Estimation of the Residual Capability of Existing Buildings Subjected to Re-conversion of Use: a Non linear Approach

H. Albertini Neto
Department of Structural Engineering, Politecnico di Milano, Milan, Italy

L. Sgambi
Department of Structural and Geotechnical Engineering, Università di Roma "La Sapienza, Rome, Italy

E. Garavaglia
Department of Structural Engineering, Politecnico di Milano, Milan, Italy

ABSTRACT: The building rehabilitation needs particular attention. It depends on a lot of factors: socio-economic, urban and engineering factors. By structural point of view the rehabilitation of an existing building needs a structural approach different by the approach usually adopted for the project of a new building, in fact the parameters involved in the problem often suffer of uncertainty that cannot be forgotten. Following this way, the possible re-conversion of an existing building is here analysed and to take in account the uncertainties involved in the analysis a non-linear approach has been adopted.

1 INTRODUCTION

The building rehabilitation needs particular attention. The first aspect involved in the building rehabilitation problem is the choice of the re-conversion of use (by industrial use to residential, commercial or public use). In this decision different aspects play an important role: social/cultural aspects, land-planning and urban-planning aspects, structural and technological aspects, etc.; in a set of possible choices, these aspects can lead to prefer a solution to any other. To make a correct choice, a complete analysis of the urban contest and of its current and future requirements is important, but the "best" choice of re-conversion cannot be based only on this analysis: it must, also, involve a correct structural analysis.

The changing of the original use of a building can involve a strong changing of the load history and, as consequence, of the residence and reliability of the whole structural system. It is clear that, a reliability analysis of the existing and future situations cannot be neglected. It can suggest the "best" rehabilitation action between those proposed (the "best" obtained by the analysis of the contest).

In an existing building the reliability analysis is not simple. Usually the structural behaviour of an existing building is not linear and the parameters involved in it change by case to case, therefore they are not univocally defined and, often, present a random behaviour.

By the previous observation emerged that in a project of rehabilitation of existing buildings the structural reliability analysis must be based on a non-deterministic approach.

In this paper the possible re-conversion of use of an existing building is investigate with attention at its residual bearing capability. The evaluation of the residual bearing capability has required the survey of the current deterioration level of the building and the formulation of hypotheses about the current performance and structural capability of the building.

2 STRUCTURAL REHABILITATION OF R.C. EXISTING BUILDING. A NON LINEAR APPROACH

In the analysis of an existing RC building the uncertainties can involve several quantities like the load histories, the geometry of the structure, the quality of connections. For structural systems having non-linear behaviour, a realistic description of the response under all load levels can be obtained only by taking this non-linearity into account.

In this context, thought the reliability of the structure as resulting from a general and comprehensive examination of all its possible failure modes, one must pay attention to the following four aspects which define the assessment process:

- Available data.
- Modelling of the uncertainties.
- Non-linear structural analysis.
- Synthesis of the results.

The data needed to operate a structural rehabilitation of R.C. existing buildings are: the structural project tables, the deterioration level reached by the system: position of the damage and its entity, and news about the new destination.

To check the deterioration level reached, important is a robust monitoring action able to give safety answers with few points of measurement.

To model the structure like in origin leads to know, in deterministic way, the performance of the system when it was "new". Starting by this modelling, the uncertainties can be introduced.

2.1 *Structural analysis of R.C. building*

In most cases the R.C. structure should be analysed by taking material and geometrical non-linearity into account and their performance should generally be described with reference to a specified set of limit states as regards both serviceability and ultimate conditions (Bontempi et al. 1998, Biondini et al. 2001).

2.1.1 *Limit states*

Splitting cracks and considerable creep effects may occur if the compression stresses in concrete are too high. Besides, excessive stresses either in reinforcing steel can lead to unacceptable crack patterns. Excessive displacements **s** may also involve loss of serviceability. They have to be limited within assigned bounds $\mathbf{s}^-$ and $\mathbf{s}^+$. Based on these considerations, the following limitations account for adequate durability at the serviceability stage (*Serviceability Limit States*):

$$\textbf{1.}\ -\sigma_c \le \alpha_c f_c \qquad \textbf{2.}\ |\sigma_s| \le \alpha_s f_{sy} \qquad \textbf{3.}\ -\mathbf{s}^- \le \mathbf{s} \le \mathbf{s}^+ \tag{1}$$

where α_c and α_s are suitable reduction factors of the strengths f_c and f_s.

