DIAGNOSIS, DISCOVERY AND SEISMIC STRENGTHENING OF A MILITARY HISTORIC BARRACKS OF 20TH CENTURY in Italy The " CASERMA PIAVE" of ORVIETO

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Abstract

The military barracks complex of Orvieto (of around 200.000 mc) is a particularly representative example of the buildings realized with mixed r.c. and masonry structure built in the years twenty-thirty of last century in Italy. These buildings were constructed on the base of technical norms not sufficiently detailed that they allowed the realization of structural elements having often remarkable congenital defects and unacceptable from the actual norms, especially if they are placed in seismic zones. In the present study some conflict aspects concerning the old and actual technical norms are presented, arising during the structural survey and restoration project of the PIAVE barracks.

Key words: diagnosis, mixed structures of Thirteen years, restoration, seismic upgrading.

Introduction

In the last decade a strong regional and national debate concerning the re-use and restoration of Piave barracks was developed. The disuse for many years and the insufficient ordinary and extraordinary maintenance have rendered the recovery much problematic one

The building, constructed between 1932-35, covers in plant approximately 1/10 of the area of the Orvieto downtown constructed on the cliff (Figs 1-4). Planned initially with structure in masonry, like the other buildings of the cliff, it has been subsequently modified in one mixed reinforced concrete and masonry structure, for problems connected to the foundation ground.

