CIVIL ENGINEERING STRUCTURES: MULTISCALE DAMAGE REPRESENTATION, IDENTIFICATION, CONTROLLED DESTRUCTION AND QUICK RECONSTRUCTION

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Abstract. In this paper we review the large field of research pertaining to damage detection and rebuilding the human environment in urban areas following the natural disasters and past military activities. This domain of current practical interest is highly interdisciplinary and belongs not only to the traditional domain of Civil Engineering, but draws upon to a large extent on domains of Applied Mathematics, Material Science, Physics and Chemistry. In particular, we first review the typical damage to engineering structures and infrastructure in urban zones, which can be caused by the extreme conditions of mechanical or thermal loads, interaction with aggressive environment, from very high-rate (e.g. explosions) to very low-rate applications (e.g. the problems of durability or ageing). We then discuss the topics of current research interest related to the techniques of damage detection in structures and infrastructure, with special emphasis on detection under heterogeneous fields and heterogeneous material properties. We only briefly discuss the techniques for controlled destruction of heavily damaged engineering structures, which are no longer deemed reparable. Finally, we give a quick overview of the techniques for quick reconstruction and the structural systems that can be employed for that purpose.

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1. Introduction

The present time of local conflicts mostly fought in urban areas brings additional challenge of damage detection and quick rebuilding of human environment, which is very important for preventing large emigration flux from the affected areas. The engineering knowledge faces perhaps the most demanding problems in this domain of application, and it needs help by a number of other scientific disciplines, such as Applied Mathematics, Material Science, Physics and Chemistry. A number of questions that need the answer related to this class of problems: first and foremost, how to describe different damage mechanism of engineering structures and infrastructure and how to account for the natural variation of these mechanisms within a complex, heterogeneous structure; in short, the main question pertains to classifying different damage mechanisms in massive structures. The second question we address is how to detect these damage mechanisms, and estimate the parameters of the model that is supposed to represent them. We note in passing that the study presented herein focuses mostly on the mechanical damage identification, with no mention of any other techniques used for non-destructive damage assessment, such as X-ray or ultra-sonar probing. When the excessive damage is found, in a structure that is damaged beyond reparation, we will face a difficult problem of controlled destruction in an urban zone, where other neighboring structures should not be damaged while removing the damaged structure. It turns out that the modeling procedure of controlled destruction calls upon practically all available tools currently used for solving different problems in structural engineering domain. The final goal to be studied pertains to the quick reconstruction and retrofit of damaged structures. The former can be accelerated by the special deployable structures, whereas the latter can be accelerated by the judicious use of modern materials.

In this chapter the most thorough part of discussion is given of the first issue pertaining to the damage mechanisms description in a massive engineering structure, by taking into account the heterogeneities of the material properties and the loading history producing different levels of damage. We draw attention in particular to a very specific form of damage mechanism for a massive structure, where the fracture process zone plays equally important role to the final crack that endangers the integrity of a given structural component. We also show that the finite element interpolation has to be constructed accordingly for the best possible representation of this kind of mechanisms. In the second part of the chapter we review two classes of identification methods based upon forward mode and inverse mode analysis. We show how these identification methods can be used for precise identification of the parameters for representing nonlinear inelastic behavior of a damaged structural component. We also indicate that it is very likely the inverse mode that has a larger potential in dealing with identification problems for a complex structure. In the last part of the work, we very briefly review the main steps in controlled destruction of a heavily damaged structure, as well as the quick reconstruction problems.

2. Damage in massive civil engineering structures

2.1. NOVEL MODELS FOR DAMAGE ASSESSMENT

Ever increasing demand to achieve a more economical structural design requires that a better understanding be obtained for the non-linear behavior of a particular structural system and reliable estimate of its limit load be furnished. However, the limit state of a complex system often implies that particular components – whose peak resistance has already been defeated – be able to function in the post-peak regime, which induces strain softening behavior. A reliable evaluation of the limit load of the considered system is thus highly dependent upon an efficient and reliable description of the softening branch of the material behavior.

The models for computing the ultimate limit load are not reliable enough as soon as we are facing the heterogeneous structures and heterogeneous stress field; the benchmark competition on a single component (a shear wall) with double notch, submitted to non-proportional loading starting first with shearing and followed by a traction force: The results show very significant difference from the experimental results (which are dominated by more than one damage mechanism), and moreover a large dispersion among them (Fig. 1).

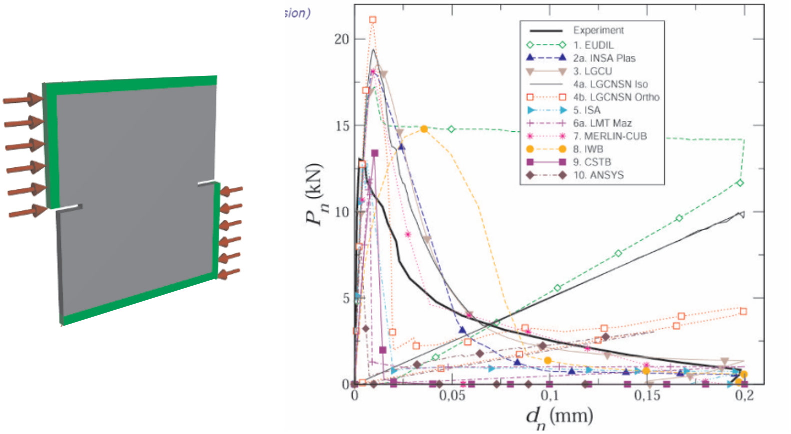


Figure 1. Results of numerical simulations Nooru-Mohamed benchmark of double-edge notched component

With new damage assessment models, most of them developed recently (Ibrahimbegovic and Brancherie, 2003; Ibrahimbegovic and Delaplace, 2003; Ibrahimbegovic and Markovic, 2003; Ibrahimbegovic and Melnyk, 2007; Brancherie, 2003; Brancherie and Ibrahimbegovic, 2006, 2009; Colliat, 2003; Colliat et al., 2005, 2007, 2008; Markovic, 2004; Markovic et al., 2005; Niekamp et al., 2009; Davenne et al., 2003; Dolarevic, 2005; Dominguez, 2005; Herve, 2005; Melnyk, 2007, see also Leger and Tremblay [2008] or Owen et al. [2008] for damage in dynamics), we can obtain not only this global information, but also the local information in a more reliable manner; for example, for the benchmark problem of Nooru-Mohamed, we also obtain close match with crack spacing and opening under non-proportional loading (Fig. 2).

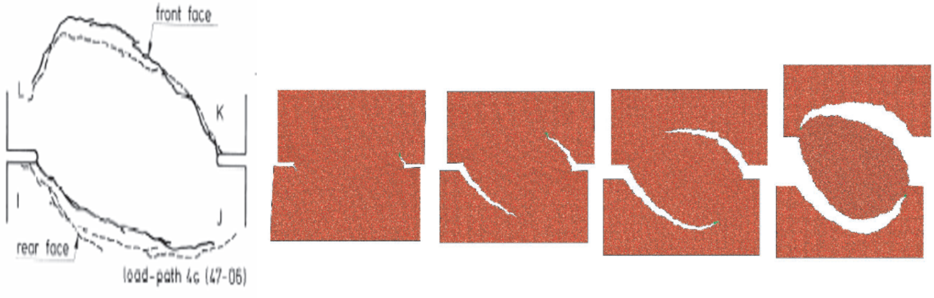


Figure 2. Crack pattern for Nooru-Mohamed benchmark: experimental results versus computations with refined damage models taking into account heterogeneities of material and stress field

The models had for the starting point the fact that no anisotropic damage model is capable of representing correctly the fine details of the solution. The guiding principles for development of new generation of damage models were:

(i) Separate complexities and assign to each mechanism of inelastic behavior its criterion.
(ii) Account for true nature of inelastic dissipation (for example, volume vs. surface).
(iii) Localized failure description cannot be described only by the UMAT kind rules specifying how to compute the stress from strain, but FE representation of strain field has to be constructed accordingly, by taking into account the heterogeneities.

