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Models and Assessment

Modèles et évaluation

Modellbildung und Beurteilung

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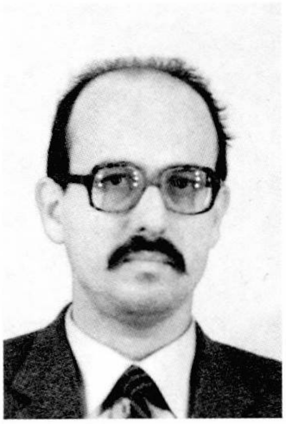

Alberto Peano born 1946, is a Dr. Sc. Civ. Eng. from Washington University, St. Louis - Missouri. He was one of the developers of the p-version of the finite element method. Later he applied FEM technology to solve research and application problems in a wide range of engineering fields including the rehabilitation of monumental structures.

SUMMARY

Difficulties to be overcome, conceptual or practical limits to be taken into account, computational strategies to be followed are reviewed. The selection of the most appropriate approach is illustrated, when useful for more clarity, by case histories related to Italian monuments analysed at ISMES.

RÉSUMÉ

On a examiné les difficultés qui doivent être supérées, les limites conceptuelles ou pratiques qui doivent être considérées, les stratégies de calcul qui doivent être suivies. Quand elle est utile pour une plus grande clarté, la sélection de l' approche plus adapte est illustrée par des cas historiques relatifs à des monuments italiens analysés chez ISMES.

ZUSAMMENFASSUNG

In diesem Beitrag werden die Verwicklungen, die präktische un konzeptuelle Begrenzungen, sowie di Rechenverfahrenstrategien, die bzw. zu üeberkommen, zu betrachten und zu befolgen sind, kritisch durchgesehen. Die Auswahl des zweckmässigsten Verfähenswird - wenn für die Klarheit nötig - mit früheren Sächfallen in Beziehung mit einigen von ISMES untermahten italienischen Monumenten erlauedert.

1. INTRODUCTION

Structural analysis of monumental structures faces a number of difficulties unusual in other areas of Civil Engineering: material properties are poorly determined due to limited access to samples for destructive testing and very often also to large property variation within the fabric of the same building; the strength of the connection of the different structural elements is often hard to assess, if at all possible; the original geometric configuration of the building might have been altered by inelastic settlements along the ages and cannot be exactly determined.

Due to the above main difficulties finite element modelling of historical buildings has been widely applied only recently. Surprisingly numerical computations are often motivated by the scarcity of data, which is simultaneously the main source of uncertainty on the quality of numerical results. In fact a very meaningful application of computational models is the interpretation of structural behaviour in order to check the consistency of the scarce data available. For instance this is achieved by calibration of some poorly known parameter of the model until some significant feature (e.g. opening of cracks, frequencies of first few vibration modes, etc.) is correctly reproduced in the numerical results.

This first phase of analysis ("model calibration" phase) is usually a mandatory step in order to assess the reliability or at least the plausibility of the computations aimed at assessing the safety margin of the structure either as it stands or after a strengthening intervention.

The second phase of analysis ("safety assessment phase") requires usually the rational management of at least two main sources of difficulty. The first one is the nonlinear behaviour exhibited by some structures before collapse. Numerical modelling of such behaviour is very difficult and it requires the assumption of additional material properties that usually play no role for model calibration. Unfortunately numerical modeling of nonlinear structural behaviour might be crucial if the linear elastic range is small and does not point out an adequate margin of safety of the structure. The second reason of concern is that the procedure to be followed in order to assess the safety of monumental structures is often not as clearly defined as in other areas of civil engineering, particularly when seismic loads are involved.

The purpose of the paper is to review the difficulties encountered at all stages of numerical modelling of monumental structures, such as: input data gathering and validation, selection of the computational approach, validation and interpretation of numerical results.
The selection of the most appropriate approach in each case will be illustrated, when useful for more clarity, by case hystories related to Italian monuments.