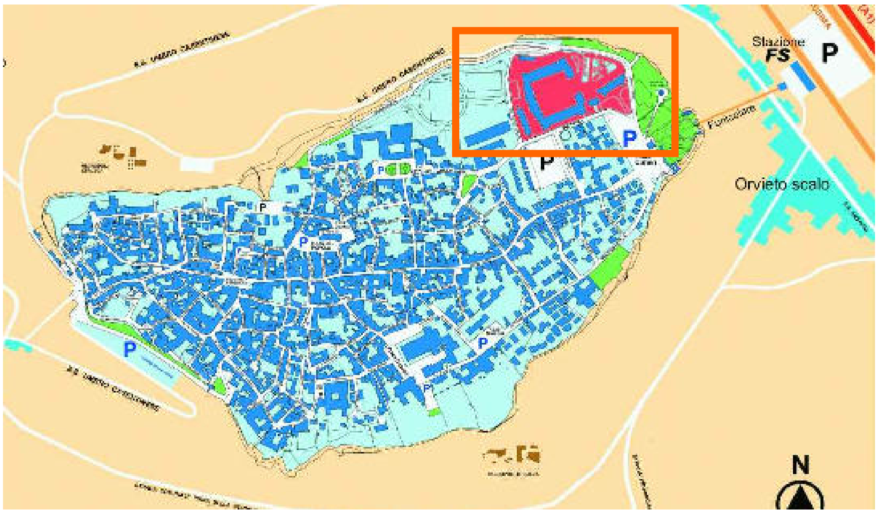

Figura.1– Location of ex Piave barracks, Orvieto, Italy

Fig. 2 PIAVE Barracks of Orvieto. Photo of Archive

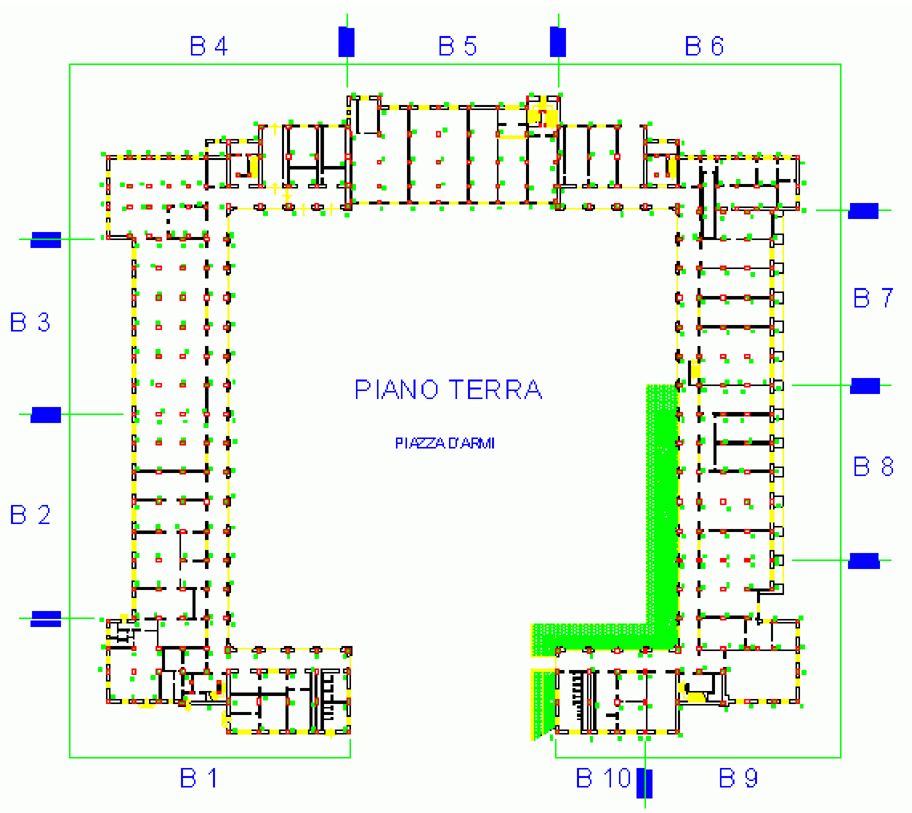

Fig. 3 ex PIAVE Barracks of Orvieto Plant of ground floor *(Survey of N. Avramidou technical office)*

In occasion of the lead diagnostic studies from the author on assignment of the Public Administration of Orvieto on the ex Barracks Piave it has been possible to develop to comparative analyses between the enforced technical norms to the age of the realization of the structure and the more recent norms and in particular of the D.M. 16/01/1996 and Ord. 32742003.
From the examinations lead on the r.c. columns of the building defects attributable to the bad constipation of the concrete emerge, to the lack of employment of spacers of the metallic reinforcements during execution, to the wrong dosage of inert granules and to the defective and insufficient concrete covering.
The framed r.c. structures realized in the years twenty- thirty of the passed century in Italy were constructed with the passed technical requirements and normative specifications that were less restrictive of that applied currently. These buildings can resist to static loads but they do not respond adequately to seismic action effects).

In the present study they are compared the results obtained applying both the previous (R.D.L. 1213 del 29/07/1933) and those recent norms, in the static and seismic analysis

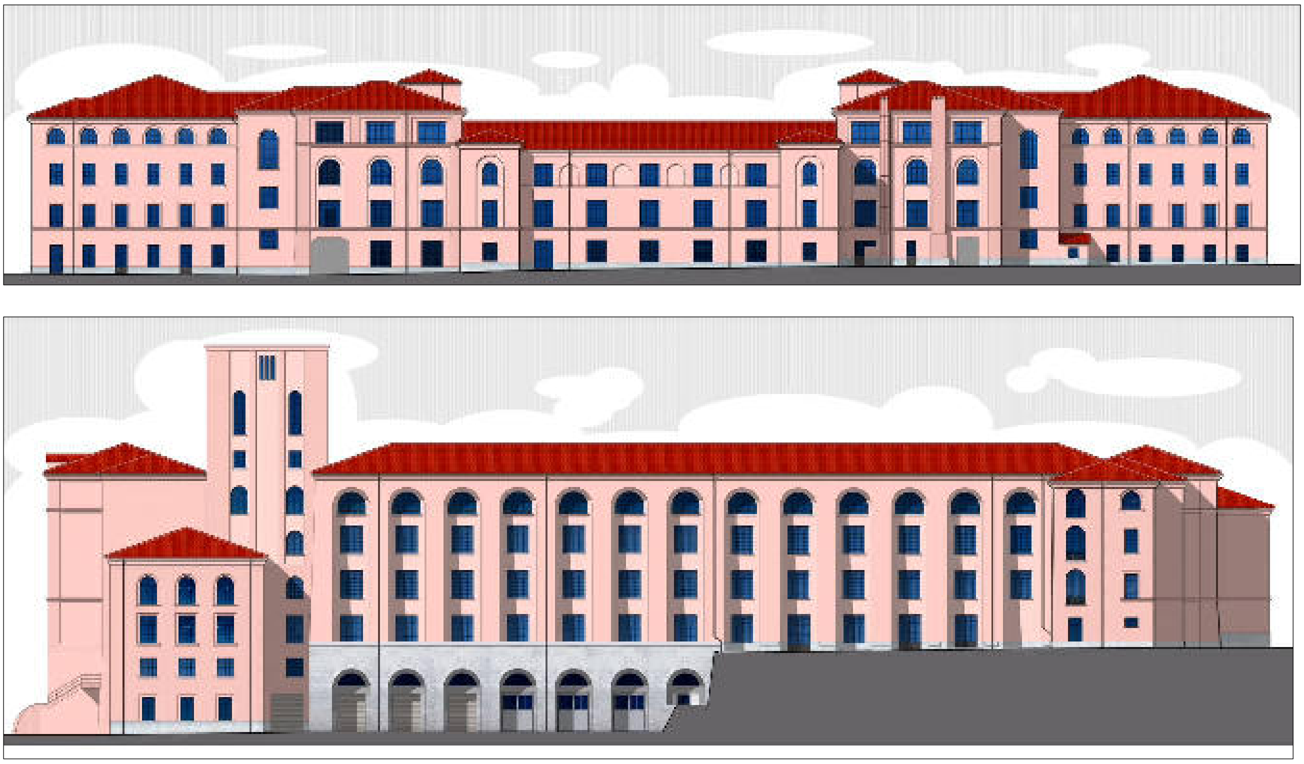

Fig. 4 PIAVE Barracks of Orvieto. The Principal facades. *(Survey of N. Avramidou technical office)*

Brief description of the structure

The building is divided in 10 understructures interconnected with technical joints. The analyzed sector (B3) has a structural modulus of approximately 4,5 m x 6 m (Fig. 3-5) and floor levels from 4,5 m until 5,3 m.