When the strain in concrete ε_c or the reinforcing steel ε_s reach the limit values ε_{cu} and ε_{su} respectively, the collapse of the corresponding cross-section occurs. However, the collapse of a single cross-section doesn't necessarily lead to the collapse of the whole structure, the latter is caused by the loss of equilibrium arising when the reactions r requested for the loads f can no longer be developed. So the following ultimate conditions have to be verified (*Ultimate Limit States*).

$$\textbf{1.}\ -\varepsilon_c \le -\varepsilon_{cu} \qquad \textbf{2.}\ |\varepsilon_s| \le \varepsilon_{su} \qquad \textbf{3.}\ \mathbf{f} \le \mathbf{r} \tag{2}$$

Since these limit states refer to internal quantities of the system, a check of the structural performance through a non-linear analysis needs to be carried out at the load level. To this aim, it is useful to assume $\mathbf{f} = \mathbf{g} + \lambda\mathbf{q}$, where **g** is a vector of dead loads and **q** a vector of live loads whose intensity varies proportionally to a unique multiplier $\lambda \ge 0$. With this position, the safe domains at both serviceability and ultimate states and the previous limit conditions can be synthetically formulated as follows:

$$\lambda \le \lambda_S = \max_{\lambda \in S} \lambda$$

$$S = \left\{ \lambda \mid -\sigma_c \le -\alpha_c f_c,\ |\sigma_s| \le \alpha_s f_{sy},\ \mathbf{s}^- \le \mathbf{s} \le \mathbf{s}^+ \right\} \tag{3}$$

$$\lambda \le \lambda_U = \max_{\lambda \in U} \lambda$$

$$U = \left\{ \lambda \mid -\varepsilon_c \le -\varepsilon_{cu},\ |\varepsilon_s| \le \varepsilon_{su},\ \mathbf{f} \le \mathbf{r} \right\} \tag{4}$$

being λ_S and λ_U the limit multipliers which define the failure loads.

2.2 *Structural model and non linear analysis*

2.2.1 *The R.C. Beam Element*

The R.C. structure here presented is modeled using R.C. beam finite element whose formulation, based the Bernoulli-Navier hypothesis, deals with such kinds of nonlinearities (Bontempi et all. 1995, Malerba 1998) (Fig. 1). In particular, both material $\mathbf{K}'_M$ and geometrical $\mathbf{K}'_G$ contributes to the element stiffness matrix $\mathbf{K}'$ and to the nodal force vector $\mathbf{f}'$, equivalent to the applied loads $\mathbf{f}'_0$, are derived by applying the principle of virtual displacements and evaluated by numerical integration over the length l of the beam:

$$\begin{aligned} &\mathbf{K}' = \mathbf{K}'_M + \mathbf{K}'_G && \mathbf{f}' = \int_0^l \mathbf{N}^T \mathbf{f}'_0 dx \\ &\mathbf{K}'_M = \int_0^l \mathbf{B}^T \mathbf{H} \mathbf{B} dx && \mathbf{K}'_G = \int_0^l N \mathbf{G}^T \mathbf{G} dx \end{aligned} \tag{5}$$

$$\begin{aligned} &\mathbf{N} = \begin{bmatrix} \mathbf{N}_a & \mathbf{0} \\ \mathbf{0} & \mathbf{N}_b \end{bmatrix} \\ &\mathbf{B} = \begin{bmatrix} \partial \mathbf{N}_a / \partial x & \mathbf{0} \\ \mathbf{0} & \partial^2 \mathbf{N}_b / \partial x^2 \end{bmatrix} \qquad \mathbf{G} = \begin{bmatrix} \mathbf{0} \,\Big|\, \dfrac{\partial \mathbf{N}_b}{\partial x} \end{bmatrix} \end{aligned} \tag{6}$$

where N is the axial force and **N** is a matrix of axial $\mathbf{N}_a$ and bending $\mathbf{N}_b$ displacement functions. In the following, the shape functions of a linear elastic beam element having uniform cross-sectional