Of special interest for this work is the failure pattern for massive structures, where the creation fracture process zone (FPZ), with a number of tiny cracks of different orientation, proceeds the creation of the macro crack, which endangers the integrity of a particular component of the whole structure (see Fig. 3).

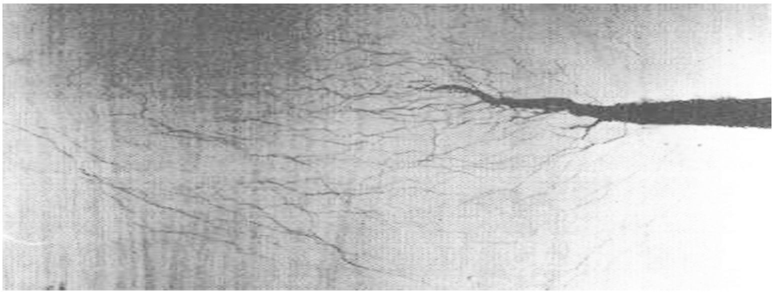
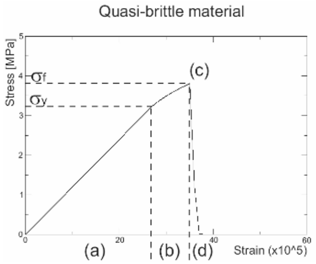


Figure 3. Typical failure pattern of a massive structure, where the fracture process zone (FPZ) precedes the macro crack creation that leads to final failure of the given component or the entire structure

The models of this kind will have at least two dissipative mechanisms, the first related to the fracture process zone and the second related to the localized failure (Fig. 4). Each of mechanisms is described by a particular plasticity failure criterion (we could also choose the damage criteria). We can thus obtain the basic set of governing equations, which can be written and illustrated as:



$$
\begin{array}{ll}
\epsilon & = \epsilon^e + \bar{\epsilon}^p + \bar{\bar{\epsilon}}^p \, \delta x \\
\Psi & (\epsilon^e, \bar{\xi}, \bar{\bar{\xi}}) = \bar{\Psi}(\epsilon^e, \bar{\xi}) + \bar{\bar{\Psi}}(\bar{\bar{\xi}}) \\
\bar{\Phi} & (\sigma, \bar{q}) := |\sigma| - (\sigma_y - \bar{q}) \\
\bar{\bar{\Phi}} & (\sigma, \bar{\bar{q}}) := |t| - (\sigma_f - \bar{\bar{q}}) \\
\dot{\bar{\epsilon}}^p & = \dot{\bar{\gamma}} \dfrac{\partial \bar{\Phi}}{\partial \sigma} \quad ; \quad \dot{\bar{\xi}} = \dot{\bar{\gamma}} \quad ; \quad \bar{\Phi} \leq 0 \quad ; \quad \dot{\bar{\gamma}} \geq 0 \quad ; \quad \dot{\bar{\gamma}} \bar{\Phi} = 0 \\
\dot{\bar{\bar{\epsilon}}}^p & = \dot{\bar{\bar{\gamma}}} \dfrac{\partial \bar{\bar{\Phi}}}{\partial \sigma} \quad ; \quad \dot{\bar{\bar{\xi}}} = \dot{\bar{\bar{\gamma}}} \quad ; \quad \bar{\bar{\Phi}} \leq 0 \quad ; \quad \dot{\bar{\bar{\gamma}}} \geq 0 \quad ; \quad \dot{\bar{\bar{\gamma}}} \bar{\bar{\Phi}} = 0
\end{array}
$$

Figure 4. 1D representation of failure model for massive structures and governing equations postulating: (i) additive split of total deformation into elastic, smooth plastic and localized plastic field, (ii) strain energy in terms of elastic strain and hardening variables, (iii) plasticity criteria for smooth and localized field, and the evolution equations for the internal variables (note that one over-bar pertains to the FPZ mechanisms, and two bars is used for the internal variables describing the localized failure)

The typical result obtained with models of this kind is given in Fig. 5, for the case of simple tension test, where the global force–displacement diagram indicating three different regimes of response (with elastic, inelastic hardening and inelastic softening) remains the same for different mesh grading.

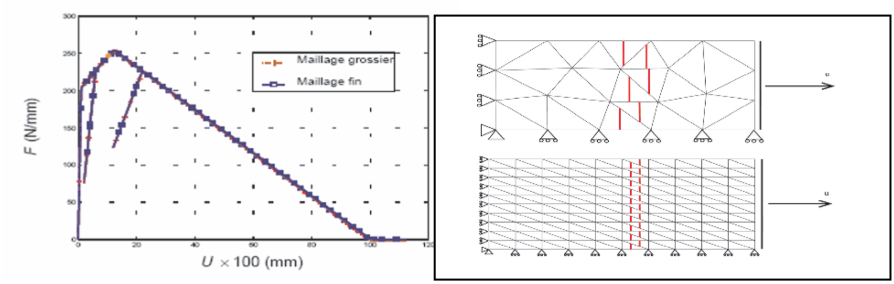


Figure 5. Computed response with failure model for massive structures with FPZ, and crack pattern by using coarse mesh and fine structured mesh

When going one step further along this line of developments, we can look at the reinforced concrete structures (as probably a dominant class of structures in civil engineering practice), where the judicious combination of steel and concrete has provided the best building material in terms of cost. However, the nonlinear inelastic behavior of reinforced concrete is perhaps the most difficult one to comprehend. More importantly, the most adequate model will strongly depend upon the goal of the present analysis, and the estimate of the damage we would like to obtain. In that respect the choice should be made between: i) global integrity of the structure built of reinforced concrete under extreme loading, ii) local integrity of concrete under extreme loading case and iii) durability of concrete under aggressive environment. The models for each of the listed goals get to be more and more detailed; see Fig. 6.

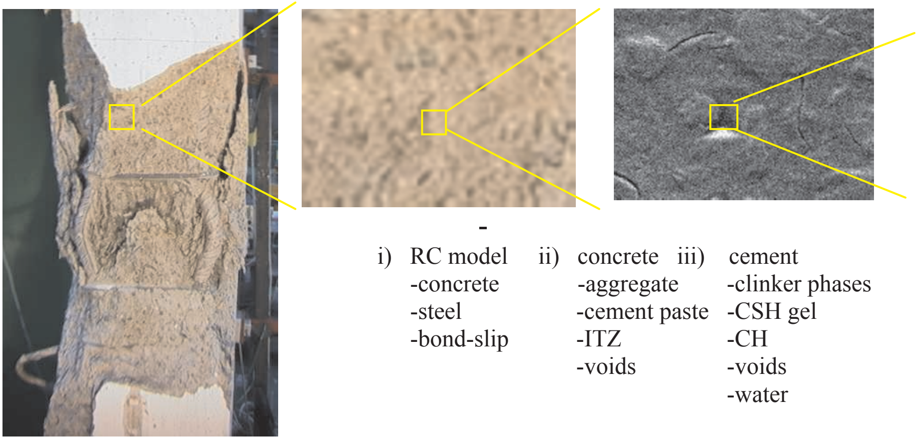


Figure 6. Different models of reinforced concrete for studies of: (**a**) structural integrity, (**b**) material integrity, (**c**) material durability

2.2. HETEROGENEITIES OF REAL STRUCTURES AND SIZE EFFECT

The damage modes on the real structures are very much influenced by the heterogeneities at the structural level, related to the spatial variation of the material properties, as well as the non-homogeneity of the applied stress field. For that reason, the type of structural failure for the same construction material should be explained by accounting for the structure heterogeneities. The case in point concerns the analysis where the material properties are not prescribed in the deterministic manner, but rather as random fields.