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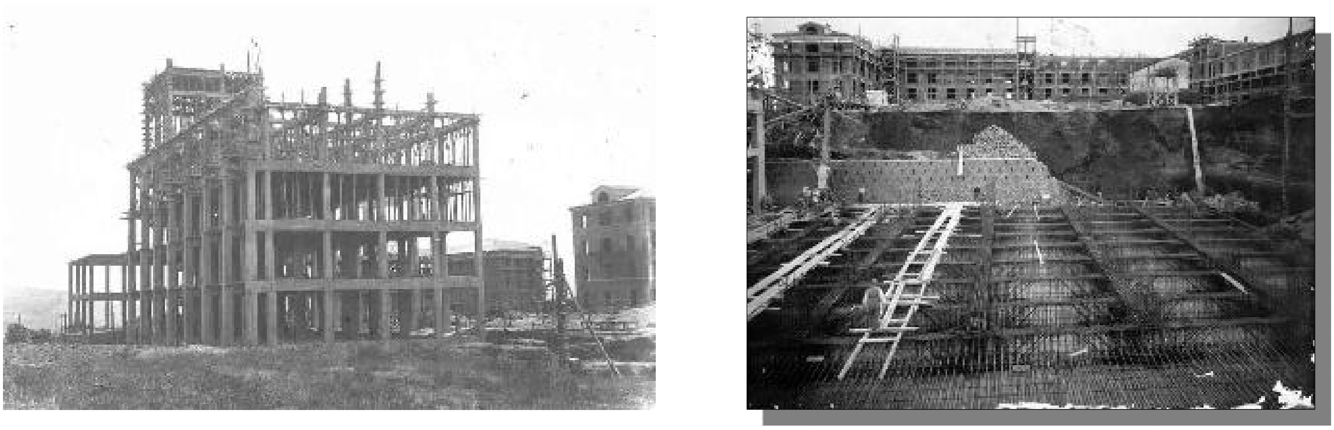

Fig. 5. Photos of archives illustrated the r.c. frame and the inadequancy of the soil foundation

The internal spaces of the building they are all as open spaces realized with r.c. columns, while the external part is a mixed r.c and masonry structure. The r.c. beams are of variable height and the floors are realized with reinforce concrete plates. All bodies are founded on r.c. continuous beams (Fig 5).
The roof covering has a wood structure anchored on the below r.c. frame.

Survey of structural damages

The ordinary maintenance interventions which were previously carried out they only considered the restoration of the damaged concrete at the bottom of the columns of the ground floor, without replacing the corroded metallic bars. The corrosion of these last ones is diffused in all the columns and has reached a loss of 100% of the original section of the metallic bars.

Fig.   6. Corrosion of the steel reinforcement at the bottom of the columns, sector B3

Fig. 7 Ground floor. B3 body. Internal and external spaces . The external columns are inserted into masonry tuff stone

Estimation of the characteristic concrete compression strength, $R_{ck,}$ in agreement to the enforced norms to the age of construction

The tests carried out on extracted carrots from the structure of the B3 body (Figs 6-7) indicated one Rck rather low ; The issue is risen therefore spontaneous if were possible to estimate the Rck of a concrete in compliance with the norm of the years Thirty (R.D.L. 1213 del 29/07/1933) and if such concrete could be considered “structural” according the actual technical norms (puts into effect Rck ≥ 15 MPa).
In fact, the prescriptions contained in the above cited R.D.L. do not indicate the minimum strength value for cast in place concrete(on the contrary of the present norm that only concrete with Rck ≥ 15 15 Mpa indicate as "structural" one). instead, the minimal value for the quality of the concrete was demanded, which had to be certified directly from the producer with tests that had to be made on cement mortar champions.

