Numerical modeling of rockfall impacts on reinforced concrete slabs for the design of new rock sheds

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ABSTRACT: In mountainous areas where the falling rocky blocks constitute a major hazard to civil structures, the structural systems to protect roads and vehicles are usually rock sheds composed of over dimensioned reinforced concrete elements. This is mainly due to the lack of knowledge of the dynamic effects caused by these falling rocks. A thick backfilling layer that prevents the direct impact of falling rocks by constituting a damping medium commonly covers the roof slab of protection structures. This allows the design of the slab with static dead loads (backfilling, concrete roof and rock weights). Recent experiments were performed in Chambery France on a new type of protection system characterized by a roof slab without damping medium (no backfilling) and simply supported on vertical elements by a set of "fuse" steel supports. The roof slab resists directly the falling rocks impacts, which cause limited local damage to the impact zone, in case of field impact, or the yielding of steel supports for boundary impact cases. The main advantage of this new protection system is to allow continuous uses of the structure trough the repair of the damaged zones in the roof slab or the replacement of the "fuse" steel support after each impact. The aim of the present study is to predict the structural response of the new proposed system by a rigorous three-dimensional modeling of the roof slab and its supporting elements. The analysis introduces the impact load in a way similar to that of the performed experiment, and a stress-strain concrete relationship that allows a realistic representation of the concrete behavior under dynamic loads and its corresponding damages. The comparison of the experimental measurements with those obtained from the present analysis proves the accuracy of the built model in predicting the real behavior of the protection structure.

Keywords: rock-fall, impact, concrete, experiment, modeling, finite element method

1 INTRODUCTION

The continuous expansion of urban zones in mountain regions increases the need of protection systems that protect the civil structures and infrastructures from natural hazards such as snow avalanches, falling rocks, and landslides. The dimensioning and designing of reinforced concrete protection systems is based, in the current codes of practice, on approximate method that consider the straining forces as static forces. The use of approximate static analysis methods leads either to over-dimensioned protection systems (massive concrete elements in usual gallery protection systems against rock fallings), or to under dimensioned systems.

The optimal dimensioning of protection systems (strength + limited cost) should be based on the following conditions:

- Considering the dynamic feature of the loading.
- Using behavior properties for the structural materials that allow an accurate description of the stress-strain relationship variation under the applied dynamic loads.
- Performing numerical structural analysis that includes realistic problem data (three dimensional geometry, generation of dynamic effects due to impacts, non-linear behavior…).

The present study focuses on the gallery rock falling protection system, conventionally composed of reinforced concrete sub-structural elements (walls, columns, and foundations) and a roof slab covered by a thick backfilling layer (Figure 1). The roof slab is rigidly connected to sub-structural elements, and the backfilling layer constitutes a damping medium, and allows therefore, the design of the system with only static dead loads (own weight, backfilling and rock weights).

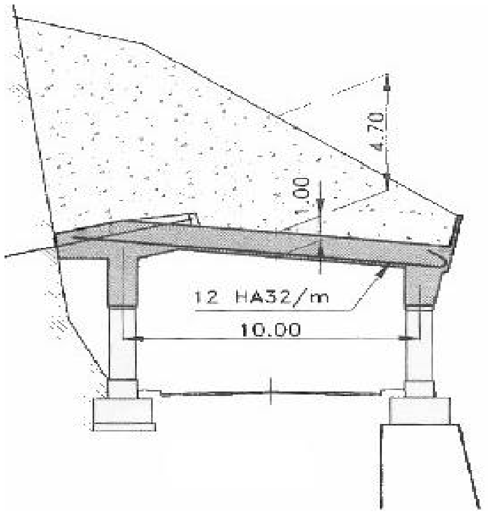

Figure 1. Schematic view of a conventional rock shed

The structure is not designed any more to resist the impact of blocks but especially to support the backfilling layer. This solution has the main disadvantage of producing over dimensioned reinforced concrete elements. The foundations, which must be dimensioned consequently, cause often some site construction problems as for the case of a raised slope, a weak soil…

Considering that the request for this type of equipment will be increasing, an investigation was carried out to improve the design and limit costs. The basic idea was to eliminate the backfilling layer and to use a semi-probabilistic approach with the notion of ''acceptable damage'' to the structure.