In Colliat et al. (2007) and Hautefeuille et al. (2009), we have studied the possibilities of describing the heterogeneities of material properties for the case where the material microstructure or mesostructure properties are known, but their geometric arrangement is random. More precisely, we have looked more closely into the class of concrete models built at the meso-scale, by accounting for Poisson's process distribution of the aggregate within a given component shown in Fig. 7.

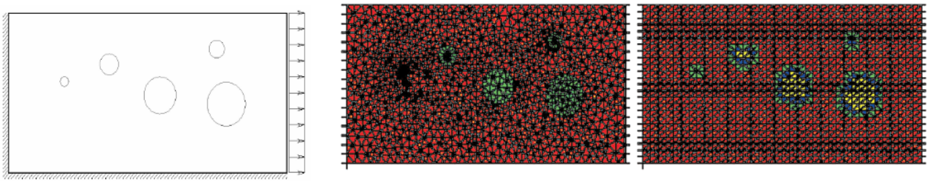


Figure 7. Schematic representation of two-phase meso-structure of concrete as two-phase material taking into account the difference between aggregate and cement paste, and exact and structured mesh representation

We assume the aggregate to remain elastic, and the cement paste constitutive behavior to be governed by the elastic-perfectly-plastic Drucker–Prager plasticity model, and both phases will have deterministic material parameters. The only randomness will stem from the uncertain meso-structure geometric arrangements,

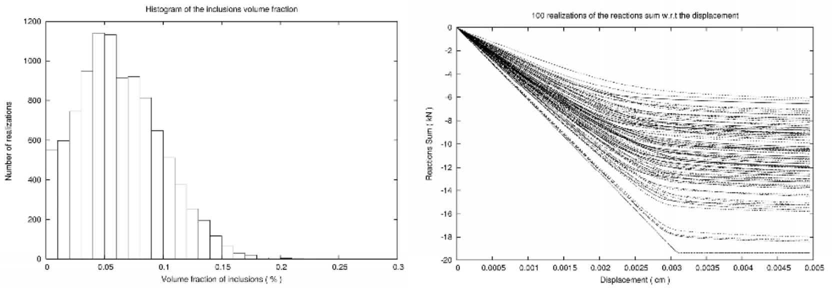


Figure 8. Phase histogram and response dispersion for different realizations of two-phase representation of concrete constitutive model

which are produced by Poisson's random process enforcing minimum and maximum aggregate size and distance. The response statistics is computed with the structured mesh (since it provides much more efficient computational procedure), by using the Monte Carlo method. The results presented in Fig. 8, in terms of phase histogram, and the dispersion of 1D representation of the constitutive behavior.

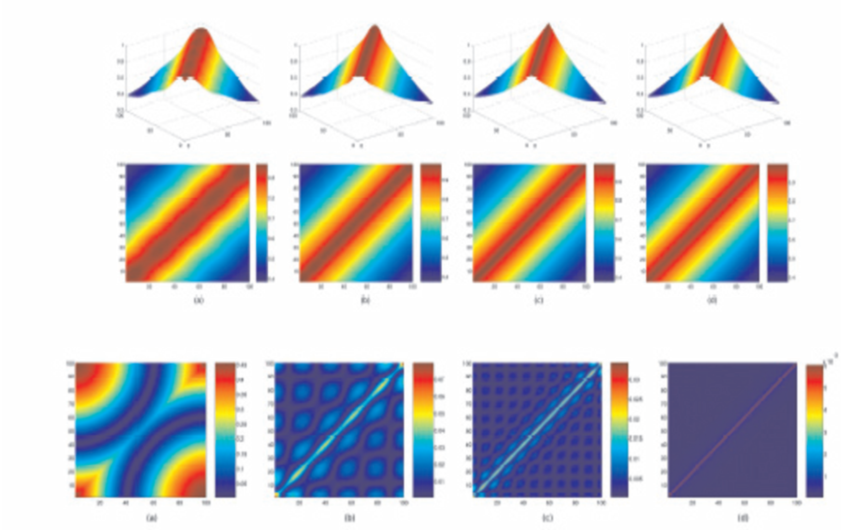


Figure 9. Covariance representation with 5, 10 or 50 Karhunen-Loeve modes versus exact representation (3D – *top* row, 2D – *middle* row, error – *bottom* row)

The main idea pursued further pertain to reinterpretation of the results of this analysis in terms of 1D model for localized failure with FPZ for massive structure defined in Fig. 4. However, the crucial difference now is that the parameters of this model governing activation of diffuse plasticity in FPZ and localized failure are defined as random field. For simplicity, we assume statistically homogeneous material, with the random field parameters characterized by the correlation length; for example, for yield stress we would have the typical space variation described by:

$$cov_{\sigma_y}(x,y) = Vexp(-\frac{||x-y||_1}{L_c})$$

where V is the variance of the field, x and y are two points in space and L_c is the correlation length. For the chosen correlation length, the spatial variation of the material parameter then can be expressed in terms of KL modes expansion, with no more than 50 modes needed; see Fig. 9.

The typical realization of material properties variation obtained in this manner for short, medium and long bars are given in Fig. 10.

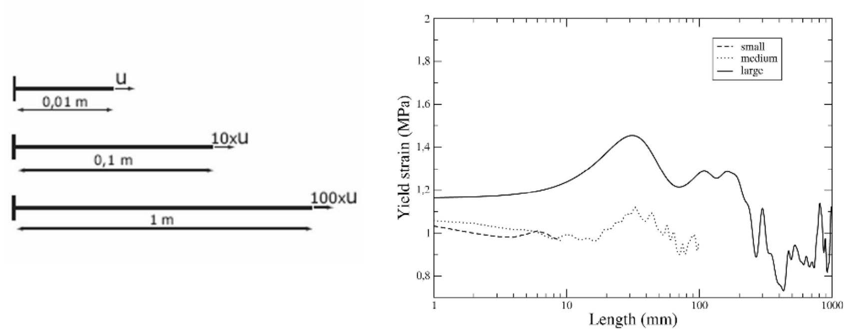


Figure 10. Realizations of yield stress spatial variation for short, medium and long bars

The computations are then carried out with this kind of probabilistic models of structures, computing the failure of each realization. The final results are used to built the cumulative distribution of failure for different structures, with the percentage of broken bar shown in Fig. 11.

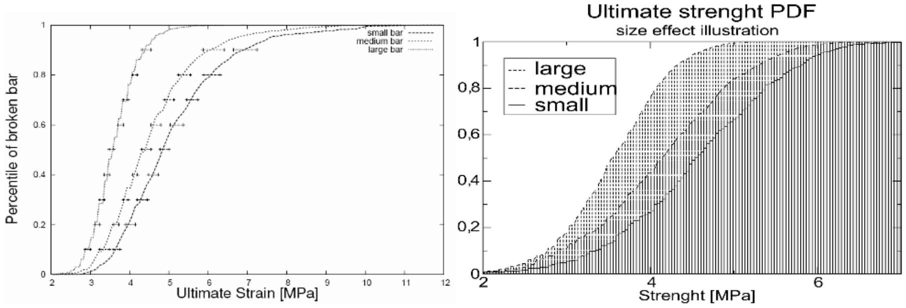


Figure 11. Cumulative distribution with percentage of broken bars for short, medium and long bar, along with the confidence index for each case

The last result can be resented in an alternative manner, by plotting for the percentage of broken bars the corresponding strength (see Fig. 12), which allows us to show that not only the long bars will break sooner (for more likely presence of the weak spot), but that the type of failure will be closer to the one described by classical fracture mechanics (with a negligible FPZ).