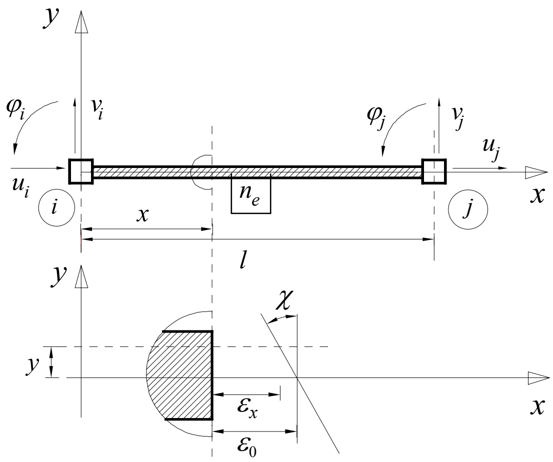


Figure 1. Bernoulli-Navier finite element.

stiffness **H** and loaded only at its ends adopted (Przemieniecki 1968). However, due to material non-linearity, the cross-sectional stiffness distribution along the beam is non uniform even for prismatic members with uniform reinforcement. Thus, the matrix **H** has to be computed for each section by integration over the area of composite element or by assembling contributes of concrete and steel. In this way, after the constitutive laws of the materials are specified, the matrix **H** of each section can be computed under all load levels. The equilibrium conditions of the beam element are derived from the principle of virtual work. Thus, by assembling the stiffness matrix **K** and the vectors of the nodal forces **f** with reference to global co-ordinate system, equilibrium can be formally expressed by **Ks**=**f**, where s is the vector of the nodal displacements. It is worth noting that the vectors **f** and **s** have to considered as total or incremental quantities depending on the nature of the stiffness matrix **K**=**K**(**s**), or if a secant or, a tangent formulation is adopted.

2.2.2 *Materials properties*

The stress-strain diagram of the concrete is described by the Saenz's law in compression without strength in tension (Fig. 2a). By assuming $E_{c0} = 9500 f_c^{1/3}$ [MPa] the diagram is completely defined by the strain limits $\varepsilon_{c1}, \varepsilon_{cu}$, and the compression strength f_c.

The stress-strain diagrams of the reinforcing steel is defined by the elastic modulus $E_s = f_{sy}/\varepsilon_{sy}$, the limit strains ε_{su}, and the yield strengths f_{sy} (Fig. 2b) (Bontempi et al. 1998).

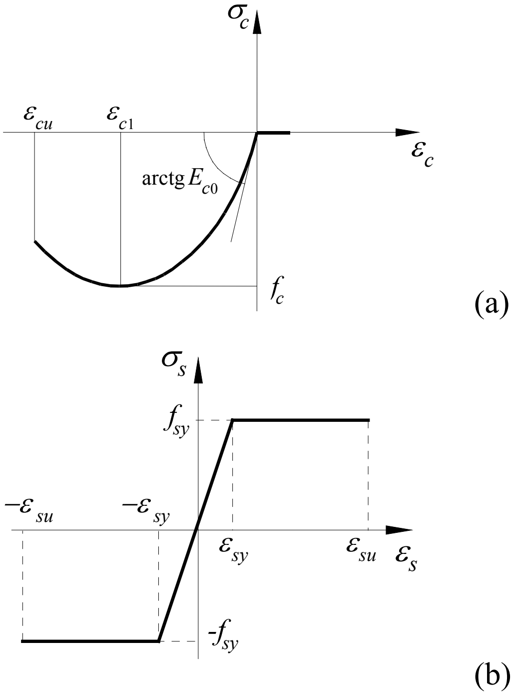


Figure 2. Stress-strain diagrams of the materials: (a) concrete; b) reinforcement.

3 APPLICATION

The presented procedure is applied at the rehabilitation analysis of the existing building shown in Figure 3 (Milia et al. 2001, Albertini 2002).

Figure 3. Fossano Building (ex Molteni) external view.

The building was built in Cantù, Milan (Italy), in 1955. Since 1989, year of its abandon, it was the Molteni's furniture farm. Now it is under preservation of work as example of "industrial archeology", therefore it cannot be pulled down. In the current City Urban Plan (CUP) the rehabilitation of this building is considered. Different solutions are proposed in it, each of them have to be evaluated under different points of view, not last the structural compatibility of the choice. One of the solutions proposed in the CUP is the re-conversion of the building in district library. This solution it here analysed.

