Safety format for non-linear analysis

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Summary

In the paper is discussed and applied a code-like proposal for the definition of safety format in non-linear analysis of concrete structures. The proposal regards structures composed by linear elements as well as structures described by means of 2D or 3D finite elements. Model uncertainties, both on actions and resistance side, currently defined in design codes, are taken into account and a methodological approach to their influence is also proposed. Two numerical applications are given to cover both the over-proportional and under-proportional structural behaviour.

Keywords: Safety, Non-linear, Analysis, Model Uncertainties

1. Introduction

Nowadays the definition of a safety format for non-linear analysis of concrete structures, consistent with the semiprobabilistic approach to the structural safety, is considered a task that cannot be deferred. In fact, the quick development of non-linear procedures and computational tools able to analyze concrete structures with a more and more realistic approach, substantially enlarged the application field of non-linear analysis. Now it's available not only for the safety evaluation both, of existing structures and of structures designed with more traditional procedures, but also for the interactive design of new structures. Procedures able to update continuously the structural mathematical model, in function of predefined performance serviceability and ultimate criteria, are now on hand. But it is self evident that such refined procedures cannot be used in the design without a safety format consistent with the overall semiprobabilistic safety frame, in which structures are designed. In fact, without safety format the non-linear analysis would be banished to the numerical interpretation of experimental tests.

Recently [1] a consistent safety format for N.L. analysis has been presented, able to cover both types of non-linearity (geometrical and mechanical one) in any kind of concrete structures. Such proposal, that can be considered the finalization of recent researches within CEB [2], [3], [4], [5], [6], has been accepted too for the Stage 34 for Eurocode 1992-2 [7]. In the following this proposal will be briefly discussed and then applied to two significant cases, both involving a vectorial safety verification: a bridge pier affected by mechanical and geometrical non-linearity and a continuous deep beam in which only mechanical non-linearity should be considered.

2. Safety format

The safety format for non-linear analysis may be expressed as

$$\gamma_{Rd} S\left(\gamma_G G + \gamma_Q Q\right) \leq R\left(\frac{q_{ud}}{\gamma_{gl}}\right) \qquad (1)$$

or

$$\gamma_{Sd}\gamma_{Rd} S\left(\gamma_g G + \gamma_q Q\right) \leq R\left(\frac{q_{ud}}{\gamma_{gl}}\right) \qquad (2)$$

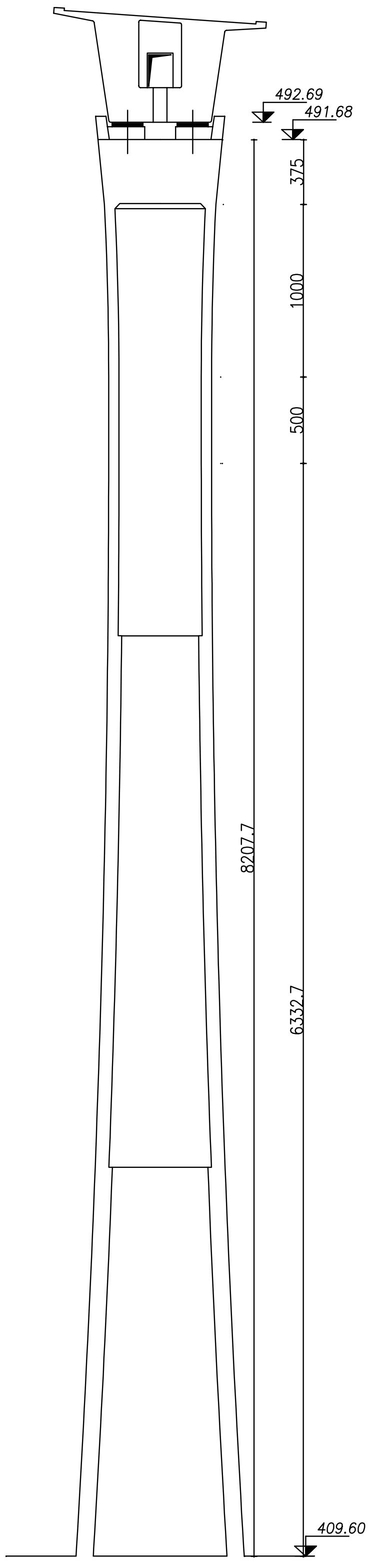

Fig. 1 – Bridge pear geometry

where:

γ_G, γ_Q are the global safety coefficients respectively for permanent and variable loads

γ_{Rd} is the model uncertainties coefficient on the resistance side (suggested value $\gamma_{Rd} = 1.06$)

γ_{Sd} is the model uncertainties coefficient on the action side (suggested value $\gamma_{Sd} = 1.15$)

γ_{gl} is the global structural safety factor (suggested value $\gamma_{gl} = 1.20$, but $\gamma_{Rd} \cdot \gamma_{gl} = 1.27 = \gamma_{Gl}$)

q_{ud} is the ultimate level of the internal actions path, reached in the incremental process of N.L. analysis.

In case no model uncertainties should be considered inequalities (1) and (2) are modified into

$$S(\gamma_G G + \gamma_Q Q) \leq R\left(\frac{q_{ud}}{\gamma_{Gl}}\right) \tag{3}$$

where the global safety factor γ_{Gl} assume the value 1.27.

For the description of mathematical structural model steel behavior is identified by means of mean mechanical properties, that is $f_{ym} = 1.1 \cdot f_{yk}$ and $f_{pm} = 1.1 \cdot f_{pk}$; for concrete the Sargin σ-ε relationship should be used, with $f_c = f_{ck} \cdot 1.1 \cdot 1.15 / 1.5 \cong 0.84 f_{ck}$.

For the evaluation of q_{ud} the analysis should be stopped when the ultimate strength and corresponding deformation is reached within the most critical region and the whole structure is unable to support any further load increment.

The linearization procedure described by inequalities (1) and (2) should be performed directly on the safety domain corresponding to the internal actions evaluated within the most critical region of the structure; for that purpose the procedure outlined in [1] should be applied.

3. Non linear analysis of bridge piers

The proposed approach in now applied to an highway boxed bridge pier with variable section, wall thickness and reinforcement along its depth of 82 m (fig. 1) characterized by $f_{ck} = 29\,\text{MPa}$ and $f_{yk} = 430\,\text{MPa}$. The ultimate combination of top actions coming by the design leads to: axial force $N_{Sd} = 83028\,\text{kN}$, transverse bending moment $M_{Sd} = 46384\,\text{kNm}$, longitudinal horizontal force $H_{Sd} = 2491\,\text{kN}$; the considered unforeseen eccentricity is 0.41 m. The critical section is located at a distance of 53.3 m from the foundation top. The ultimate internal actions in the critical section are, after the application of safety format (fig. 2) $\overline{N}_{Rd} = 143313\,\text{kN}$ and $\overline{M}_{Rd} = 225647\,\text{kNm}$, and the corresponding external actions at the top of the pier are $N_{Rd} = 125997\,\text{kN}$ and $M_{Rd} = 70389\,\text{kNm}$.

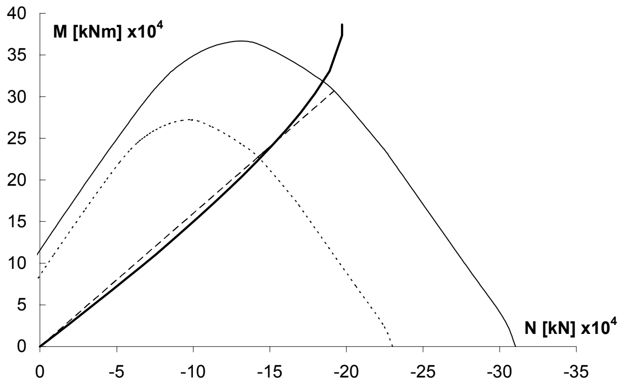

Fig. 2 – Internal actions path and safety verification of critical section

Then, by comparison with the corresponding internal action coming by the combination, the safety level is clearly demonstrated.

The same pier has been further analyzed increasing the length of the constant section region (initially 5 m) to 10/15/20/25 m and maintaining unchanged all the other parameters; the relevant internal actions paths and the corresponding application of safety format are pictured in fig. 3. The resisting parameters of critical section are listed in table 1.