2. NUMERICAL VERSUS PHYSICAL MODELS. VALIDATION OF COMPUTATIONAL APPROACHES

The safety of monumental structures has been investigated also by testing physical models. These tests are usually carried out on reduced scale replicas under static or dynamic loading. A list of monuments investigated at ISMES by physical models is presented in table 1

MONUMENTS	SCALE	LOADING	YEAR
Mole Antonelliana (Torino, Italy) Partial Model	1:4	Static	1956
Duomo di Milano (Milano Cathefral Italy) - Tiburium model	1:15	Static	1967
Duomo di Milano - Models of main columns	1:4.7	Static	1967-81
Colonna Antonina (Rome, Italy) n.3 models	1:26 1:18 1:15	Dynamic	1988
Rotonda di Diana (Baia, Naples,Italy)	1:15	Dynamic	1990

Table 1.: Physical models of some Italian monuments

Of course physical models are often not competitive with numerical models for reasons of costs, time of execution and convenience of use (e.g. it is usually not possible to carry out parametric analyses). Moreover physical models have many drawbacks in common with computational models. Indeed, as mentioned before, when information on materials and geometry of the structure is scarce, it is difficult to define the specifications of any model.

Still there are cases where experimental analysis plays a very important role. Here the example of one static and one dynamic models may be mentioned. The main pillars of the tiburium of the Duomo of Milan deteriorated with ages and have been replaced after extensive investigations [1] Several physical models of one pillar in scale 1:4.7 have been made out of the same Candoglia marble used to build and repair the duomo throughout the centuries. The tests have been carried out under a static load representing the portion of the dead weight of the structure carried by the most stressed pillar.

The aim of the test was to prove that no excess settlement would be caused by the repair strategy adopted. Therefore one sector at a time of the scaled pillar was replaced. The main features of this physical model were:

- the use of the same material. This eliminates possible concerns about the accuracy of numerical modelling of fragile and other nonlinear material behaviours;

- the reproduction of the complete sequence of the operations. Really the only difference with the real intervention was the scale. This eliminates possible concerns about incompleteness of the models (have all possible effects and scenarios taken into account in the specifications of the model?).

Note that the writer, maybe because of his own scientific preferences, believes that appropriate results could have been obtained by numerical models in a cheaper and faster way. Still the uniqueness and importance of the monument, the tremendous consequences in case of a structural failure during strengthening, the need to provide decision makers with evidence robust both from the technical and psychological standpoint well justified the resort to physical modelling.

A second quite different example is the dynamic testing on a shaking table of a number of models of columns built according to different scale. The tests do not appropriately reproduce any specific monument because inertia forces are not correctly reproduced in scaled models made of the same material. Still the tests, carried out for ENEA, provided a set of data that are precious for validation of numerical models. Note that a number of papers have been presented recently on the dynamic behaviour of Greek and Roman temples or of other structures made of superimposed stone blocks.

The assumptions used in the proposed numerical methods are very reasonable, still the complexity of the dynamic behaviour of this kind of structures suggests that a combination of numerical and experimental methods should be used to clarify and validate the range of the computational models.

In conclusion the assessment of monumental structures is more and more carried out by means of numerical models. Under service loading the models are usually validated using monitoring or other experimental data of the real structure, as illustrated in the following section. In case extreme load conditions (such as earthquake) are analysed, the required validation of the methodological approach is difficult, therefore the level of approximation of such analyses should be carefully evaluated.

As data for back analysis of structural failures are scarce, destructive testing of physical models or of ancient buildings of no historical value should be used for validation of computer models.

3. MODEL CALIBRATION STUDIES

3.1 Aims and Technologies

Extensive numerical computations are usually carried out in combination with the experimental investigation of structural behaviour. As mentioned before, the aim is to check the consistency of the available data as well as of the assumptions that are necessary to build a numerical model.

Moreover the studies may be carried out in order to improve the postprocessing of the experimental tests or in order to derive data that cannot be directly measured (e.g. settlements through ages) [19,15,17,18].

Calibration of poorly known parameters is often based on parameter sensitivity analysis and on heuristic approaches. Nevertheless mathematical optimization techniques are also used and in such case it is more appropriate to define such analyses as "structural identification" processes rather than simple model calibration procedures.
Such "structural identification" can be based on direct or inverse procedures:

- in the direct approach a standard direct analysis procedure is iterated and at each step the free parameters are modified on the basis of a criterion to measure discrepancy between expected and effective response and of a convenient mathematical method (Symplex method, Rosenbrook method, alternate variables method, etc.);

- in the inverse approach equilibrium equations must be rewritten in such a way that the free parameters appear as independent variables. A number of different approaches have been proposed both for static and dynamic problems and in either deterministic or probabilistic procedures.

