Simulation strategy for impact against a reinforced concrete industrial building

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Abstract

In the context of nuclear projects, Electricité de France (EDF) studies the mechanical consequences of impact loads on reinforced concrete shell structures. We developed a numerical methodology to simulate this kind of accident. Both local behaviour in the impact zone and vibration of the whole structure are considered even if they belong to different fields: during impact, the local behaviour is a fast dynamics issue with strong material non-linearity whereas the structure shaking is a slow dynamics linear elastic problem which is similar to a seismic analysis. To treat properly the physical phenomena taking place on different time scales, our modelling strategy is based on a domain decomposition method implemented into EUROPLEXUS, a fast dynamic software. The non-linear local behaviour is described by a material law using resultant shell variables and accounts for plasticity and damage.

Keywords: impact, reinforced concrete, shells, domain decomposition.

1 Introduction

In order to obtain realistic predictions of the dynamic behaviour of a building during impact in safety studies, it is important to consider both local damage next to the impact and the vibratory response of the global structure. Generally these issues are difficult to study simultaneously because they belong to different physical fields:

- The shock phenomenon, which generates a strong localized damage during about a hundred of milliseconds, is a fast dynamics issue. The complexity usually comes from the non-linearity of the material modelling. A fine mesh is required to represent the impacted area.
The shaking behaviour has to be studied during a few seconds because the energy injected and not dissipated locally at the impact produces some vibratory effects in the structure responding in a linear elastic way. This phase can be assimilated to a classical seismic analysis of a large size structure.

Let us consider the case of a reinforced concrete shell structure impacted by a large projectile at low or medium speed. According to the previous analysis, the most important features to simulate are obviously:

1. the strong non-linearity of the reinforced concrete
2. the simultaneous computation of the localised non-linear problem on a refined mesh and of a linear elastic problem on an industrial size structure mesh.

We present in this paper the methodology allowing us to solve efficiently the whole problem and we show its application to an industrial size case.

2 Non-linear behaviour of reinforced concrete shells

2.1 General description

Since the 3D constitutive models of concrete in FEM programs are often heavy and slow from the computational viewpoint, we developed a simple material model for the composite orthotropic RC shells, using resultant shell variables ($N$ membrane force; $M$ bending moment; $\varepsilon$ average strain in the thickness; $\kappa$ curvature). This model, called GLRC (GLobal Reinforced Concrete) is built to comply with the bending behaviour of a beam under monotonic loading (Figure 1) and to take into account the influence of the membrane force on the yield moment $M_p$ (Figure 2) [1].

This behaviour, extended to plates, implies both damage (to model concrete cracking) and plasticity (to model steel reinforcement yielding) in bending. Indeed bending seems to be the most characteristic behaviour of the impacted RC structure, compared to the membrane effects which are supposed to remain elastic. Damage and plasticity are thermodynamically uncoupled [2].
2.2 Damage

An isotropic damage has been introduced to the model in order to represent the cracking of concrete and the stiffness recovery in case of cyclic loading [3]. Two damage scalar variables are used (one for cracks on each face of the RC plate). If we consider the free energy $\Phi$ for bending:

$$
\Phi(\kappa, \kappa^p, \alpha, D_I, D_{II})
= a \left[ \tau (\kappa - \kappa^p) \right]^2 \xi(\tau (\kappa - \kappa^p), D_I, D_{II}) + b \sum_i \left( \kappa - \kappa^p \right)^2 \xi_i \left( \kappa - \kappa^p \right), D_I, D_{II} \right] + \frac{1}{2} C : \alpha : \alpha
$$

(1)

$A, b, C$ material characteristics

$\kappa^p, \alpha$: internal variables for plasticity

$D_I, D_{II}$: internal variables for damage

$\xi(x, D_I, D_{II})$ is a function that distinguishes positive and negative bending damage. This function is defined by:

$$
\xi(x, D_I, D_{II}) = \frac{1 + \gamma D_I}{1 + D_I} H(x) + \frac{1 + \gamma D_{II}}{1 + D_{II}} H(-x)
$$

(2)

with $\gamma < 1$ and $H$ a Heaviside’s function:

$H(x) = 0$ if $x < 0$ et $H(x) = 1$ if $x \geq 0$.