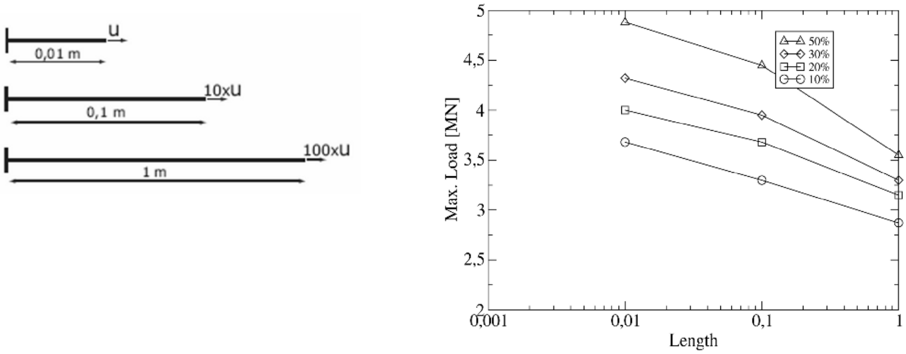
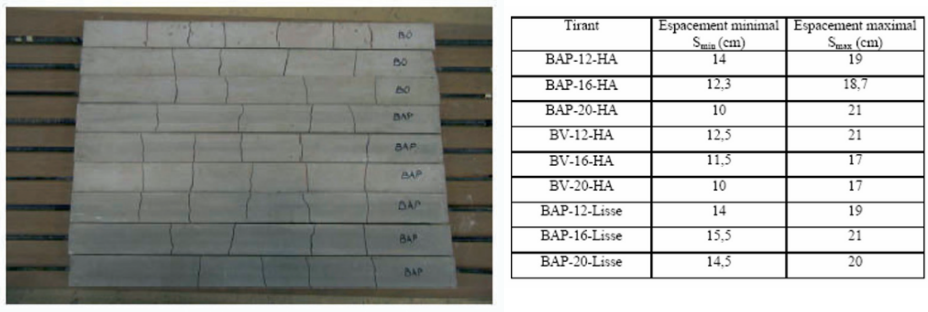


Figure 12. The corresponding values of ultimate strength for short, medium and long bars indicating size-effect, with longer bar breaking sooner with a small FPZ size

Similar effects on material heterogeneities are obvious in response of RC structures, producing rather a different crack spacing that does not remain constant. The case in point is the result presented in Fig. 13, which present the outcome of different RC tie-beam tension tests.



Tirant	Espacement minimal S_{min} (cm)	Espacement maximal S_{max} (cm)
BAP-12-HA	14	19
BAP-16-HA	12,3	18,7
BAP-20-HA	10	21
BV-12-HA	12,5	21
BV-16-HA	11,5	17
BV-20-HA	10	17
BAP-12-Lisse	14	19
BAP-16-Lisse	15,5	21
BAP-20-Lisse	14,5	20

Figure 13. Experimental results of tension test on RC tie-beams producing variable values of crack spacing (from Daoud [2003], doctoral thesis INSA Lyon)

The test of this kind is carried out under displacement control directly imposed upon steel bar, which puts the bar and the surrounding concrete in tension. At certain value of deformation, the concrete can no longer follow the bar deformation, and produces the first crack followed by additional cracks each occurring at certain distance with respect to the previously created crack. We have carried out the numerical simulation of these tests by using the RC model, which consists of the concrete model described in the previous section, the elasto-plastic steel model and the bond-slip model described in Dominguez et al. (2005). The bond slip model of this kind can account for three different phases of slip, the elastic phase, the plastic slip with hardening and the last phase of large slip with softening. We note in passing that the softening phenomena of bond-slip do not lead to any mesh-objectivity problem, since the length scale is imposed by the presence of the steel bar that connects all the slips along the bar. In Fig. 14, we present the computed results obtained for the cases of: perfect bond between steel and concrete, the constant value of the bond slip resistance, along with the heterogeneity-imposed variation of the bond along the bar with different choices of standard deviation. We can see that the bond-slip do play important role and that its absence (perfect bond) produces rather non-physical results. We can also see that the best match to experimental results for crack spacing is produced by the standard deviation of 5%.

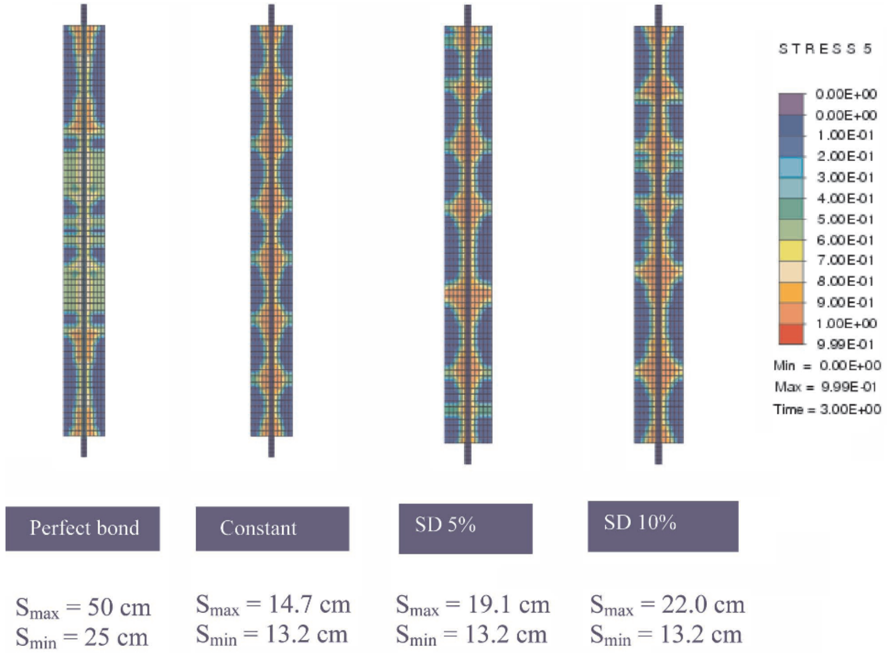


Figure 14. Results of numerical simulations of RC-tie tension test with (from left to right): perfect bond, constant bond-slip resistance, variables resistance with 5% and 10% standard deviation

3. Damage parameters identification in civil engineering structures

In this section we discuss the main issues pertaining to the damage parameter identification, which is the main requirement for successful damage assessment. We start with the methods that are usually used for material damage parameters, and than comment upon the suitable extensions to the methods for structural damage assessment.

The testing procedure for estimating the damage parameters of quasi-brittle materials is very much geared towards the tests under heterogeneous stress field, which are easy to perform. The case in point is three-point bending test, shown in Fig. 15.

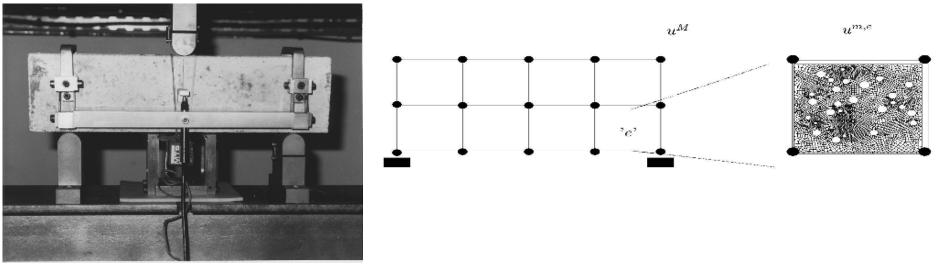


Figure 15. Typical test for quasi-brittle materials: three-point bending test

The typical measurements are no longer only the point-wise observation of the displacement corresponding to a given force, but the optimal measurements spread over certain area, which provides the displacement field throughout that area; see Fig. 16.