In particular, the cubic compression strength of the cement mortar (after 28 days cast in place) had to be, according to the R.D.L. 1213 of the 29/07/1933, at least 450 kg/cm2 (for concretes not of high resistance). Lacking of data concerning the correlation between the strength obtained with tests on "normal" cement mortar and tests on concrete, it is possible to resort to some considerations:
A concrete in compliance with the R.D.L 1213 could be imagined as a cement mortar to which it has been added large inert ; its compression strength, to relationship parity a/c, would not have to be smaller of that one of the mortar. But what we can say of the resistance to varying of the relationship Water/cement? Supposing, with caution, for the cement mortar standardized a relationship w/c = 3 and for the concrete a relationship w/c = 5, on the base of diagrams in literature, it is possible to deduce that a reduction of the relationship w/c of a concrete from 0,5 to 0,3 is reflected in an increase of compression strength (to 28 days) of approximately 84%, and therefore the concrete used in the Former Piave Barracks, manufactured in compliance with the R.D.L. 1213 of the 29/07/1933, must have a compression strength not much minor of = $\frac{450 \cdot kgf / cm^2}{1{,}84}$ = 245 kgf/cm^2. It seems without doubts that the concrete of the Former Piave Barracks can be considered as “structural” concrete according to the D.M. 09/01/1996..

The safety factor of concrete design strength. Comparison between the *safety load* of the R.D.L. 832/1932 and the *admissible stresses* of the D.M. 09/01/1996

The study of the pre-existing technical norms has concurred to make an interesting observation with regard to the safety factor of the concrete design strength. This last value in the R.D.L. 832/1932 were called “safety load” and were defined equal to 1/4 of the “ultimate load”, in its turn defined like the average of the 3 turn out better results obtained on 4 cubic compression tests (to 28 days). The concrete design strength in the D.M. 09/01/1996 correspond to the “admissible stress” defined through well known formulae.
It can be interesting to compare, to parity of concrete, the two design strengthens. These values are not directly comparable because of the different initial data and methods of calculation used; so, on the base of some simplified hypotheses made it was possible to write up the diagram of fig. 8 in which the values of the “safety load” as well as of the “admissible stresses” are compared, to varying of the average strength of the concrete. The two values turn out to be very similar, and, moreover, repeating the procedure assuming a standard deviation δ = 4,56 MPa instead of δ = 5 Mpa, it is stated that the two values coincide perfectly. This result demonstrates the tight existing tie between the “safety load” of the R.D.L. 1213 of the 29/07/1933 and the “admissible stresses” of the D.M. 09/01/1996, in spite of the differences of the methods of calculation and of the definitions (in particular are observed that the “safety load” of the R.D.L. 832/1932 still do not have the formulation of present statistical formulation in the definition of the “admissible stresses”.

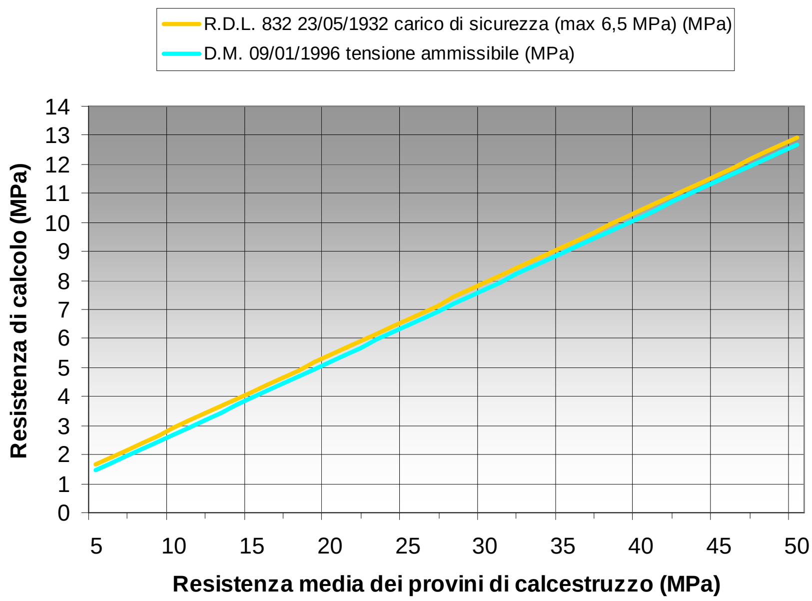

Fig. 8 Compression strength of the concrete carrots versus design strength according to two different normative

Reliability of the sclerometric survey

In reference to the norm UNI EN 12504-2 and the specifications regarding the appraisal of the uniformity of the mechanical characteristics of the concrete and the estimation of the strength by means of correlation with direct methods of test, it can be asserted that:

- The concrete has mechanical characteristics sufficiently uniforms between the various bodies of the PIAVE barracks, the plans, and inside of the same column; the maximum differences are approximately equal to the standard deviation **δ**;
- The **correlation** between sclerometric index and strength measured with compression tests on cylindrical carrots has turned out **nearly null;** the in site sclerometric tests cannot therefore be used in order to estimate the strength of the concrete in site, in the points where they have not been extracted carrots, Fig. 9.
- This result probably is attributable to the diffuse presence of gross inert, that they have influenced both the sclerometric index and the strength of carrots.
- The strength estimation of carrots, may be affected by the presence of large inert, but in opposite way (underestimated), if the diameter of the carrot it is insufficient.