The state relations describing the material behaviour can be classically written as:

$$
\begin{align*}
M &= \frac{\partial \Phi}{\partial \kappa} \\
m &= \frac{\partial \Phi}{\partial \alpha}
\end{align*}
$$

and

$$
\begin{align*}
Y_I &= \frac{\partial \Phi}{\partial D_I} \\
Y_{II} &= \frac{\partial \Phi}{\partial D_{II}}
\end{align*}
$$

(3)

Where $Y_I$ and $Y_{II}$ are the thermodynamic forces linked to the damage variables $D_I$ et $D_{II}$ and $m$ the backmoment (for a kinematic hardening).

The damage criteria are defined by :

$$
\begin{align*}
g_I(Y_I) &= -Y_I(\kappa, D_I) - k_I \leq 0 \\
g_{II}(Y_{II}) &= -Y_{II}(\kappa, D_{II}) - k_{II} \leq 0
\end{align*}
$$

(4)

A typical GLRC behaviour under cyclic loading is shown on Figure 3:

2.3 Plasticity

For the plastic behaviour of the plate, which usually occurs when the steel reinforcement yields, we use a Johansen criterion ([4] and [5]):

$$
f(M) = -\left( M_x - M_{px} \right) \left( M_y - M_{py} \right) + M_{xy}^2 \leq 0
$$

(5)
According to Figure 2:, we make the yield moment depend on the membrane force \([1]\):

\[
M_p = M_p(N)
\]  

(6)

As a consequence, the material model is not standard.

The bending plastic characteristics (double Johansen’s criterion, one for positive and one for negative bending (Figure 4:), kinematic hardening with the backmoment \(m\) depending on the plastic curvature \(\kappa_p\), and normal law for the plastic flow) are expressed as:

\[
\begin{cases}
 f_I (M - m) \leq 0 & \text{positive bending} \\
 f_{II} (M - m) \leq 0 & \text{negative bending}
\end{cases}
\]  

(7)

\[
m = C \kappa_p
\]  

(8)

\[
\dot{\kappa}_p = \lambda_1 \frac{\partial f_I}{\partial M}(M,m) + \lambda_{II} \frac{\partial f_{II}}{\partial M}(M,m)
\]  

(9)

3 Domain decomposition method

3.1 General description

The domain decomposition method implemented in **EUROPLEXUS** fast dynamics software \([6]\) can take into account:

- different time-steps in the subdomains. It allows choosing a time-steps that respect the stability of the time integration scheme in each subdomain of the structure.
- non-matching meshes at the interfaces of the subdomains. That makes possible the mesh refinement in the impact area without drawback for the rest of the mesh.

After space and time discretization with an explicit integration scheme, the dynamic equilibrium can be expressed as:

\[
M \ddot{U}^{n+1} = F_{\text{ext}} - F_{\text{int}}(U^{n+1}) = F^{n+1}
\]  

(10)
where $M$ is the diagonal mass matrix, $\ddot{U}^{n+1}$ the acceleration at time $t_{n+1}$ and $U^{n+1}$ the displacement at time $t_{n+1}$ deduced from the known quantities at time $t_n$.

The whole structure is split into two subdomains. The consistency of the global problem is enforced by the continuity conditions at the interface:

- the equilibrium of the interface forces:
  \[ F_1 + F_2 = 0 \]  
  \[ \text{(11)} \]

- the kinematic continuity:
  \[ \dot{U}^1 = \dot{U}^2 \]  
  \[ \text{(12)} \]

To deal with the continuity at the interface, the dual Schur method is used: the equilibrium of the interface forces is imposed \textit{a priori} and we check \textit{a posteriori} the kinematic continuity, which is expressed with velocity at the interface (rather than displacement) to get a better stability of the integration scheme.

3.2 Characteristics of the multi-time scales

In case of a computation with constant time-steps [7], time-step on subdomain 1 must be a multiple of time-step on subdomain 2 (Figure 5):

\[ \Delta t_{sd1} = m \Delta t_{sd2} \]

![Figure 5: Constant time-steps.](image)

In explicit dynamics variable time-steps are more widely used, in order to adapt the time-step to the actual stiffness and length of the elements. In such a case, it is necessary to impose enough integration instants common to the different subdomains (Figure 6):

![Figure 6: Variable time-steps.](image)

3.3 Characteristics of the subdomain meshes

In case of matching meshes, the kinematic continuity at the interface leads to simple nodal relations coupling all the degrees of freedom of the facing nodes. Concerning the non-matching meshes, it is necessary to set special bonding
relations to ensure the kinematic continuity. The mesh bonding principle used is detailed in [8] and [9].