3.2 Input data treatment

Material properties are usually measured at a few isolated points where the flat jack test may be carried out or, exceptionally, samples may be extracted. In order to prepare a numerical model, however, appropriate values must be assigned to all other points in the structural model. Values can be selected in different ways:

- values measured at a specific point may be applied to a "tributary area" which is considered more or less homogeneous on the basis of visual inspection, historical information on the construction process or simplified experimental methods.

- values may be interpolated between measurement points. For example this is applicable in case discrepancies are ascribed to deterioration and the state of deterioration varies gradually within the structure;

- stochastic values (average values and statistical distribution) may be derived from the measured data set and applied uniformly.

Sometimes it may be convenient to solve an identification problem in order to extract from an experimental test the most reliable values of the needed structural parameters. For instance numerical simulation of the flat jack test was carried out and lead also to some improvement of the testing procedure [1]. The simulation, which also identified separate values for the elastic properties of bricks and mortar, should nowadays be revised and extended to take advantage of state of art knowledge on micromechanics of masonry.

3.3 Use of crack opening data

In many historical buildings cracks are too large to be ignored even though the lack of detailed information on structural material properties and the analytical complexity make unrealistic any attempt to model the cracking process [19,16,23,22,20]. In such cases the relative displacement of the two crack faces should be regarded as very meaningful information to be used for model calibration.

Probably the first case is which this approach was adopted is the analyses carried out in 1979 in view of the diagnostic and strengthening of the Palazzo della Ragione in Milano fig. 1.

This is a 13th century building which in the last two centuries housed the Notary archives of Milan [2]. The analyses before and after reinforcement were limited to the facade which exhibited severe cracking. The mesh was very detailed in order to model the main cracks as well as the reinforcement. (fig. 1). The aim of the analyses was to correlate the opening displacement of the main cracks with the settlement of the pillars. The analysis was carried out for unit settlements of each pillar, then a least square procedure identified the combination of settlements which minimizes the discrepancy between computed and measured crack opening displacements.
Note that this was the only possible way to identify the settlements and recover the stress distribution in the facade before reinforcement. The flat jack tests, used to identify the elastic properties of the masonry, also provided the stress values at a few control points, where the consistency of the procedure was checked.

Another important case history where the crack opening displacement plays a major role is the Brunelleschi dome in Florence. In the well known studies by Chiarugi, Fanelli and Giuseppetti [4] the main vertical cracks of the dome are correlated with the dead load while periodic opening / closing is associated with the periodic thermal loading due to seasonal temperature variations. More recently the crack opening displacement of Brunelleschi dome has been used by Chiostrini for identification of the most realistic value of the elastic module [19]. The analysis also leads to tensile stresses which are in agreement with the crack pattern. On the contrary the elastic module is not in agreement with that needed to fit the frequency of the first few eigenvalues of the dome.

The two examples mentioned above illustrate an approach (linearity of constitutive equations, explicit modelling of cracks) that is nowadays the most usual and well established.