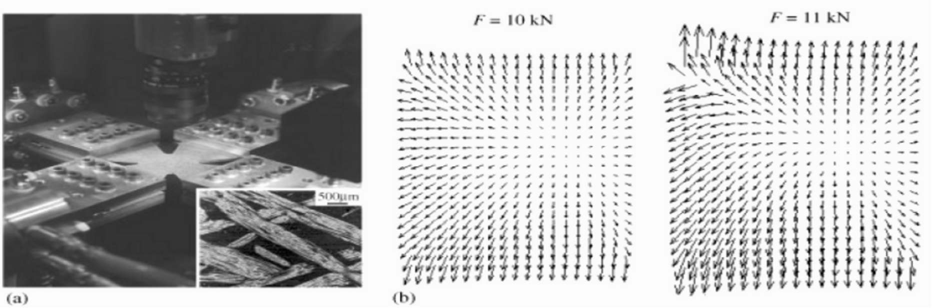


Figure 16. (**a**) Sample of the testing machine ASTRÉE and microstructure of the studied composite, (**b**) Displacement filed in biaxial test measured by digital image correlation for two load levels, with failure load F = 11.1 kN (Claire et al., 2001)

The modern identification procedures are nowadays adapted to this point of view, treating each specimen as the structure. We review briefly two types of methods: the forward and the inverse mode of identification (e.g. see Kucerova [2007] for more details).

3.1. FORWARD MODE IDENTIFICATION

The forward mode indentification provides the estimate of the model parameters from the given experimental measurements. If we denote the set of parameters of the model as X, the model operation as M and the measurements as Y^E, the forward mode identification reduces to:

$$F(X)=||Y^E-M(X)||$$

This kind of relation can be established either experimentally (results denoted with superscript E) or by means of the model (results denoted with superscript M), as illustrated in Fig. 17.

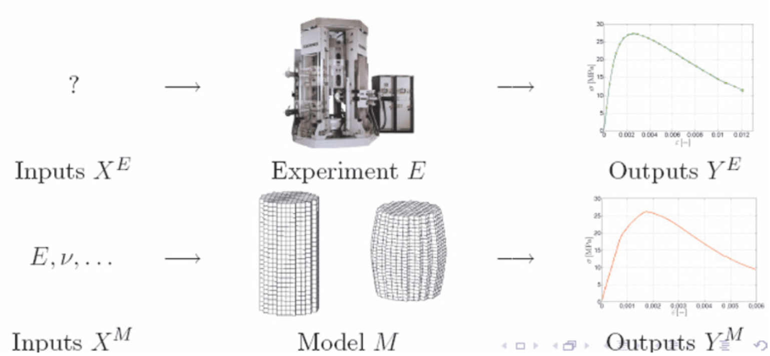


Figure 17. Forward model identification with relationship established either experimentally or by the chosen model

The forward mode identification can be carried out by minimizing the error functional, as a suitably chosen measure expressing the difference between the results computed by the model and the experimental results; see Fig. 18. We also show in Fig. 18 that the results provided by finite element model, the most frequently used kind of model for complex structures nowadays, should first be processed through the verification procedure to provide the sufficient guarantees for model accuracy, before carrying on with the model validation procedure from experiments.

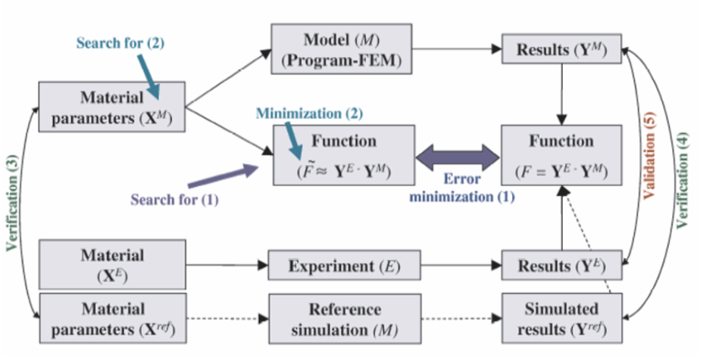
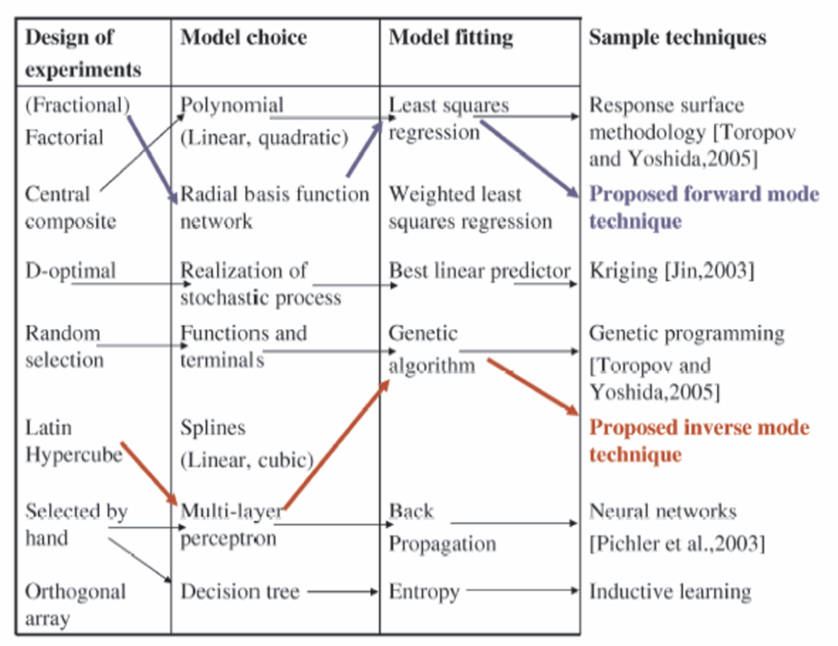


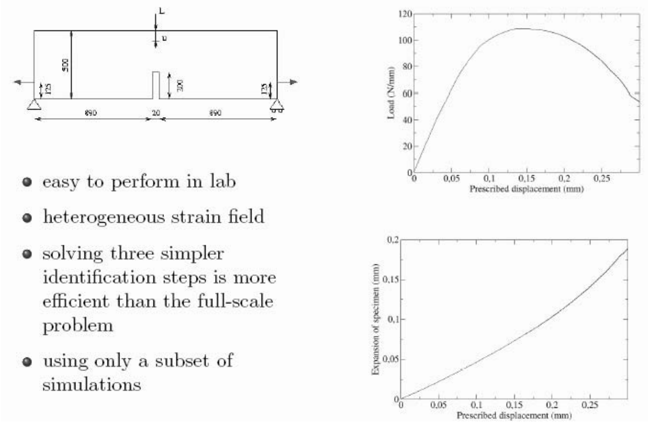
Figure 18. Forward mode of identification and definition of error function

A number of solution tools are used currently in order to improve the efficiency and the robustness of forward mode identification; some of them are listed in Fig. 19 given below (we also refer to Kucerova [2007] for recent state-of-the-art review):



Design of experiments	Model choice	Model fitting	Sample techniques
(Fractional) Factorial	Polynomial (Linear, quadratic)	Least squares regression	Response surface methodology [Toropov and Yoshida,2005]
Central composite	Radial basis function network	Weighted least squares regression	**Proposed forward mode technique**
D-optimal	Realization of stochastic process	Best linear predictor	Kriging [Jin,2003]
Random selection	Functions and terminals	Genetic algorithm	Genetic programming [Toropov and Yoshida,2005]
Latin Hypercube	Splines (Linear, cubic)		**Proposed inverse mode technique**
Selected by hand	Multi-layer perceptron	Back Propagation	Neural networks [Pichler et al.,2003]
Orthogonal array	Decision tree	Entropy	Inductive learning