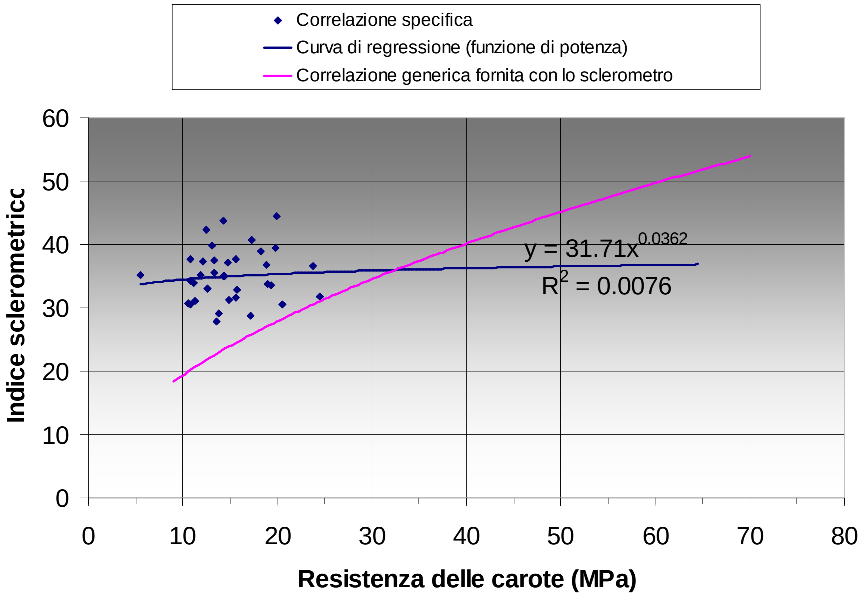

Fig. 9 correlation between scelrometric index and strength measured with compression tests on cylindrical carrots

Axial compression tests

The 79 carrots of Ø100 mm of diameter and 250 mm of length have been extracted from different points along the columns height. After the crushing test, the way of rupture of such carrots has been examined, in order to establish their acceptability, based on the indications of norm UNI EN 12390-3:2003[1], that it distinguishes the type of *"satisfactory"* and *"not satisfactory"* rupture of the concrete carrots (Fgs. 10-12a,b).

[1] UNI EN 12390-3:2003 del 01/08/2003, "Prova sul calcestruzzo indurito - Resistenza alla compressione dei provini"

Fig. 10 Concrete carrots from B3 columns

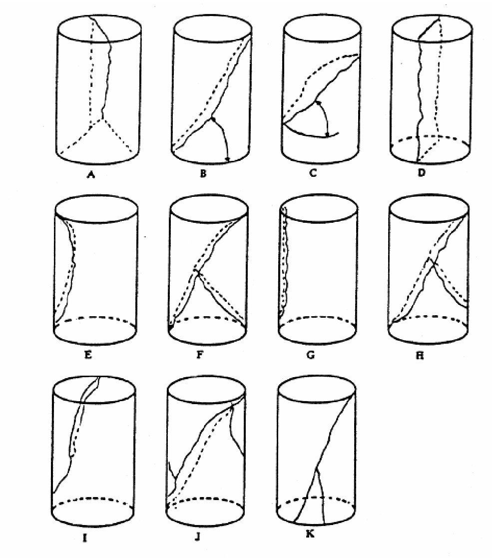

Fig. 11 Satisfactory rupture of the tests secondly the UNI EN 12390-3

The rupture of “satisfactory” type, according to of UNI EN 12390-3, is from generally attributing to the inert presence and disposition of the large ones, often of the maximum dimension around to 6-7 cm (not agreement therefore to the norms of the age that previewed the maximum value of the diameter of 5 cm).

Fig. 12a,b Example of rupture of type “not satisfactory” (on the left) and satisfactory (to right) according to of norm UNI EN 12390-3

The compressive strength measured in laboratory has been corrected based on the specific relationship height/diameter of concrete carrots, according the indications of British norm BS 1881: From the obtained cubical compressive strength therefore they are obtained the medium strength, the standard deviation, the coefficient of variation of the data. Based on these results, has been observed a rather elevated value of the variation coefficient, that it is from attributing itself to the low value of the medium resistance medium $R_{c\ media}$ rather than to the value of the standard deviation δ, that it turns out online with the usual values of dispersion of the data for the axial compression tests on concrete carrots[2].