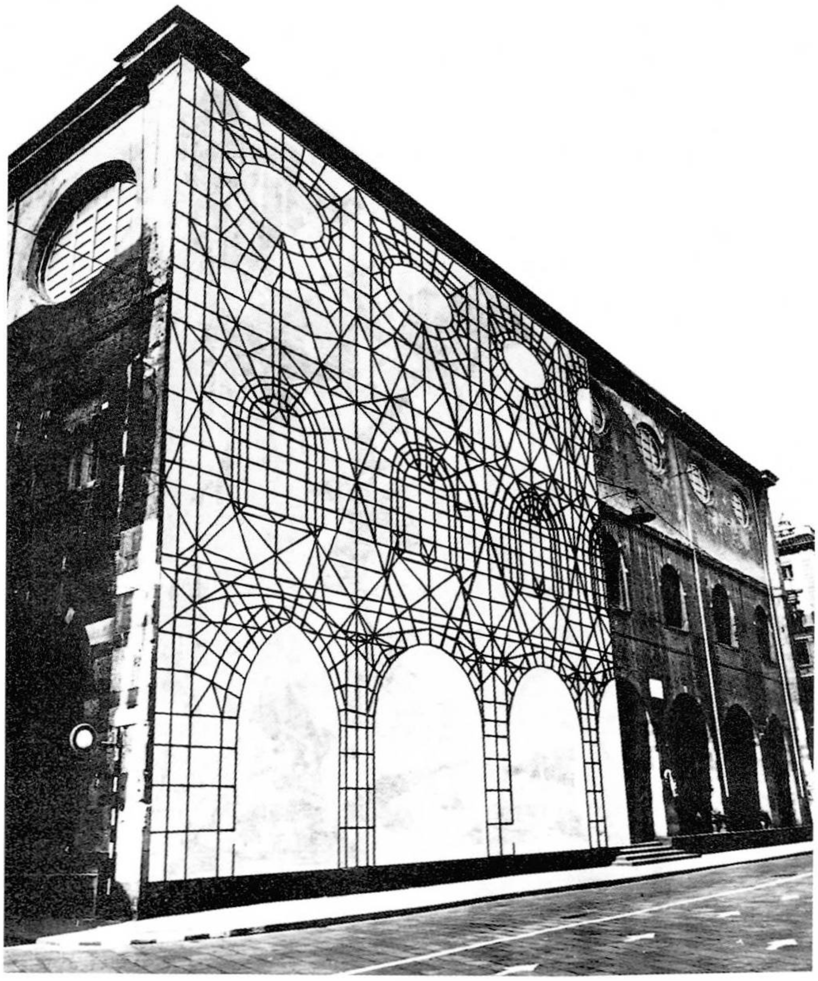

Figure 1: Palazzo della Ragione, Milano

It could be surmised that this simple approach is dictated by reasons of convenience; namely by the uncertainties associated with the definition of a nonlinear material model and with the execution of complicated nonlinear analyses. On the contrary many case histories indicate that this simple approach is usually adequate to provide realistic results and it is far from a "force majeure" approach. The details of the practical implementation are however sometimes different. The approach first adopted in the analysis of the Palazzo della Ragione is crude: the only cracks modelled are the ones that exhibit a significant crack opening displacement; moreover the length of the cracks is determined in advance.

Later the enormous reduction of computer costs has encouraged more sophisticated implementations even though the advantages are not always meaningful or easy to prove. The most interesting improvement is the execution of a non linear no-tension analysis instead of a linear elastic one. No-tension analysis has been developed for and extensively applied to concrete structures. When applied to historical buildings, it is crucial that the location of existing cracks is taken into account in the model. This is a partial remedy against lacking information on the variability of strength. The approach is as follows:

- cracks exhibited by the structure are taken into account during mesh preparation but they are initially closed; i.e. the model is initially undamaged;
- the full loads are applied or alternatively in case the loading changed during the life of the structure, such different phases of the load application are also simulated by the numerical model;

- no tensile stresses are admitted; therefore the boundaries of adjacent elements in regions of tensile stresses are disconnected and cracks are generated. This procedure is repeated step by step until complete relaxation of tensile stresses is achieved.
- A further refinement is to model no tension behaviour as a constitutive behaviour of masonry material in regions where no crack shape has been predetermined in the mesh. This is handled by introducing an anisotropic elastic model with zero elastic modulus in the direction parallel to tensile stresses. In this case the plausibility of computed results must be checked by investigating if a corresponding area of diffused microcracking exists in the real structure.

Of course this approach is much more computer intensive and occasionally numerically difficult than the linear elastic approach of many authors. On the other hand it may deliver two advantages that in some cases should not be neglected:

- first of all, in case loads are applied in loadsteps or tensile strength is reduced to zero in steps, it is possible to model the sequence of crack formation. This information may be compared with available historical records in order to collect evidence supporting the validation of the simulation or in order to interpret ancient strengthening interventions.
- Secondly, the consistency of the model is more carefully pointed out because it can be checked that cracks which do not open in the real building, also do not open in the numerical model.

One of the bolder masonry structures ever built, the Mole Antonelliana, provides an example of no tension analysis applied to the assessment of strengthening measures [5].