Figure 19. Solution tools for forward mode identification problem

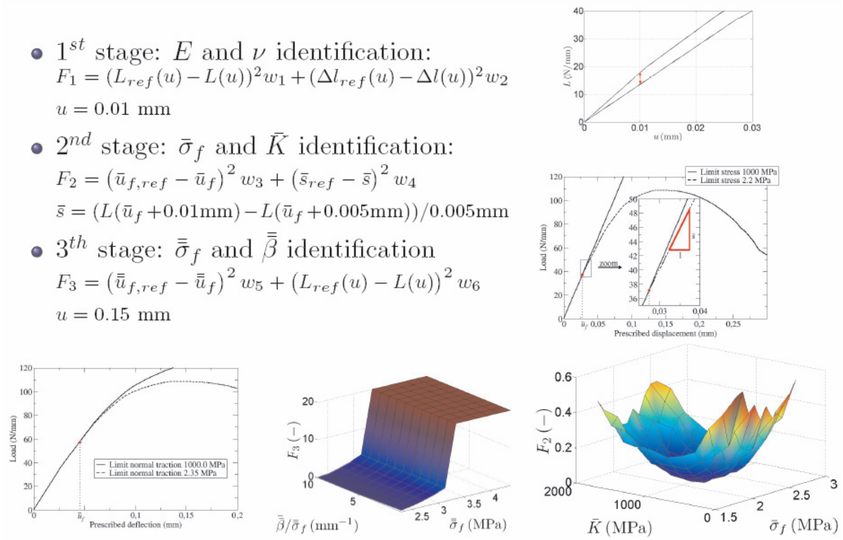


- easy to perform in lab
- heterogeneous strain field
- solving three simpler identification steps is more efficient than the full-scale problem
- using only a subset of simulations

Figure 20. Three-point bending test on notched specimen and typical measurements

The forward mode identification is not very difficult to solve if the chosen model has the sound theoretical foundation with clear interpretation of each and every inelastic mechanism. This is certainly true of the present model for localized failure of massive structures with a significant contribution of the FPZ. The case in point is provided by the results of identification problem from the standard three-point bending testing, performed on the notched specimen shown in Fig. 20.

The robustness of the identification procedure in this case is not only guaranteed by the sound theoretical basis of the model, but also by the judicious solution strategy where the complete set of material parameters is split in several groups, corresponding to three prominent phases of nonlinear behavior, and only one group is identified at the time. This strategy is illustrated in Fig. 21.



- 1^{st} stage: E and ν identification:
  $F_1 = (L_{ref}(u) - L(u))^2 w_1 + (\Delta l_{ref}(u) - \Delta l(u))^2 w_2$
  $u = 0.01 \text{ mm}$
- 2^{nd} stage: $\bar{\sigma}_f$ and $\bar{K}$ identification:
  $F_2 = \left(\bar{u}_{f,ref} - \bar{u}_f\right)^2 w_3 + \left(\bar{s}_{ref} - \bar{s}\right)^2 w_4$
  $\bar{s} = (L(\bar{u}_f + 0.01\text{mm}) - L(\bar{u}_f + 0.005\text{mm}))/0.005\text{mm}$
- 3^{th} stage: $\bar{\bar{\sigma}}_f$ and $\bar{\bar{\beta}}$ identification
  $F_3 = \left(\bar{\bar{u}}_{f,ref} - \bar{\bar{u}}_f\right)^2 w_5 + \left(L_{ref}(u) - L(u)\right)^2 w_6$
  $u = 0.15 \text{ mm}$

Figure 21. Forward mode identification procedure for three-point bending test, with parameter group identified from different regime measurements

We note in passing that the identification procedure of this kind is very much adapted to the problems of material parameter estimates, where we can choose a sufficiently reliable model of inelastic behavior, and afford the complete destruction of the chosen specimen, which allows us to gather the measurements over all regimes of nonlinear inelastic behavior.

3.2. INVERSE MODE IDENTIFICATION

The inverse mode identification is more suitable for the problems where the system behavior is neither clearly defined nor possible to capture by a sound theoretical model. The latter can be the consequence of extremely complexity of material behavior, extreme complexity of geometric setting (e.g. complex structures) or inability to carry out the tests and obtain the corresponding measurements over all different regimes of material behavior.

In the inverse mode identification we ought to solve for the approximation to the system behavior, without really having a very precise notion on the kind of model one can use. This can formally be written as:

$$\Rightarrow \text{determination of its } \textit{approximation } M^{INV}$$
$$X^M = M^{INV}(Y^M).$$

where M^{INV} is the inverse approximation to the complex system operator. The illustrative representation of the inverse mode identification is presented in Fig. 22.

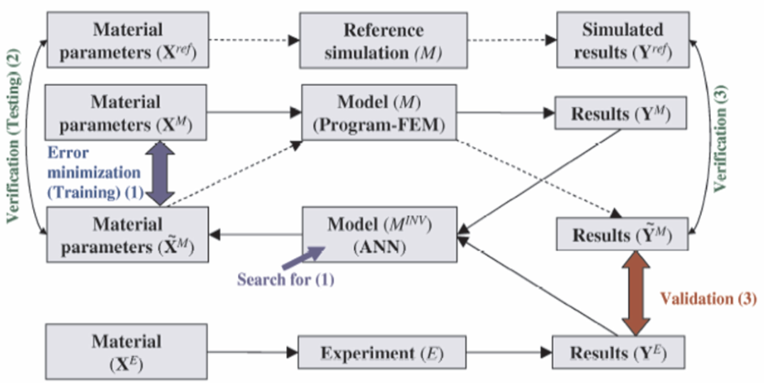


Figure 22. Flow-chart for inverse mode identification procedure

One can count with the most important advantage for such a inverse mode identification, such as the possibility to replace a complex systems rules with the corresponding approximation that allows a frequent and an efficient use of the artificial model, but also with a number of disadvantages, such as the need to carry out an exhaustive exploration of the inverse relationship, inability to solve for multi-modal problems especially with the data with measurement noise. One of the popular approaches for reconstructing the behavior of a complex system relies on neural networks, which can build the equivalent system response as the collection of simpler rules put together as a multi-layer representation (see Fig. 23 for illustration).

$$O_{l,j} = f_{act}\left(\sum_{i=0}^{n_{l-1}} O_{l-1,i} \cdot w_{l,i,j}\right) \qquad f_{act}(\Sigma) = \frac{1}{\left(1+e^{-\alpha/\Sigma}\right)} \qquad \alpha = 0.8$$

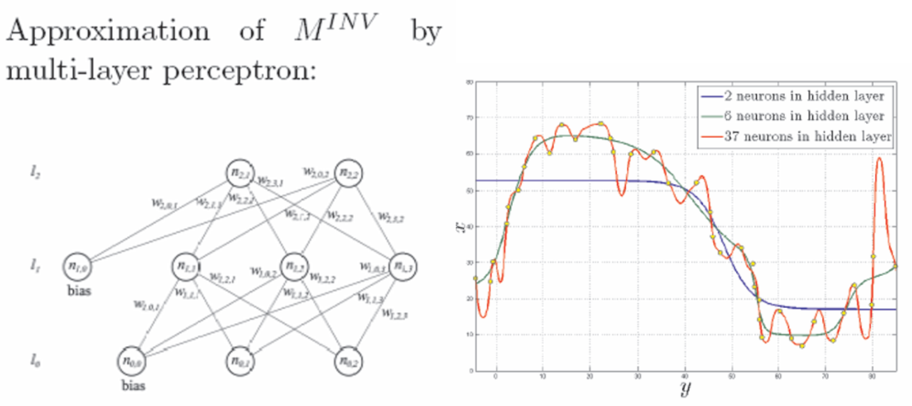


Figure 23. Approximation of M^{INV} by multi-layer representation and computed complex system response in terms of layer complexity

It is also shown in Fig. 23 that the complex system response representation by neural networks is then very much affected by the type of representation and the complexity of each links, in the sense that the increasing complexity can provide a more detailed representation (as opposed to the smoothed, averaged values given by coarse representation). One recent example of inverse mode identification is given in Kucerova (2007), dealing with the system identification capable of representing a very complex behavior of concrete under different stress states of either compression or tensile stress. This kind of behavior was supposed to be represented by the micro-plane “model” of Bazant et al. (2000), which assumes that 1D response in a number of planes (32 for 2D case, and 128 for 3D case, the numbers being imposed by the rules of numerical integration over a circle or a sphere, respectively), without any coupling in axial and tangential direction on each plane, can somehow describe the global response. In that respect, the micro-plane constitutive model can be considered as the model of complex structure whose behavior is influenced by a number of its components (i.e. “micro” planes). The micro-plane model can be packed in the form featuring a number of model parameters, which do not posses (except two: Young’s modulus *E*, and Poisson’s ratio *v*) clear physical meaning.

$$E, v, K_1, K_2, K_3, K_4, C_3, C_{20}$$

The lack of clear physical basis of most of coefficients and the need to understand behavior under different deformation modes make this identification problem very much equivalent to the model estimates for a complex structure from the estimate of its components. The importance of each parameter throughout the deformation history of interest can be obtained by Pearson’s product moment correlation coefficient for each parameter (see Fig. 24), which clearly shows that the system response does not remain equally governed throughout the deformation history.

