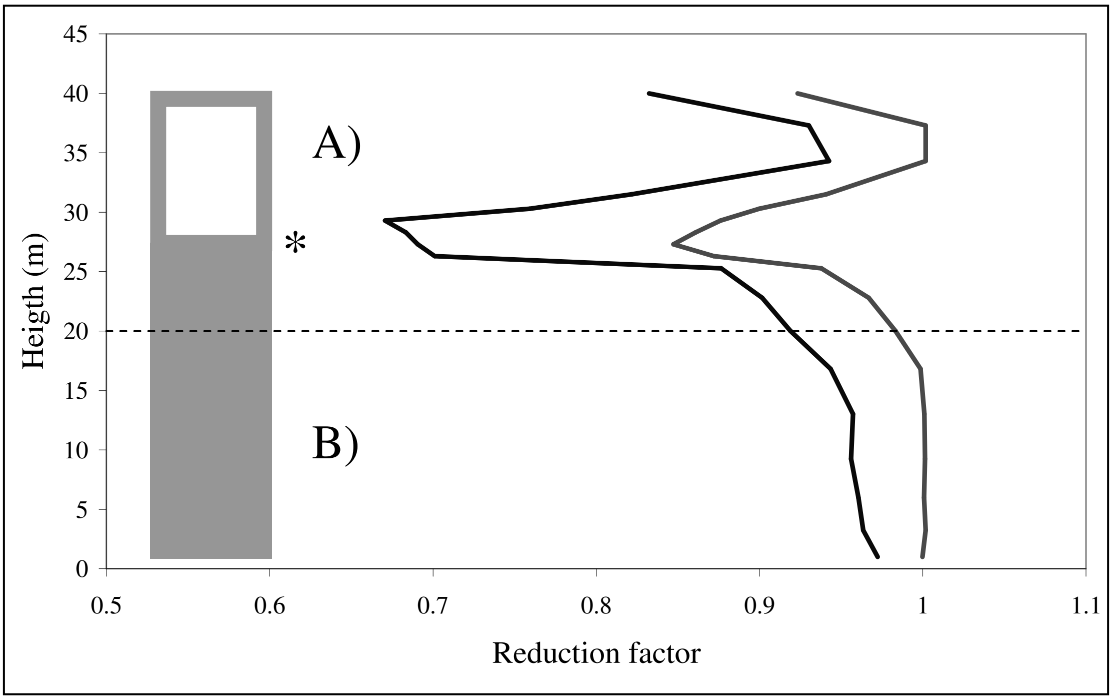

Figure 6. Influence of the deterioration on the ultimate behavior of the pier. On the left, the curve of the displacement reduction factor; on the right, the curve of the strength reduction factor.

CONCLUSIONS

A modern conceptual design of a bridge structure should be open to wider criteria that assure that the structure is endowed with static, dynamic and ductile characteristics sufficient to face the seismic events. The double-blade pier is a structure that can be designed to answer to the requisites of ductility demanded from the problem. The nonlinear behavior of this structure is not simple because is strongly influenced from the shear mechanisms; therefore, the evaluation of the strength and the ductility is an important problem.

The authors show how the behavior of this structure is influenced from a localized deterioration. Many investigations were performed to find the section most sensitive. The analyses show that the region close to the change of the section is the most sensitive and, for an initial deterioration of *30%* the ductility decrease of the same value while the ultimate strength decrease of *15%*. Deterioration in the bottom zone of the pier has not relevant effect on the nonlinear behavior of the structure.

ACKNOWLEDGMENTS

Thanks to Professor D.M. Frangopol for his useful discussions on this study. The financial supports of University of Rome "La Sapienza" and COFIN2004 are acknowledged. Nevertheless, the opinions and the results presented here are responsibility of the Authors and cannot be assumed to reflect the ones of University of Rome "La Sapienza".

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