3.1 *Geometry of the building*

The building is constitutes by four floors (Fig. 3).

The plan shows an "L" form (Fig. 4). In Table 1 the principal dimensions are reported.

The structure is a R.C. frame with R.C. beams-and waffle slabs. Instead, a R.C. cylindrical shell (Fig. 5) constitutes the roof.

Though a visual analysis the damage level reached by the structural elements has been recorded (Fig. 6). Of course, more detailed analysis are necessary, but already this simple, not invasive and not expensive analysis is able to give useful information. In our case it has put in evidence that the relevant

Figure 4. Fossano Building: ground level plan (Milia et al. 2001).

Table 1. Building geometry.

Building Dimensions		
Transversal width		12.4m
Length	On Via G. Fossano	47m
	On Via Torre	19m
Floor high-ness	Ground floor	5.0m
	1st floor	3.5m
	2nd floor	3.5m
	3rd floor	3.0m
Inter-column span	On the fronts	4.7m
	transversally	varying

damage is the carbonation of concrete with the consequent reduction of the resistant section of beams and columns and the reinforcing steel damage; this can seriously compromises the bearing capability of the whole structure or of a part of it. The results of this analysis have been considered in the modeling has uncertainties.

3.2 *The case study: linear and non linear analysis*

The analysis of residual capability of an existing building cans be dealt in different ways:

- *Linear analysis* with controls on conformity with structural design code prescriptions.
- *Non linear analysis* with search of ultimate limit state level.
- *Non linear fuzzy analysis* able to take in account the uncertain involved in the problem (Biondini et al. 2002, Sgambi 2003).

All these analyses will require different information; of course, as output, they will give different answers more or less suitable.

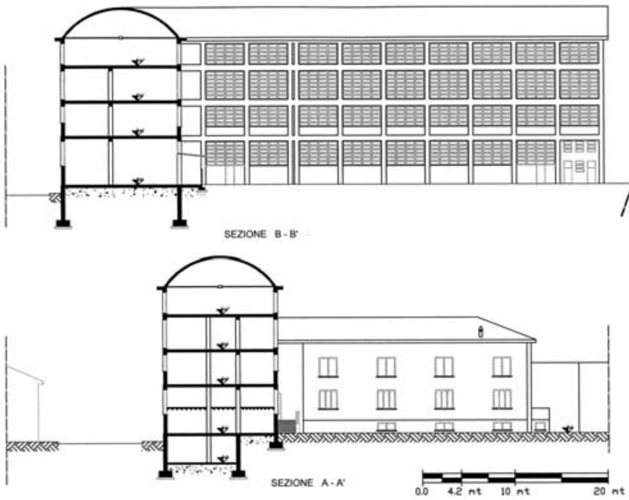
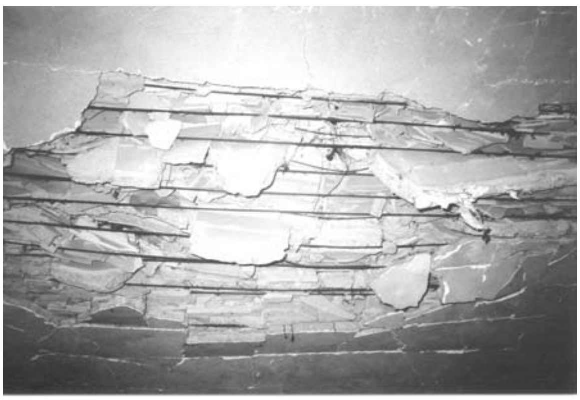


Figure 5. Building sections (Milia et al. 2001).

Facade: particular of the structural joint

Columns



Example of deck decay

Volt decay

Figure 6. Present state of deterioration.