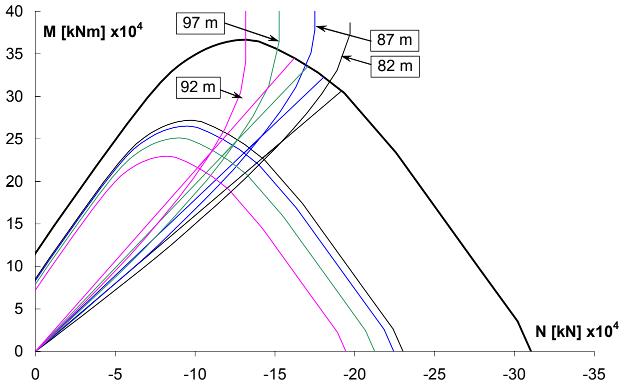

Fig. 3 – Internal actions and safety verification of different critical sections

Pier depth [m]	Critical section [m]	$\overline{N}_{Rd}$ [kN]	$\overline{M}_{Rd}$ [kN m]	N_{Rd} [kN]	M_{Rd} [kN m]
87.0	53.3	131928.0	231322.0	114225.0	63812.0
92.0	53.3	118348.0	227838.0	100933.0	56386.0
97.0	53.3	102962.0	214645.0	86515.0	48332.0
102.0	43.1	92229.0	229535.0	73430.0	41022.0

Table 1 – Comparison of resisting parameters for different pier depth

The effect of application of model uncertainties safety coefficient γ_{Rd} to different levels ($1 \le \gamma_{Rd} \le 1.12$) has been exploited on a series of squat piers characterized by a variable slenderness within the range $40 \le \lambda \le 140$, subjected to the same external actions to the top: axial force of $G_k = 10000\,\text{kN}$ and $Q_k = 2500\,\text{kN}$ and longitudinal horizontal force in less inertia direction, $H_k = 400\,\text{kN}$; fig. 4 pictures the internal action path for the bottom section, the critical one, of the different piers. The results of the analysis including the λ variability and the effect of longitudinal stress level ($\sigma_m = N_k / A_k$) in the range $2.0 \le \sigma_m \le 6.0\,\text{MPa}$ are pictured in fig. 5 in which the percentage variation of N_{max} with different γ_{Rd} values as a function of λ and σ_m is pictured. It appears clearly that the increase in the bearing capacity with the application of suggested γ_{Rd} values is in the range 1.4÷4.0% and its variation is characterized by absence of irregularities.

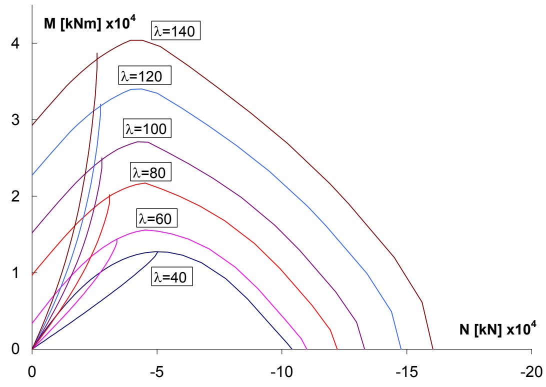

Fig. 4 – Internal action path for bottom sections

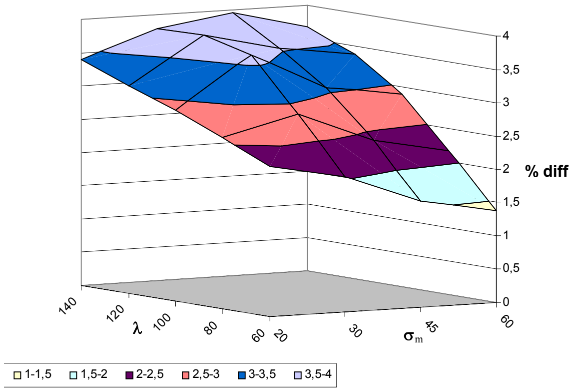

Fig. 5 – Resisting axial force variation as a function of σ_m and λ

4. Non linear analysis of a continuous deep beam

For this second case, a symmetric, 2 spans, continuous deep beam experimentally tested has been considered [8], in particular the beam identified in the paper as 3/1.5T1 has been analyzed.