For the purpose of finding an optimal solution, a new system was proposed in France by the consulting company TONELLO IC, which consists of a roof slab pin supported (no continuity) on the sub-structural elements. The roof slab is subjected to the direct impact of falling rocks and slab reactions are transmitted to the sub-structures throughout ductile steel supports that act as dissipating energy fuses and protect the sub-structural elements (Figures 2 and 3). The slab is then designed in order to resist directly to a falling rock impact that causes a local damage limited to the shock zone in case of field impact

The first example of this protection system was built in 1999 at ''les Essariaux'' between Albertville and Chamonix in French Alps.

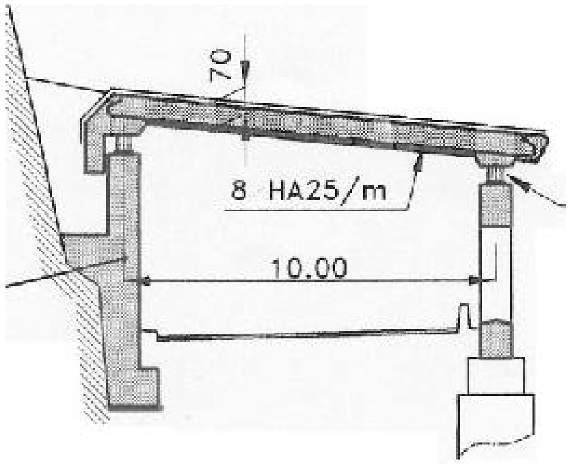

Figure 2. Schematic view of the new rock shed system

Figure 3. Photography of the new rock shed system - gallery "les Essariaux" - and a fuse support

2 EXPERIMENTS

During June and July 2001, the TONELLO IC company has performed experiments in collaboration with the LOCIE (University of Chambery) on a one third reduced scale system model of the roof slab of the proposed new system in order to evaluate the response and the performance of this system.

The experiment consisted of a concrete slab (12 x 4.4 x 0.28m) set on two lines of 11 steel fuses (12.6 cm high and spaced at 1.14 m) and impacted by a 450 kg reinforced concrete cubic block (58 cm side) falling from different heights to various locations of the slab.

The slab dimensioning and reinforcement was performed using a simplified method based on the principle of momentum and energy conservations

(Tonello 2001; Perrotin *et al.* 2002). Assuming that the governing flexural deformed shape is the first mode of vibration for the impacted slab, the slab mass is substituted by an equivalent mass M* (figure 4):

$$M^* = \int \frac{y_i^2}{y_0^2} dm \qquad (1)$$

Where y_0 is the amplitude of the flexural deformation of the slab, y_i is the displacement of the partial mass dm of the slab.

Assuming also that after the impact there were neither a bounce of the projectile nor a penetration into the slab, the principle of momentum and energy conservations leads to the following relation:

$$E\frac{m}{M^*+m} = P_u\,(0.5f_e+f_p) \qquad (2)$$

Where m and E represent the mass and the kinetic energy of the mass respectively. The right hand side represents the dissipated energy in the slab; P_u is the peak impact force, f_e and f_p represent the elastic and plastic strains respectively assuming an elastic plastic behavior of the slab with no hardening. The slab thickness and the reinforcement bars are then calculated with the static force P_u according to service or ultimate limit states (Figure 4).

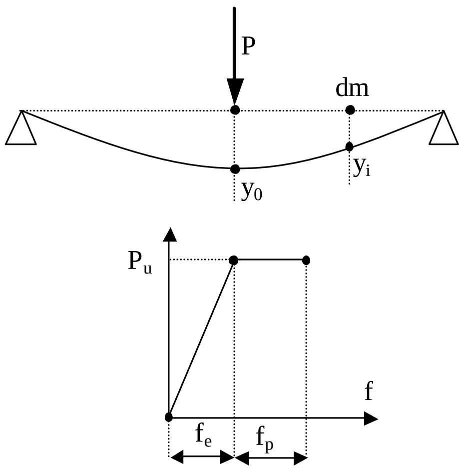

Figure 4. Simplified model used for the design of the concrete slab

The concrete block was released to fall freely from 15 and 30 m high, and to impact the slab, almost on one of its faces. The impact velocity varied from 17.2 to 24.2 m/s and the kinetic energy from 67.7 to 135 kJ (Figure 5).