3.2.1 *Some code prescription*

Following the Italian structural code prescriptions (D.M. 09/01/1996) concerning the ultimate limit state conditions, the expression $\mathbf{f} = \mathbf{g} + \lambda\mathbf{q}$, could be written in the following terms:

$$\mathbf{f} = \lambda_g\, \mathbf{g} + \lambda_q\, \mathbf{q}, \tag{7}$$

where **g** and **q** identify the dead load and live load respectively. For the re-conversion of use proposed for the building in study, **q** is assumed to be major than 6 kN/m^2. Following the code prescription also these values are assumed:

- The materials safe coefficients:
  $\gamma_{cls} = 1.6$ for concrete; $\lambda_{seel} = 1.15$ for reinforced steel (of course, this will modify the constitutive laws of materials described in § 2.2).
- The dead load multiplier:
  $\lambda_g = 1.4$

In non-linear analysis, the live load multiplier λ_q has been assumed variable: it increases until the value connected with the collapse of the structural system (or a structural element). Therefore, the ultimate limit load of $\lambda_q\mathbf{q}$ represents the maximum live load admitted for the structure.

If the multiplier λ_q obtained by the analysis is:

$$\lambda_q \geq \overline{\lambda}_q \tag{8}$$

where $\overline{\lambda}_q = 1.5$ is the live load multiplier prescribed by Italian structural code, the building can be considered adequate for the re-conversion of use proposed.

3.2.2 *Structural system modeling*

The structural analysis of an existing building start by its structural project tables; for the building investigate this step has not been possible, therefore, the structure has been re-projected. The re-project has been made using the measurements made on site and following the Italian structural code in use at the age of constructio. As live load, the original live load as been assumed. Assuming the materials properties reported in Table 2 and supposing simple reinforced beams the results obtained seem to be in accordance with the geometry of elements and structural code prescriptions (Tab. 3) (Fig. 7). The structural system obtained has been assumed as the "real" structural system. On it, through linear and non-linear analyses, the possible re-conversion has been investigated. The structural computer code used has been SAP2000 based on the finite element theory in the evaluation of strains and displacements.

3.2.3 *Linear analysis*

The first step of analysis has been the evaluation of the strengths level in damaged structure due to the loads connected with the re-conversion of use proposed. This can be made through the evaluation of internal forces of the damaged structure due to actual loads.

This kind of analysis, based on a deterministic linear approach, is not able to give complete response on the real residual strength capability of the damaged building. In the case study proposed, this analysis has been putted in evidence that in some damaged elements of the building the structural code prescriptions were not respected, but about the last load level (failure load) no indication are given. Therefore a non-linear analysis is needed.

Table 2. Material properties.

Material properties	
Concrete characteristic strength	R_{ck} = 25 Mpa
Steel type	FeB32k
Young modulo	E_c = 25.000MPa E_s = 210.000MPa

Table 3. Structural elements geometry.

Structural Element	Real size b(cm)	Real size H(cm)	Resulting size b(cm)	Resulting size H(cm)
Boundary roof beams	40	45	40	43,5
" 3rd foor beams	40	50	40	43.5
" 2nd foor beams	40	45	40	36.8
" 1st foor beams	40	45	40	39.6
" ground foor beams	40	30	40	23.2
Beams	40	83	40	63.9
Beams	60	83	60	53.9
Beams	40	60	40	29.0
Beams	40	60	40	49.6
	L(cm)	L(cm)		
Columns (3rd foor)	30	40	Verified for differnt: $\mu = A_s/A_c$	
Columns (2nd foor)	40	40		
Columns (1st foor)	40	50	A_s=steel area	
Columns (grou. foor)	50	60	A_s=concrete area	

3.2.4 *Non-linear analysis*

Considering the non-linear materials laws described in § 2.2 and the live load limit at collapse defined in § 2.1, the non-linear analysis is applied, where the load multiplier λ_q is assumed varying between zero until the value connected with the collapse of the system. In Figure 8, 9 and 10 the results concerning the different values of λ_q are reported.

The analysis made shows that the first plastic hinges appear with a λ_q = 1.3 (Fig. 8), but they do not lead at total or partial collapse of the structure. Therefore, the non-linear analysis continues and other plastic hinges appear (Fig. 9). The procedure finish when in a structural element the ultimate strength level is reached: in our case it is happen for a column with λ_q=2.5 (Fig. 10).