Finally it should be noted that another well established finite element technology, the use of joint elements, is rarely applied to monumental structures. Joint elements are widely used in rock mechanics and have found very useful applications in other engineering fields. Cracks in compression are also common in monumental buildings, there is however little experimental data on the constitutive equation of masonry joints.

3.4 Low amplitude dynamic data

The analysis of the dynamic behaviour is an important diagnostic tool to be used for validation of the numerical model, calibration of average material properties as well as investigation of the connectivity of different structural elements.
The dynamic excitation of the structure can be achieved in different ways:

- by means of environmental vibrations (traffic, wind, bells, microseismic noise, etc.). In this case the testing is easier and inexpensive but signal interpretation is more difficult [19,14,15];
- by means of devices which generate sinusoidal or random forces. In this case the test is costly and more difficult to set up (e.g. because of the need for authorization of load application). The main advantages are the better signal to noise ratio and the possibility to carry out tests at different levels of dynamic excitation [15,21,17].

The level of dynamic excitation is in both cases quite low. This is not so much a technological limit but rather a form of respect to the monument. As the dynamic stresses are small in comparison with the permanent stresses there is no doubt that linear elastic models are applicable even in case the investigation of the static structural behaviour points out nonlinear effects.

In table 2 are summarized the main dynamic test carried out by ISMES on monumental structures. The table shows that many tests, particularly the old ones, have been aimed mainly at characherizing the level of enviromental noise (traffic, wind, bells) in order to asses the risk of damage and therefore no numerical model was created to own knowledge neither by customer nor by others. After 1987 and the numerical/experimental studies carried on the Arnolfo Tower the numerical interpretation of the experimental tests has been performed when structural stability was under examination. Numerical models were set up also for some monuments previously tested (e.g. Marte Ultore Temple).

Year	Monument	Town	Wind	Traffic	Bells	Shaker	Modeling
1982/4	Duomo of Milan	Milan		x			
1984	Marte Ultore Temple	Rome	x	x		x	x
"	Flaminio Obelisk	"		x		x	
1985	Coliseum	"		x			
"	Trajan Column	"		x			
"	Minerva Temple	"		x			
"	Terme of Caracalla	"		x			
"	Trofei of Mario	"		x			
"	Arch of Costantino	"		x			x
1987	Arnolfo Tower	Florence	x			x	x
"	S.Maria del Fiore Cathedral	"	x				x
1989	Grotte di Catullo	Sirmione				x	x
"	S.Francesco Church	Arezzo		x	x		
"	Theater "Alla Scala"	Milan		x			
1990	Majno Tower	Pavia	x				
"	Fraccaro Tower	"	x			x	x
1992	Campanone Tower	Bergamo	x		x	x	x

Table 2: Dynamic tests

Comparison between measured and computed response can be based on time hystories or other directly available data such as transfer functions. This approach is often numerically expensive during model calibration. Alternatively the measured data may be processed in order to extract natural frequencies and mode shapes. In this way the numerical effort can be more efficiently targeted to a limited number of low frequency modes.

The easiest and more common use of dynamic data for model calibration is the fitting of the first (or the first few) frequency (or frequencies) by adjusting the average elastic modulus. More powerful technologies for model calibration are available even though application to monumental buildings are missing yet. The most common ones are the "Theoretical stiffness matrix" and the "System error matrices" approaches. Both are able to identify and quantity global as well as local sources of discrepancy between computed and measured data.

4. SAFETY ASSESSMENT STUDIES

4.1 Role of numerical computations for safety management

Probabilistic risk assessment has been increasingly used to predict the level of safety or risk associated with structures. The real meaning of such analyses is not completely clear, when applied to unique or potentially catastrophic structures such as monuments, large dams or nuclear power stations [6,7]. Research in the nature of failures and disasters in a general sense has indicated that failure modelling itself fails to account for the occurrence of many failures. This is due to events and unforeseen combinations of events that fall outside the necessarily closed set of scenarios used in probability of failure calculations. Probability theory provides a theoretical framework for dealing with uncertainty, but does not take into account the problems of completeness associated with theories and models.