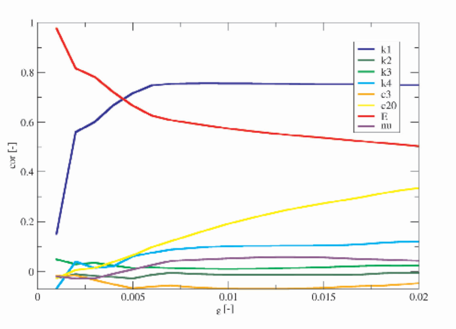
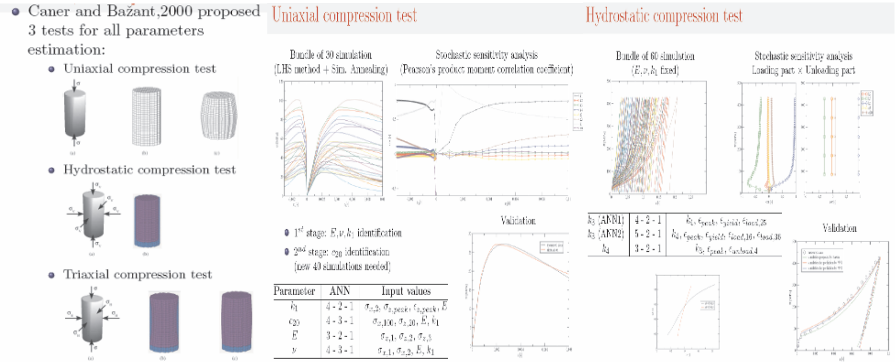


Figure 24. Pearson’s product moment correlation for micro-plane model parameters

Bazant and co-workers have proposed several typical tests in order to provide the best estimates of the micro-plane model parameters, with the most of parameters lacking a clear physical meaning. These tests include: uniaxial compression, hydrostatic pressure and triaxial compression; see Fig. 25. We can see that the tests of this kind can reveal the particular sensitivity of the model parameters in the course straining throughout different regimes.

It is very much open question how to achieve the same goal with a complex structure, and especially with the existing one, which might have been partially damaged. The question of this kind will be addressed by several keynote speakers, each focusing upon particular type of structure and particular natural or man-made disaster.



- Caner and Bažant,2000 proposed 3 tests for all parameters estimation:
  - Uniaxial compression test
  - Hydrostatic compression test
  - Triaxial compression test

Uniaxial compression test

Bundle of 30 simulation (LHS method + Sim. Annealing)

Stochastic sensitivity analysis (Pearson's product moment correlation coefficient)

- 1st stage: E, ν, k_1 identification
- 2nd stage: c_{20} identification (new 40 simulations needed)

Parameter	ANN	Input values
k_1	4 - 2 - 1	$\sigma_{z,2}, \sigma_{z,peak}, \epsilon_{z,peak}, E$
c_{20}	4 - 3 - 1	$\sigma_{z,100}, \sigma_{z,20}, E, k_1$
E	3 - 2 - 1	$\sigma_{z,1}, \sigma_{z,2}, \sigma_{z,3}$
ν	4 - 3 - 1	$\sigma_{z,1}, \sigma_{z,2}, E, k_1$

Validation

Hydrostatic compression test

Bundle of 60 simulation (E, ν, k_1 fixed)

Stochastic sensitivity analysis Loading part × Unloading part

k_3 (ANN1)	4 - 2 - 1	$k_1, \epsilon_{peak}, \epsilon_{yield}, \epsilon_{load,25}$
k_3 (ANN2)	5 - 2 - 1	$k_4, \epsilon_{peak}, \epsilon_{yield}, \epsilon_{load,16}, \epsilon_{load,36}$
k_4	3 - 2 - 1	$k_3, \epsilon_{peak}, \epsilon_{unload,4}$

Validation

Figure 25. Micro-plane model for concrete: uniaxial compression and hydrostatic pressure tests to determine all model parameters, stochastic sensitivity analysis of parameters and neural network response

4. Controlled destruction of damaged structures and quick reconstruction

Once the damage assessment has confirmed that extent of the structural damage, one are left facing two different possible solutions. The first pertains to the controlled destruction of the structure that is damaged beyond the level that is possible to repair, and the second concerns the reparation and quick reconstruction of the damaged structural component or the complete structure. In this section, we briefly review the main steps that are necessary for either of these two tasks. More detailed presentations are given in keynote lectures of Breysse (2008) or Bruhwiler (2008).

4.1. CONTROLLED DESTRUCTION OF DAMAGED STRUCTURE

Any structure that is damaged beyond possible reparation or retrofitting, which has to be removed, posses quite a difficult problem if placed in the urban area where the uncontrolled destruction can induce to a significant damage to any neighboring structure. As briefly shown in this section, it is in fact more difficult to destroy the partially damaged structure in a controlled manner, than to understand the damage mechanisms and carry out the damage assessment. Namely, not only do we need to provide the reliable estimates of the damage mechanisms accounting for possible heterogeneities (as described in the previous sections), but also to carry out the simulation of the complete structural failure in post-stable regime. This kind of problem was studied both experimentally and numerically in a collaborative project involving four university teams in Germany (see Hartmann et al. [2008]), for the particular structure shown in Fig. 26.

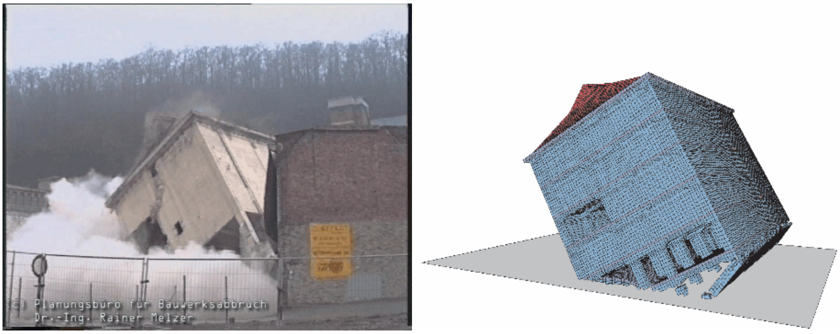


Figure 26. Typical residence building used for experimental and numerical (finite-element-method-based) studies of complete failure induced by controlled destruction

It is important to understand that the problem of this kind calls for the numerical modeling and simulation tools capable of going beyond the nonlinear structural mechanics applications into the area of multibody dynamics, so that we can predict the post-critical path of the failing structure. Moreover, we would also need the computational tools for handling the contact problems in order to predict the final position of the failing structures produced upon hitting the ground. These three phases carried out for the chosen building and the corresponding analyses are carried out by FEAP, MBS-code and LS-DYNA computer codes, respectively (see Hartmann et al. [2008]).