Two kinds of bonding methods have been implemented in EUROPLEXUS:

- The optimal method, which produces a perfect kinematic continuity at the interface. Nevertheless, when there are too few coincident nodes, an artificial added stiffness may appear.
- The Mortar method, which does not involve a perfect bonding between the subdomains. But this method allows getting the right stiffness at the interface.

When a hierarchical mesh is used (when the interface mesh of subdomain 2 is obtained subdividing the interface mesh of subdomain 1) the two methods are identical.

It has been shown on a preliminary simplified study that a good use of these methods (for example it is necessary to put the incompatible interface far from the high stress gradient areas [9]) can divide by 8 the CPU time comparing to a mono-domain calculation.

4 Industrial size case

The strategy described previously has been applied to an industrial size case. Computation has been performed with the fast dynamics code EUROPLEXUS [6].

4.1 Mesh

The building has been discretized essentially with plate elements. The mesh is made of 7270 DKT elements (1022 DKT for the refined zone) and 5425 Q4γ4 elements.

A local mesh refinement routine has been developed: starting from a given impact point, it splits by extrusion the original mesh in two parts; the part around the impact point is then refined. In our case, the meshes are incompatible (non-matching) still remaining hierarchical (Figure 8:).

![Figure 7: Original mesh.](image1)

![Figure 8: Modified mesh around the impact.](image2)
4.2 Material characteristics, boundary conditions and loading

Far from the impact zone, the concrete and steel characteristics remain elastic. In the impact area, the parameters of the GLRC model are determined thanks to the beam behaviour (similar to Figures 1 and 2), that can be obtained analytically or with a simple software. These parameters are detailed in Table 1. The damping matrix used in the computation is proportional to the mass matrix (7% for 5 Hz). The soil-structure interaction is modelled with springs distributed along the foundation of the structure. The springs characteristics have been previously computed with Code_Aster [10], starting from global stiffness and damping prescribed by French rules.

We consider a load distributed on a disc around the impact point with the time history shown on Figure 9:

![F(t) vs t](image)

Figure 9: Loading.

Table 1: Material parameters for GLRC.

<table>
<thead>
<tr>
<th>p₁ slope before cracking</th>
<th>8010 MN.m²/ml</th>
</tr>
</thead>
<tbody>
<tr>
<td>p₂ slope after cracking</td>
<td>1220 MN.m²/ml</td>
</tr>
<tr>
<td>pₚ plastic slope</td>
<td>14.6 MN.m²/ml</td>
</tr>
<tr>
<td>Cracking moment Mᵢ</td>
<td>1.24 MN.m/ml</td>
</tr>
<tr>
<td>Yield moment Mᵧ (for N=0)</td>
<td>2.90 MN.m/ml</td>
</tr>
</tbody>
</table>

![Vertical displacement at the impact point](image)

Figure 10: Vertical displacement at the impact point.
4.3 Results

In order to analyse the local damage next to the impact zone, we plot the displacement at different points (Figure 10:), the cracking level (0) and the plastic curvature (0) in the non-linear area. We check that the non-linear behaviour does not reach the boarders of the refined subdomain, which means that this area is large enough to take into account the local non-linearity.

Figure 11: Cracking on the external side (at 1 s).
Figure 12: Plastic curvature $\kappa_{00}$ (at 1 s).

Figure 13: Acceleration within the structure.
Figure 14: Acceleration spectrum within the structure.

For the vibratory analysis of the building, we can get the accelerograms (0) and the floor spectra (0) within the structure. It is also of strong interest to evaluate the energy dissipated during the impact, comparing to the total energy received by the structure (Figure 15:).
5 Conclusion

The realistic dynamic analysis of a building under impact requires to deal simultaneously with shock and shaking. In order to simulate the local non-linear behaviour as well as the vibratory response, we use a domain decomposition method. On one hand we concentrate on the damage and plastic concrete behaviour next to the impact point by developing a global resultant shell variables model. On the other hand, far from the impact zone the structure is assumed to remain elastic. With this modelling strategy, it is possible to get relevant predictions in terms of response spectra for a very large industrial building. The next research steps will be to consider membrane plasticity and then perforation of the reinforced concrete shell.

References