Three impacts were carried out: the first and the second from 15 and 30m high in the inner part of the slab and the third from 30m on the edge of the slab (the support line).

The slab response to the different impacts was obtained through general displacement measurements and with strain gauges.

These relatively large-scale experiments provide interesting and complete data that allow better understanding of the structural response to impact loads. They also allow the evaluation of the numerical analysis performance of the modeled structure.

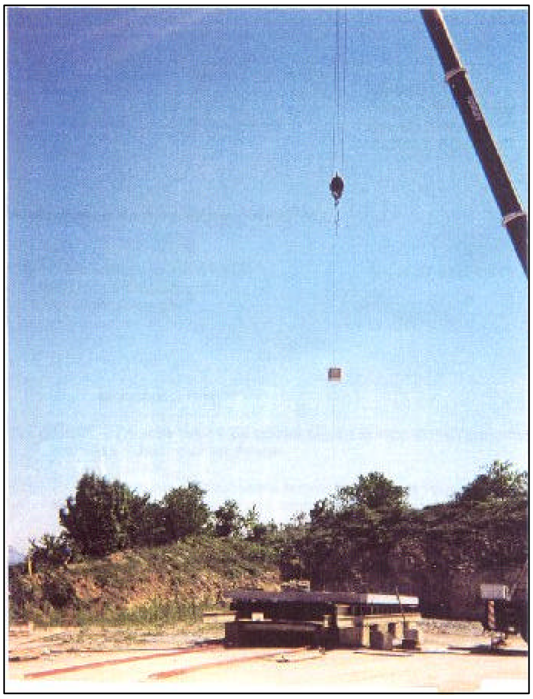

Figure 5. Rock fall test

3 NUMERICAL ANALYSIS

A realistic prediction of the structural response through numerical analysis requires rigorous three-dimensional finite elements modeling of the different structural components. In the present study, the finite elements code Abaqus was used. The explicit module of this code allows highly non-linear transient dynamic analysis of phenomena like impacts.

Abaqus offers also the possibility of managing several interactive entities (the slab and the block in the present case). The analysis can, therefore, introduce the impact in a way similar to that of the experiment, managing only the impact characteristic.

3.1 *Finite element modeling*

The slab and the block were modeled with volumetric finite elements (Figure 6) with different degree of mesh refinement (essentially at the impact zones of the slab). The reinforcement was represented by bar elements or with the Abaqus "rebar" option that uses a superposition of stiffness matrix method.

The steel supports were modeled using non-linear springs in a first (preliminary) analysis, then with volumetric elements in a second (refined) analysis (Figure 7).

The constitutive behavior of the non-linear spring was identified through the knowledge of both the cross-section area of the steel support and its stress-strain relationship.

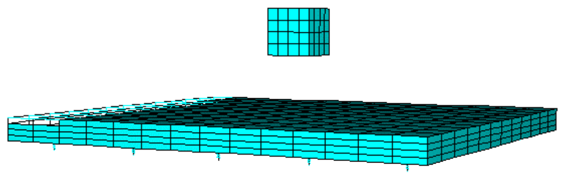

Figure 6. Three dimensional modeling of the slab and the block

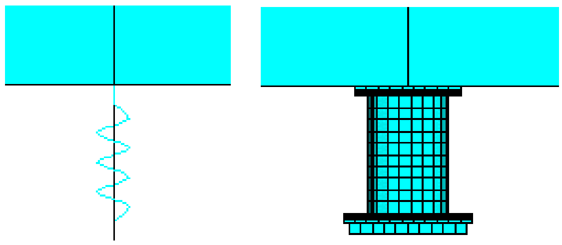

Figure 7. Non-linear spring and 3D model of a steel support

3.2 *Constitutive behavior modeling of concrete*

For an accurate simulation of the structural response, it is necessary to use a realistic representation of the materials behavior under dynamic loads. For the concrete, the behavior properties must include some phenomena that are related to the damage under dynamic loads such as decrease in material stiffness due to cracking, stiffness recovery related to closure of cracks, and inelastic strains concomitant to damage.