As the likelihood of failure of a unique structure is essentially unknowable, its safety should be approached as a problem of management. Whereas risk prediction is a calculation at some point in time, management of safety is a continuous procedure, similar to that used in HAZOP studies in the chemical industry, which is a structured discovery technique for identification of hazard in a project design and operation.
In this approach safety must be always under review and important structures must periodically undergo "Hazard Audits", aimed at detecting whether an accident is incubating by testing for the sort of factors which can lead to failure. Safety management is therefore centred on collecting and elaborating information through periodic check-up and, when appropriate, continuous monitoring of the structure. Of course elaboration of information concerning safety may also include, but is not limited to, probabilistic risk analysis.

The concepts introduced in the above summary, which is based on references [8,9], will now be applied to the field of monumental structures. A hazard audit should address three main obvious issues: the extent of our knowledge of the structure, the identification and interpretation of any anomalous trend in structural behavior and finally the capacity to resist exceptional loads or further deterioration. The model calibration studies discussed in the previous section are obviously very relevant to the first two issues. Infact the writer believes that computer interpretation of structural behaviour both as it evolved during the life of the monument and as measured under different test conditions is a cornerstone of the safety management procedure.

For what concerns the third issue, the availability and the actual use of advanced capabilities for estimation of structural capacity is well documented in these proceedings as well as in other previous technical literature. The methodologies presented cover both linear and nonlinear, deterministic and probabilistic approaches. Reviewing this valuable material is not possible in this short document. Here and in the next paragraph another well known, but usually unspoken of, crucial topic will be addressed: the interaction between safety assessment studies and the often extreme variability of structural properties.

A meaningful example is the collapse of the Civic Tower of Pavia documented in [10]. As the permanent load produced mean stresses much lower than the average strength but close to the 5% fractile values, it has been surmised that a local failure has occurred due probably to the combination of a poorly constructed zone in an area of stress concentration. Finite element computations have also proved that a local failure could trigger an overall collapse as the residual load capacity was limited.

This case history poses a crucial question: how could the risk have been diagnosticated in advance? Simplified analysis methods could have indicated the opportunity of further investigations only if combined with very conservative masonry strength estimation. As the number of material tests is usually small, it is likely that the structure would have been considered safe. In conclusion the writer believes that the uncertainty of numerical results due to with incomplete material characterization is the main problem in safety management of monuments. Sensitivity analyses and probabilistic methods should be used in order to identify priorities and minimum standards for data collection.

In the next section a simplified example will demonstrate the possibility of carrying out such kind of analysis.

4.2 Simplified example of data completeness analysis

A typical medieval church of Southern Italy was selected as representative of a class of structures quite repetitive and widely diffused in Italy. The analyses carried out will be sketched only, as they are not yet completed. Only dead load has been considered and attention is focused on elastic stiffness uncertainties.

A preliminary analysis, based on best estimate material properties, was carried but in order to identify critical areas and to point out the most significant structural parameters, such as stresses or deflections.

Then a linear sensitivity analysis was used to identify the influence on the critical structural parameters of material property variations within the building. During this analysis the finite elements, which belong to the same structural element, are grouped together and the investigated material property perturbations are assumed constant within each structural element.

In our case the vertical stresses in some pillars are the main critical parameters. Of course the vertical stress is mainly influenced by material property variations within the pillar itself, but the sensitivity analysis identifies, which adjacent structural elements are of importance and the extent of the influence (in our case approximately one half of the influence of variations within the pillar itself).

Sometimes the state of stress is large and there is the need for increasing the level of confidence in the computed values by carrying out additional in situ tests for material property characterization. In this case the analysis provides also a rational tools for distributing the (scarce) additional tests among the various members.

As the number of material property tests is always limited the safety of the structure should be evaluated taking into account the effect of non-parametric bounds.

5. APPLICATION OF ADVANCED MODELLING TECHNOLOGIES

The most advanced modelling technologies are gradually being transferred from other engineering domains to the analysis of architectural heritage. Here a few examples are briefly reviewed.