Non linear analysis has been performed using ADINA non linear program implemented for the definition of concrete strength [9], [10]; fig. 6 pictures the mesh of half beam and the critical elements that governed the behaviour. In the same figure the load-displacement relationship for the mid span node, A, is pictured, in comparison with the experimental one.

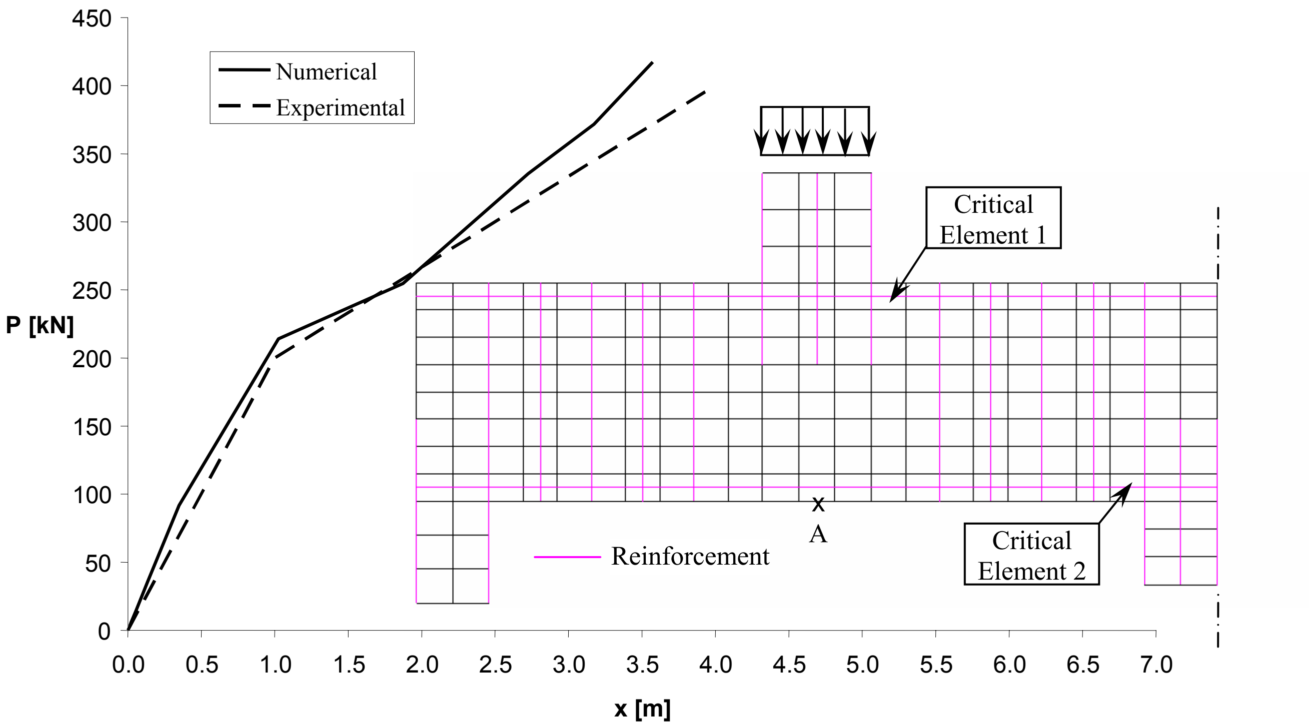

Fig. 6 – Deep beam half mesh and load-displacement behaviour of point A

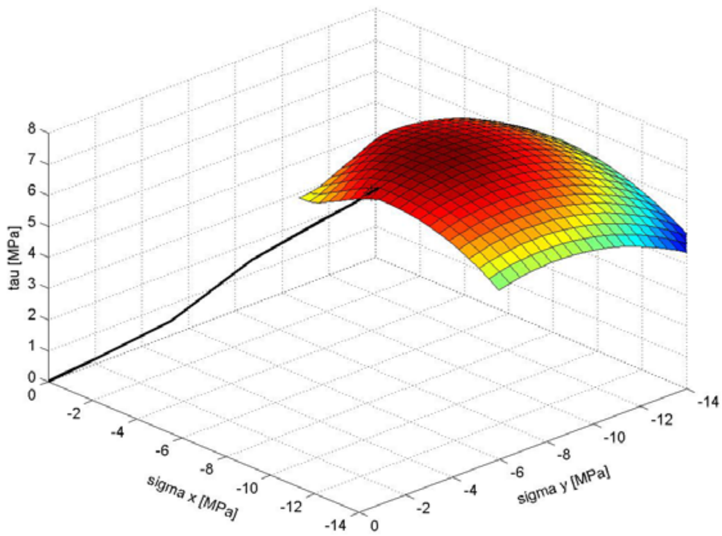

Fig. 7 – Resisting interaction surface and stress path

During the analysis initially crushed the first critical element, but the structure was able to carry further load increments up to the crushing of the second one, in which case the model was unable to reach the equilibrium for further load increments. This last step has been considered as the final point of the internal actions path. Fig. 7 pictures the resisting interaction surface σ_x, σ_y, τ for the critical second element, drawn in agreement with [10], and the internal action path in the same element, up to the intersection with the resisting surface. Fig. 8 illustrates the procedure for the application of safety format in a vectorial combination of internal actions and, by means of definition of a safety interaction surface, derived by the limit one by a linear transformation referred to the axes origin.

In practice, in agreement with the procedure outlined in [1] and [7], by application of equation (1), the following steps have been performed:

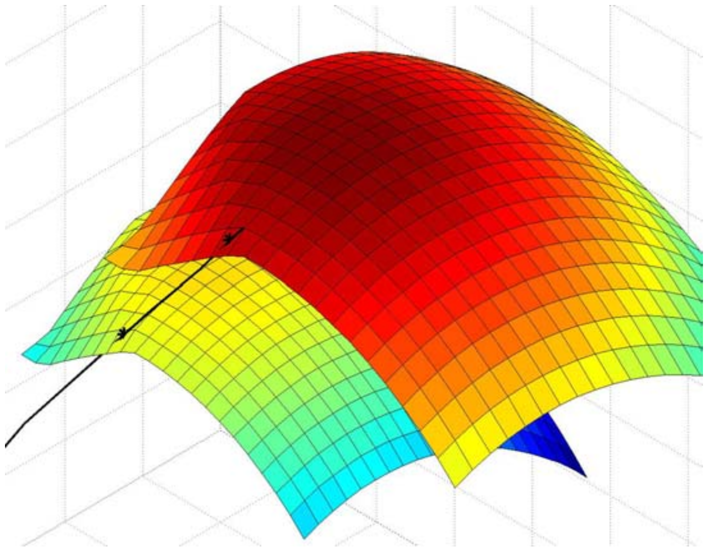

Fig. 8 – Application of safety format in vectorial combination of internal actions

- individuation along the internal action path of internal action set corresponding to q_{ud}/γ_{gl} ;