The value relatively low of characteristic cubical strength (regarding that attending based on the norms of the age, of 25 MPa) is from attributing itself partially to the diffuse presence of inert large, the whose maximum dimension - like saying previously - are up 6-7 cm, that is greater one of 1:3 of the diameter of carrots. In fact in the norm UNI EN 12504-1 it is notice that “the relationship of the maximum dimension of the aggregate regarding the diameter of the carrot

[2] Da 2 a 7 MPa per provini confezionati con calcestruzzo fresco, maggiore per provini composti da carote di calcestruzzo indurito gettato in opera a causa delle condizioni di compattazione e stagionatura maggiormente variabili, e del disturbo subito dalle carote nelle fasi di estrazione e trasporto.

meaningfully influences the measured strength when is approached greater values of 1:3 approximately). It has been therefore carried out a prudential estimation of the effect negative on the which had resistance of the carrots to inert with greater diameter of 1:3 of the diameter of the carrot, extrapolating the data supplied based on the norm UNI EN 12504-1. The diagram of Figure 13 illustrates, joined from the continuous lines, the increments of the strength, regarding the carrot having 2 strength 5 mm of diameter, for several combinations of diameters of carrots and the maximum diameters of inert, on the base of the data supplied from cited norm UNI EN 12504-1. On line outlined, the increment, extrapolated graphically, of the to strength increasing of the diameter of the carrot it is illustrates. Without to enter in the details it can be asserted that it appears lawful and much prudential to preview that, if they had been used carrots of 200 mm of diameter, so as to have the maximum diameter of inert the not too much advanced one to 1/3 of the diameter of carrots, one would have been estimate strength Rck of carrots advanced at least 14% regarding that one measured with carrots of 100 mm of diameter.

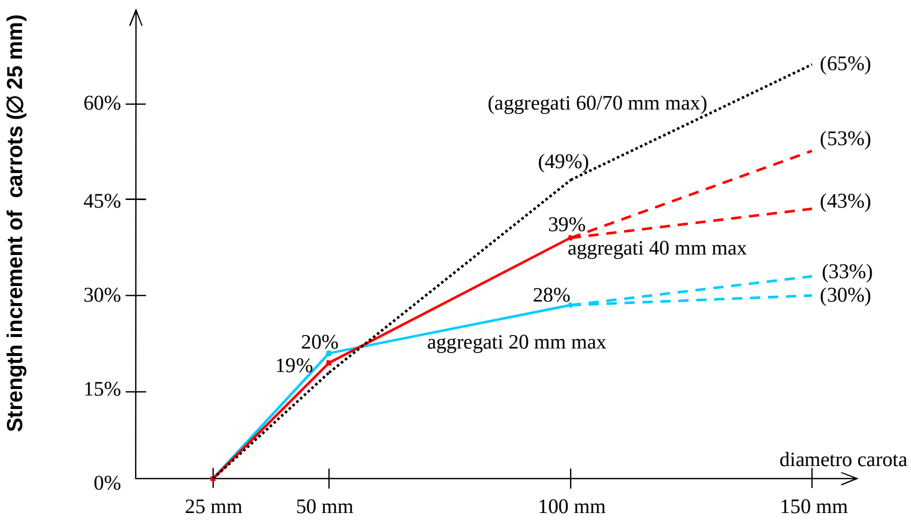

Fig. 13 Incidence of the dimension of the aggregate and the diameter of the carrot on its compression strength: hypothesis of extrapolation based on the data supplied from UNI EN 12504-1996

In order to obtain, from this value of Rck , the design strength (upgraded), Rck must be divided for a coefficient comprised between 0,65 and 0,85, like approval previously. Be a matter itself of a structure realized beyond 70 years ago, with relatively backward technologies of the concrete elaboration, seem lawful to adopt a coefficient of 0,70. The verifications to the Ultimate Limit State according to the D.M. 16/01/1996 show that, on the contrary of the verifications to the admissible stresses, the columns turn out all verified, and the not verified beams are essentially those secondary ones (Fig. 14). The verifications on the displacements turn out results analogous to those to the admissible stresses.

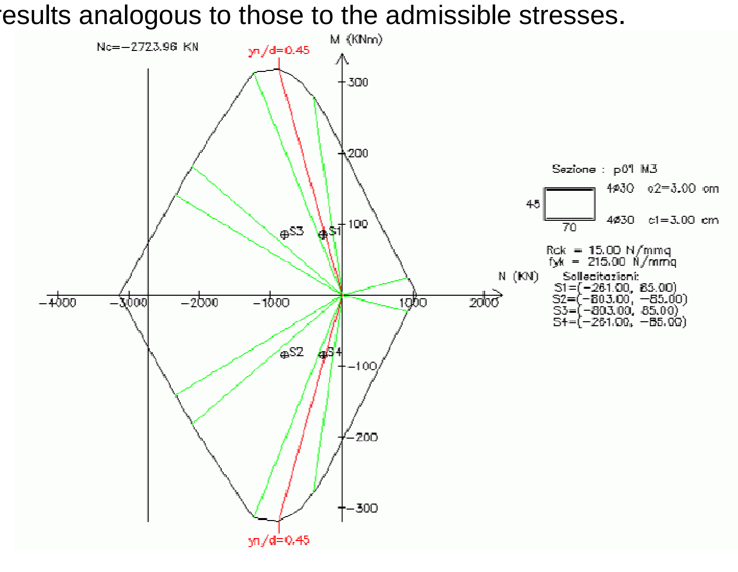

Fig 14   Dominion of rupture M-N of the section P01 type (column); the stresses deriving from the analysis (straight bending) are inner to the dominion