5.1 Geometric modelling

Solid modelling has been first developed in mechanical engineering for drafting applications. It has now spread to every kind of complex finite element analysis because it has the advantage of addressing separately two difficult tasks: the geometric description of complex shapes and the subdivision of a solid into a finite element mesh. This splitting of the difficulties is essential to keep the most complex problems manageable. Recently a second advantage has been achieved: the second

task, namely the mesh generation, has been completely automated thereby considerably reducing the cost and time required for the analysis.
A first and very substantial application of geometric modelling to architectural heritage is the modeling of Pisa Tower, demonstrated in these same proceedings [11]. The complexity of this model could not have been achieved with other sophisticated but more traditional approaches.
A word of caution should however be spent on the automated mesh generation. Unfortunately meshes automatically generated are comprised only of tetrahedral elements. If a linear interpolation of displacement is used, tetrahedral are very stiff and an extremely fine mesh may be necessary to get reliable results.
It is therefore important that a quadratic or higher interpolation of displacement be used at least in regions when the values of computed stresses is important. Algorithms to check the adequacy of the mesh and to improve the solution either by mesh or by polynomial level refinement should be used in combination with automated mesh generation techniques.

5.2 P-version modelling

The capability to improve the accuracy of the numerical approximation by increasing the polynomial order P of interpolating displacements is known as "P-version" of the finite element method. This capability is the most efficient numerically and it also opens the way to automated accuracy refinement and control [12] P-version may be applied in combination with geometric modelling and automated mesh generation techniques. The advantage is that the degrees of freedom may be assigned automatically in areas of stress concentration in order to improve the numerical results.

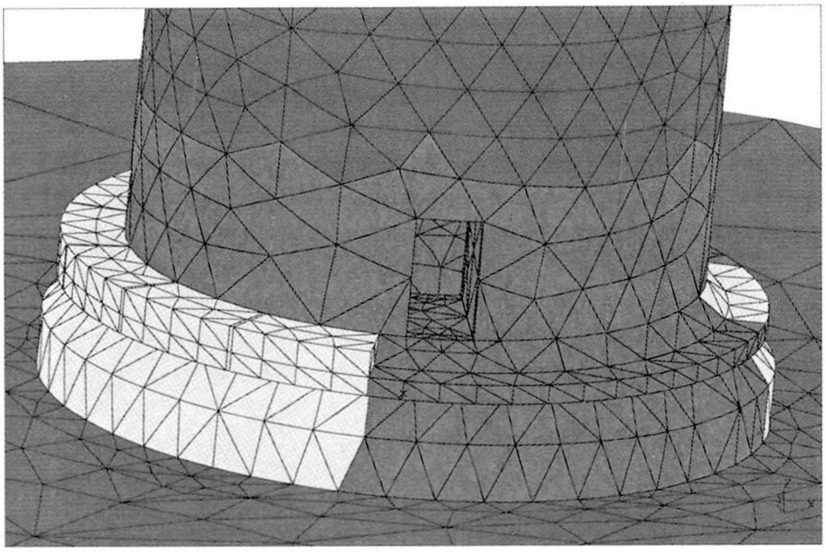

Fig. 2 Use of different polynomial levels according to specific computational tasks

5.3 Explicit integration in time of nonlinear problems

Complex nonlinearities such as opening and closing of joints, cracking, geometric effects and plastic behaviour usually lead to algorithmic difficulties and unmanageable computer costs. On the other hand successful results have been achieved in impact loading and rock mechanics problems by means of explicit time integration of nonlinearities and thanks to the stabilizing effect of inertia forces. Realistic modelling, if at all possible, of the seismic resistance of architectural heritage will be based on this approach, which is however nowadays inadequate to manage realistic problems, both for lack

of experimental validation and for excessive computational effort.
The use of distinct elements, proposed and by other papers [25] in these proceedings, should be further experimented and validated.

5.4 Knowledge-based systems

Artificial intelligence technologies have found interesting niche applications in civil engineering. One of the most relevant is the development of decision support systems.
For architectural heritage two prototypes are being experimented: one is related to interpretation of monitoring data [13] and the other to seismic resistance of masonry structures [15]. If a structural code applicable to architectural heritage will ever be developed, it has to be formulated as a knowledge based system, defining procedures for the management of safety.

6. CONCLUSIONS

Substantial progress has been achieved with respect to the state of art illustrated by the 1983 IABSE symposium in Venice.
Modelling of architectural heritage is a rapidly exploding field of research, where significant results have already been achieved. Further progress relies on a close cooperation between numerical and experimental work, as both are often meaningless if not integrated.

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