- linearization of this set with respect to the origin dividing it by γ_{Rd};
- definition by homotethy of the interaction surface passing trough the new linearized set of internal actions;
- determination of intersection of internal action path with the second interaction surface and then of the limit set of internal action corresponding to the required safety level.

For the case considered are found the following sets of internal actions and corresponding values of applied load (q_{ud} and q_{max}):

- $q_{ud} = 404.0\ \mathrm{kN}$, $\sigma_x = \text{-}8.47\ \mathrm{MPa}$, $\sigma_y = \text{-}5.77\ \mathrm{MPa}$, $\tau = 6.99\ \mathrm{MPa}$
- $q_{max} = 319.5\ \mathrm{kN}$, $\sigma_x = \text{-}6.82\ \mathrm{MPa}$, $\sigma_y = \text{-}4.75\ \mathrm{MPa}$, $\tau = 5.69\ \mathrm{MPa}$

Can be remarked that in this case, due to limited non-linearity in the internal action path of the critical second element, the ratio q_{ud}/q_{max} (1.26) practically coincides with the corresponding one evaluable with $\gamma_{Rd} = 1.0$ and $\gamma_{Gl} = 1.27$.

5. Conclusion

In the paper some applications of safety format for non-linear analysis of concrete structures are presented, putting in evidence the effect of model uncertainties as a function of slenderness and mean value of longitudinal stress level in bridges piers. The application of the procedure to finite element non-linear analysis clearly put in evidence the necessity to define the final point of the incremental action process, which, by engineering judgment and in agreement with experimental results, has been proposed to be considered as the last load level beyond which the structure mathematical model is unable to reach the convergence in presence of any load increment.

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