Some observations on the ductility and the level of safety

The reduction of the elastic spectrum in order to obtain the design spectrum according to the method of the ductility factor presupposes that the structure has a sufficient degree of ductility. The local and total ductility is in great part assured from the constructive details indicates from the norms (C.M. of the 10/04/1997). Since the constructive details indicate to the norms, in order to assure the ductility, generally is disregarded (for the absence of inner stirrups, fastenings, smooth bars, etc) persist many doubts on the implicit level of emergency in the verifications.

The results obtained with the State Limit of Collapse according the Ord. 3274/2003

The Ord. 3274 place two conditions for the applicability of linear dynamic analysis to the existing r.c. buildings: · **i) ρmax/ρmin < 2**; · **ii) ρmax < 7** for the columns and **ρmax < 15** for the beams.
ρ it indicates, for every structural element, the relationship between the bending moment supplied from the analysis and the resistant moment. The analysis evidences that the first condition of applicability (ρmax/ρmin < 2) for linear dynamic analysis does not turn out satisfied; in fact ρmax/ρmin = 59, more over of the suggested limit (2)from the norms. Also the second condition of applicability is not satisfied for the columns; in fact the ρmax of the pillars it exceeds the limit of 7, while the ρmax of the beams, Fig. 15, it is always inferior to 15 .

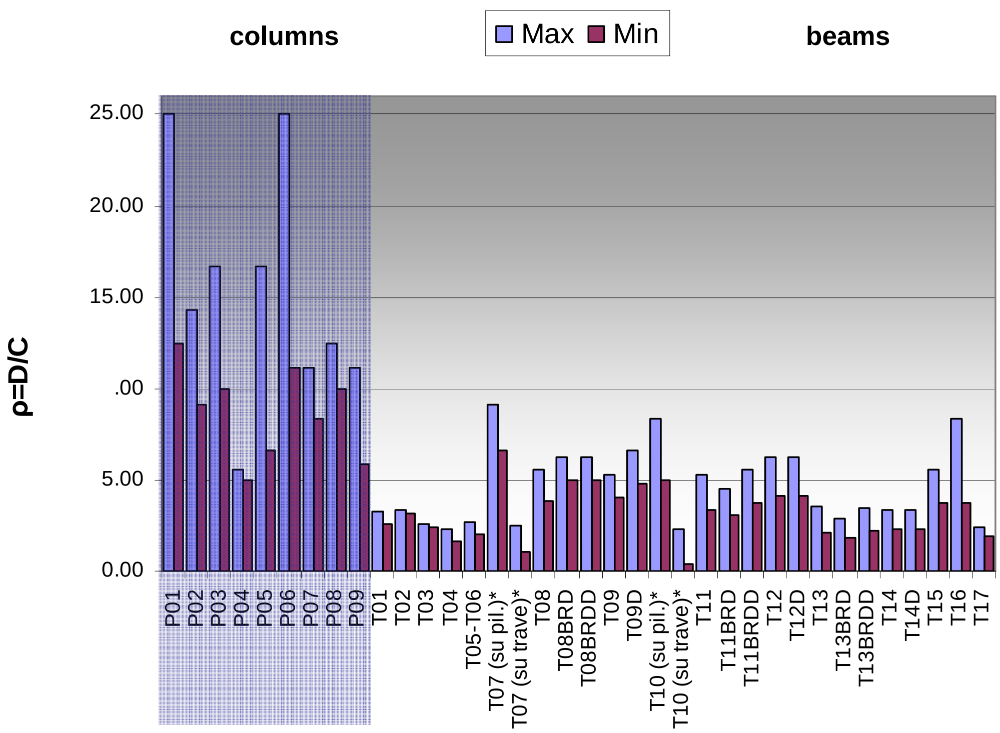

Fig 15 Values of ρ (calculated to constant axial stress) in the present state for all the elements of the structure grouped for type of section

From the comparison between the three methods of verification it is evident that the number of structural elements that do not satisfy the verifications to the Ultimate State of Collapse according the Ord. 3274/2003, 22%, are meaningfully more high of the number of structural elements that do not satisfy the verifications to the admissible stresses and the ULS, respective 5% and 7%, according to the D.M. 16/01/1996. The comparison between the stresses turning out from the analysis to the Limit State of Collapse according the Ord. 3274/2003 and to the ULS according to the D.M. 16/01/1996 evidence the great differences in the two cases (see diagram of Fig. 16); the stresses in the case of the Ord. 3274/2003 are much greater that in the case of the D.M. 16/01/1996. It does not, however, have to forget that the remarkable differences partially are compensated from the degree of ductility admitted from the Ord. 3274/2003 for beams and pillars.
On the diagrams of Figure the positions of the stresses (blue points) supplied from the analysis to the Limit State of Collapse according the Ord. 3274/2003 (up) and to the Ultimate Limit State according to the D.M.