The stress-strain relationship was represented in the numerical analysis by the PRM (Pontiroli-Rouquand-Mazars) (Pontiroli 1995; Rouquand and Pontiroli 1995) damage model that uses one scalar damage variable D, which is the damage indicator.

The one-dimensional expression of stress-strain relationship is the following :

$$(s - s_{ft}) = E_0\,(1-D)(e - e_{ft}) \quad (3)$$

Where E_0 is the Young modulus, σ_{ft} is the crack closure stress in tension and ε_{ft} is the irreversible strain corresponding to σ_{ft}. A similar expression is used with tensors to describe the three-dimensional states.

D includes damages due to compression and tension; its value varies from 0 (for uncracked material) to 1 (for macro-cracked material). The variation of D is governed by the equivalent strain $\tilde{\varepsilon}$ (Mazars 1984 and 1986):

$$\tilde{\varepsilon} = \sqrt{\sum_{i=1,3} \langle \varepsilon_i \rangle_+^2} \quad (4)$$

Where $\langle \cdot \rangle_+$ denotes the positive part and e_i are the principal strains. The damage variable D is calculated through the damage indicators in tension (D_t) and in compression (D_c):

$$D = a_t^{\beta} D_t + (1-a_t)^{\beta} D_c \quad (5)$$

The damage evolution is given by:

$$D_a = 1-(1-A_a)\frac{\varepsilon_o}{\tilde{\varepsilon}} - A_a\; e^{-B_a(\tilde{\varepsilon}-\varepsilon_o)} \quad (6)$$

With α = c,t ; σ_{ft} and ε_{ft} are calculated with:

$$\sigma_{ft} = E_o\;\; (1-D_c)(\varepsilon_{ft} - \varepsilon_{fc}) + E_o \varepsilon_{fc} \quad (7)$$

$$\varepsilon_{ft} = \varepsilon_{ft0}(1-D_c) - \frac{D_c}{1-D_c}\varepsilon_{fc} \quad (8)$$

σ_{fc} and ε_{fc} are considered as material parameter data.

The threshold e_o depends on the strain rate $\dot{e}$ in order to model the strain rate effect under dynamical loading.

A typical stress-strain response produced by this model for a uni-axial alternate loading in multi traction-compression steps is given in figure 8. The three dimensional version of this model (used in the numerical analysis) was implemented in Abaqus-Explicit using an external Fortran subroutine.

The stress-strain relationship for the reinforcing bars is considered as simply elastic plastic, with hardening or without hardening.

The material parameters of the model were identified from classical data for concrete (f_c = 30 MPa, E_0 = 30 GPa) and rebars (f_s = 500

MPa) where f_c and f_s are the ultimate strengths of concrete and steel.

The Hillerborg *et al.* (1976) regularization technique is used in order to avoid mesh dependency.

4 ANALYSIS RESULTS

A wide range of results was obtained from the different loadings cases (impacts locations and falling heights), the modeling of steel supports (non-linear springs versus 3D complete model), and the reinforcement modeling (rebar versus bar elements).

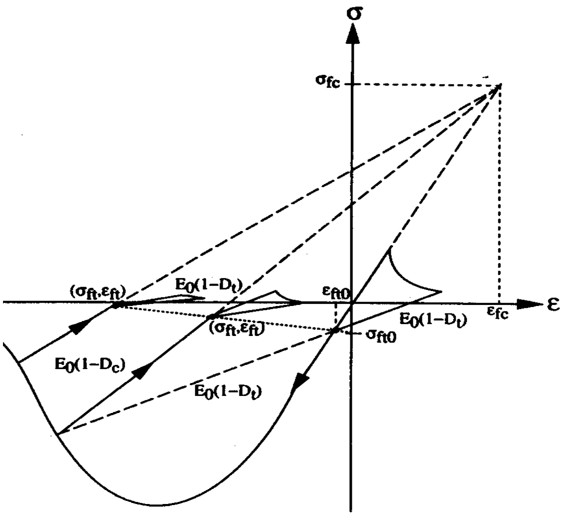

Figure 8. PRM modeling of the concrete behavior

Figure 9 shows the transversal profiles of the measured maximum slab deflection. Figure 10. gives evolution with time of the computed transversal profile of the slab deflection.