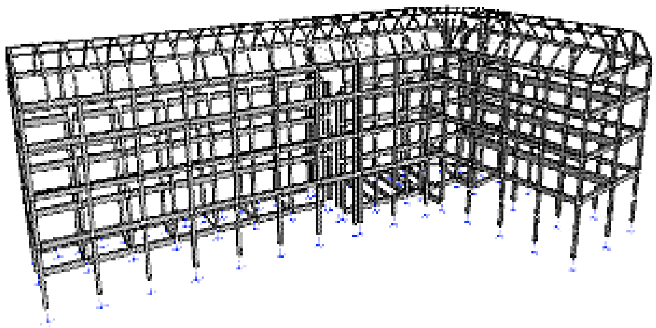


Figure 7. Fossano Building: Structural schema.

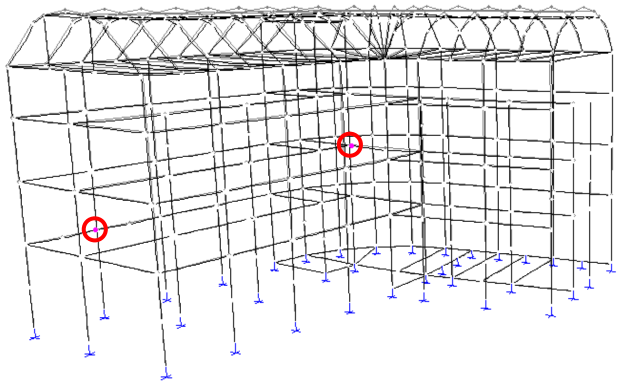


Figure 8. $\lambda_q = 1.3$: formation of the first plastic hinges.

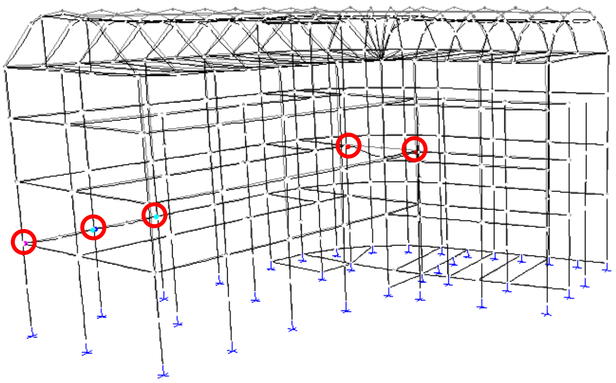


Figure 9. $\lambda_q = 1.8$: formation of other plastic hinges.

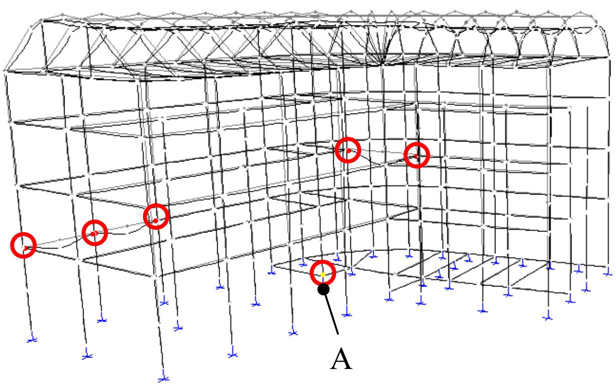


Figure 10. $\lambda_q = 2.5$: collapse of column A.

4 CONCLUSION

In an existing building the re-conversion of use cannot be approached only by an architectonic point of view. In fact, a choice, correct by the architectonic point of view, can be "not suitable" by the structural point of view; it can require the rehabilitation of structural system.

In this paper the re-conversion of an existing building has been analysed by the structural point of view. Since this problem involves a lot of uncertainties, a non-linear approach, able to take into account some of these uncertainties, has been here proposed.

The results obtained show that, for the new destination of use proposed, the building seems in suitable conditions. However, by Figure 6 appear that, on volt and waffle slabs serious works of maintenance are needed. So that the first plastic hinges could appear for a load level greater than that prescribed by the actual structural code, also the reinforcement of the connections shown in Figure 8 is required. The reinforcement of the column A (Fig. 10), cause of collapse, is suggested too. It is true that the collapse seems to happen for a load multiplier greater than the multiplier prescribed by the actual law, however, a failure of a structural element is a dangerous event that it is always better to prevent.

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