16/01/1996 (low) in comparison to the dominion of rupture of the P4 section (column of the ground floor) are illustrated.

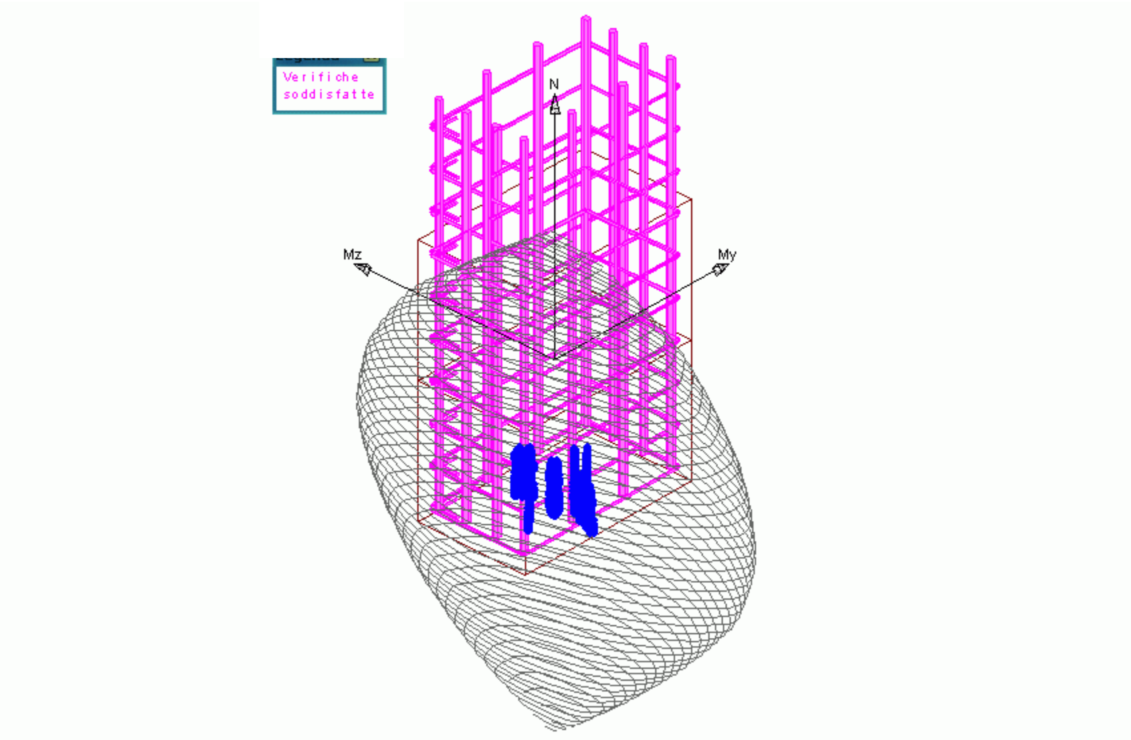

Fig. 16 Position of the stresses (blue points) supplied from the analysis to the Limit State of Collapse according the Ord. 3274/2003 (up) and to the Ultimate Limit State according to the D.M. 16/01/1996 (low) in comparison to the dominion of rupture of the P4 section (column of the ground floor)

Conclusion Remarks

The previewed structural verifications from the Ord. 3274/2003 for the existing buildings are more restrictive than those demands from the previous normative, although that the ductility factors admitted for the beams (15) and columns (7) appears generous.
The first condition of applicability of linear dynamic analysis for the existing buildings previewed from the Ord. 3274/2003 ($\rho max/\rho min < 2$) appear difficult to respect in rigorous way. That although the use of methods of no linear analysis, in the case of reinforce concrete with smooth bars, is hindered from the limited acquaintance of the deformability of the structural elements in not linear field.

It turns out of the diagnostic tests, also of those of destructive type, with justice considered more reliable, must be subordinates to one deepened critical analysis. The burdens, in terms of time for their elaboration, of such analysis are not to be underestimated.

The gravity of the damages finds on the columns of the ground floor is connected to its foundation earth that from the time of the construction it has already induced the designers to modify the original structure during execution, as well as to the diffuse presence of the ascending humidity that plagues all the architectonic patrimony of the city. The recovery works of the military barracks Piave of Orvieto cannot disjoined from the structural recovery to which it goes given absolute priority.

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