Figure 11 shows the computed vertical displacement of the slab impact point. The comparison of the analysis results with the experiment measurements showed global agreement, which indicates a good performance of the model in predicting the real behavior of the structure.

The comparisons between measured and calculated maximum values of vertical displacements of the slab at the different impact points are given in Table 1. For the 30 m high test in the inner part, the prediction of the maximum vertical displacement with an elastic behavior for concrete gives a 60% lower result confirming the necessity of modeling the non linear behavior of concrete.

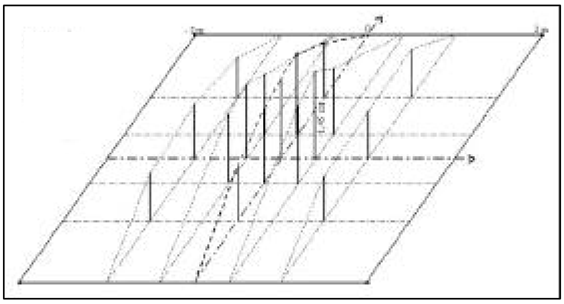

Figure 9. Transverse profiles of measured maximum vertical displacements

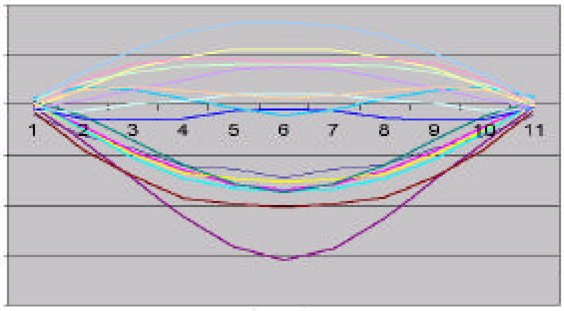

Figure 10. Predicted transversal profile of vertical displacements in the slab versus time

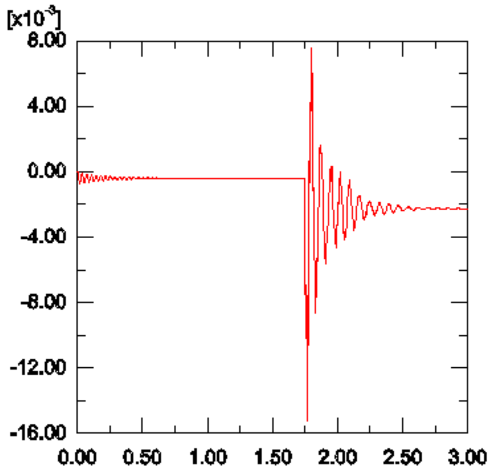

Figure 11. Displacement of the bottom face of the slab at the location of impact versus time

The displacements obtained from the third test (impact on the support line) were equivalent for the two models of supports. The 3D modeling gave a deformed shape similar to that obtained in reality (Figure 12).

Table1. Experimental and numerical vertical displacements

Test and impact point	Measurements (mm)	Model results (mm)
15 m high inner part	14.5	15.2
30 m high inner part	23.0	23.6
30 m high support line	21.5	21.2

The advantage of this type of analysis, in addition to traditional analysis output (strain, stress, internal forces, reactions…), is to allow obtaining additional type of results throughout the implemented subroutine. In particular evaluating the damage states by mapping the values of D in the slab. Figure 13 shows the observed damage localization, i.e. the cracks on the sub-face of the slab due to different impacts.