However, there is still one important phase missing from the analysis of this kind, which concerns any debris from the fallen structure big enough to induce potential damage in the neighboring structures in the urbane zone. However, this last phase of computation would again require a change of the computational model in that the discrete rather than finite elements should be used for the debris trajectories computations. Among available models of this kind, the most promising appears to be the models of Ibrahimbegovic and Delaplace (2003) or Delaplace and Ibrahimbegovic (2006), using the geometrically exact beams (e.g. see Ibrahimbegovic and Taylor [2003]) for cohesive force representation, which can handle both large displacements and rotations in the invariant manner; an illustrative example for failure modeling in dynamic bending test simulation is given in Fig. 27.

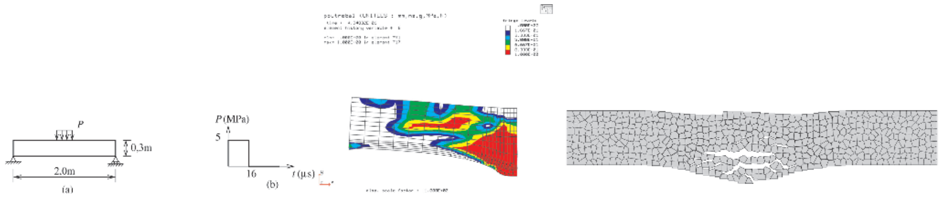


Figure 27. Simulation tools for post-failure modeling and debris computations, based either upon the finite element code (which only predict complete failure of heavily damaged zone) and discrete element code (which can also provide the complete trajectory of detached debris)

4.2. QUICK REPAIRMENTS AND RECONSTRUCTION

If the structural damage is reparable, one should carry out this kind of procedure by using modern materials, such as Ultra-High Performance Fiber Reinforced Concrete or Carbon Fiber, which can help us recover the structural integrity to a desired level. Several keynote lectures are focusing on this particular class of problems (see Bruhwiler [2008], Breysse [2008] or Owen et al. [2008]).

If the structures, however, are damaged beyond the reparation and would take much longer period of time to rebuild, one can then employ the temporary structures, which can re-establish the operations in the urban zones. Case in point is the deployable bridge in Fig. 28, which can be used for quick reconstruction of the city infrastructure.

*Figure 28***.** Deployable bridge in Hamanizuki Park

Such an example is quite typical of deployable structures that ought to be used repeatedly, which are often built as quite slender and lightweight. The latter implies that the deformation of the structure is sufficiently important and ought to be taken into account both in exploitation phase and in deployment. In fact, the deployment of such structure is often facilitated by the snap-through phenomena, which occur in-between the zero-stress-with-zero-displacement initial configuration and the zero-stress-with-finite-displacement final deployed configuration. Such a deployment procedure can be formulated and solved within the framework of real-time control problems, in terms of computing the most suitable sequence of actuators $\boldsymbol{\nu}$ that should bring the structure into the desired form $\boldsymbol{\Phi}$, or rather as close as possible to this form in terms of the chosen cost function, where we will also satisfy the equilibrium equations $G(\boldsymbol{\Phi};.) = 0$

$$\hat{J}(\hat{\boldsymbol{\phi}}(\boldsymbol{\nu}),\boldsymbol{\nu}) = \min_{\forall \boldsymbol{\nu}_t, G(\hat{\boldsymbol{\phi}}^*_t(\nu^*_t);\cdot)=0} \Big|_{\hat{\phi}_d} \hat{J}(\hat{\boldsymbol{\phi}}_t(\boldsymbol{\nu}^*_t),\boldsymbol{\nu}^*_t)$$

An efficient solution procedure can be constructed for this class of problems (e.g. see Ibrahimbegovic et al. [2004]), where the constraints defined through equilibrium equations is brought-up to the same level as the cost function by exploiting the Lagrange multiplier procedure and defining and solving the min–max problem with the following Lagrangian:

$$\max_{\forall \boldsymbol{\lambda}_t} \min_{\forall (\boldsymbol{\phi}_t,\boldsymbol{\nu}_t)} \Big|_{\bar{\phi}_d} L(\boldsymbol{\phi}_t,\boldsymbol{\nu}_t,\boldsymbol{\lambda}_t)\,; \quad L(\boldsymbol{\phi}_t,\boldsymbol{\nu}_t,\boldsymbol{\lambda}_t) = J(\boldsymbol{\phi}_t,\boldsymbol{\nu}_t)+G(\boldsymbol{\phi}_t,\boldsymbol{\nu}_t;\boldsymbol{\lambda}_t)$$

The solution for this kind of problem is rather difficult to obtain by the traditional solvers, since they often lead to a set of stiff nonlinear algebraic equations. Namely, the nonlinear mechanics problem is already prone to solution difficulties brought by an important difference between the soft, bending-dominated modes against the stiff, membrane-deformation modes; one can expect additional difficulties brought about by an arbitrary choice of the cost function, which can lead to a set of highly heterogeneous equations. We reported on two different solution procedures (e.g. see Ibrahimbegovic et al. [2004]), one based upon the surface response techniques constructed by a judicious application of diffuse approximation (e.g. see Brancherie et al. [2008]) and the genetic or rather evolutionary algorithms. The performance of this method is illustrated in a typical control problem presented in Fig. 29, where we can see that the dominant control components converge quickly during evolutionary algorithm, but the secondary components (whose contribution is far from being dominant) will require many more steps and would impair convergence.

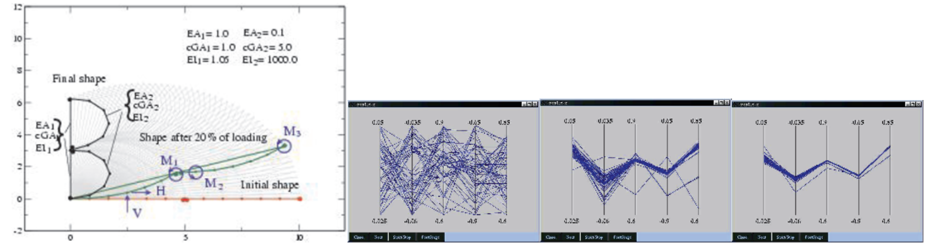


Figure 29. Control problem of placing flexible multibody system in final position, and selected results of solution procedure based on evolutionary algorithm presenting first, 4th and 18th step

5. Conclusions

In this works we have briefly reviewed the very broad class of problems focusing upon the damage assessment and reconstruction of structures and infrastructure in the urban zone, following natural disasters and previous military activities. The most thorough discussion is given for a reliable description of damage mechanisms, including the special form of damage for massive structure where the fracture process zone provides the same order dissipation as the macro-crack that is likely to endanger the integrity of a particular structural component.

We have also presented on how to account for heterogeneities of the material properties, as well as the heterogeneities induced by different loading history for each structural component and its nonlinear behavior. The most important result in this sense pertains to explanations of size-effect from the statistic on different interplays between main damage mechanisms.

The main methods for solving identification problems, both forward mode and backward mode identification are illustrated within the context of material parameters estimates for a particular damaged structural component. How to carry out the damage identification and assessment for a complete, complex structure is still an open question.

The problem of controlled destruction of a heavily damaged structure in a urban zone, without damaging the neighboring structures, brings about a very interesting combination of modeling tools, starting with the finite element models of inelastic nonlinear behavior, going over to multi-body dynamics systems and finally to the contact problem. If the debris area computation is of strong interest, we ought to use yet another model based upon the discrete models, with cohesive forces represented by the geometrically exact beams that can provide a reliable estimate of the mass and velocity of particles which split from the main structure.

The quick reconstruction and retrofit of damaged materials and structures is also an important goal for this class of problems. The reconstruction pertains both to reparation by special materials (such as Ultra-High-Performance Fibre-Reinforced Concrete), as well as the quick replacement of a destroyed structure or a heavily damaged structural component by deployable structures.

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