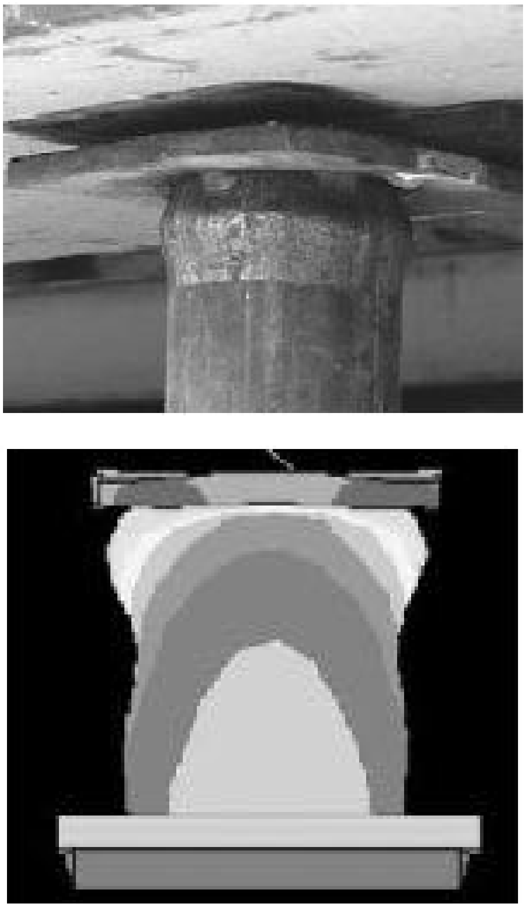

Figure 12. Deformed shape of the experimental and modeled steel support

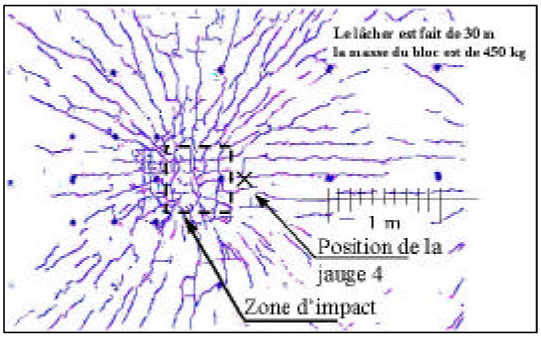

Figure 13. Mapping of damage distribution in the slab

The repartition of damages within the slab thickness can also be obtained. Whereas further investigations has to be performed into the relationship between D and the real state damage, initial comparison of experimental and numerical results for the described tests showed already an interesting qualitative agreement between damage repartition obtained from the analysis and that observed with the experimentation as shown in Figure 14 (spalling, radial cracking….). Note that the damage due to the three tests was cumulated by simulating the three impacts performed on the same slab.

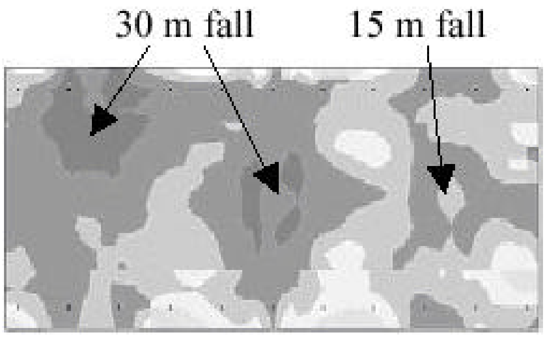

Figure 14. Damage pattern due to tension (D_t eq. (5)) on the sub-face of the slab. The darker the slab, the more damaged it is.

5 CONCLUSION

The evaluation of the performance of the new proposed system for reinforced concrete galleries was the main aim of this study. This was achieved by a numerical analysis including: a 3D model of the structural elements (block, slab, and the steel supports), a damage mechanics based model for concrete, and a realistic representation on the straining actions (different impacts of the projectile).

The numerical analysis allowed also the evaluation of the influence of different parameters as the impact velocity of the block, the block mass, the bearing capacity of the fuses, the material characteristics…

The analysis results proved to be in agreement with the experimental measurements, as well as representing the damage states under the different cases of loadings.

Based on the obtained results it can be concluded that:

- The new proposed system well fits its uses: optimal dimensioning, ability to resist blocks impacts that cause reparable damages, this would be done without affecting the serviceability state of the structure,
- The numerical modeling represents a powerful mean in predicting the behavior of this type of structures, and can therefore be used for their design

Further research must however be performed to include additional cases such as the response of a repaired slab to new impacts, the structure response when subjected to several consecutive impacts.

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