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Analytical Model for Rockfall Protection Galleries - A Blind Prediction of Test Results and Conclusion

Kristian Schellenberg[1,a], Norimitsu Kishi[2,b] and Hisashi Kon-no[3,c]

[1]Institute of Structural Engineering, ETH Zurich, Zurich, Switzerland

[2]Muroran Institute of Technology, Muroran, Japan

[3]CERI for Cold Region, PWRI, Sapporo, Japan

[a]schellenberg@ibk.baug.ethz.ch, [b]kishi@news3.ce.muroran-it.ac.jp, [c]konno@ceri.go.jp

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Abstract. A system of multiple degrees of freedom composed out of three masses and three springs has been presented in 2008 for analyzing rockfall impacts on protective structures covered by a cushion layer. The model has then been used for a blind prediction of a large-scale test carried out in Sapporo, Japan, in November 2009.

The test results showed substantial deviations from the blind predictions, which led to a deeper evaluation of the model input parameters showing a significant influence of the modeling properties for the cushion layer on the overall results. The cushion properties include also assumptions for the loading geometry and the definition of the parameters can be challenging. This paper introduces the test setup and the selected parameters in the proposed model for the blind prediction. After comparison with the test results, adjustments in the input parameters in order to match the test results have been evaluated. Conclusions for the application of the model as well as for further model improvements are drawn.

Introduction

Improving the prediction of the structural response and of the ultimate capacity of rockfall protection galleries has been the aim of several research projects in the recent years. Reinforced concrete structures covered by a soil cushion layer are of main interest since they are the most common type of rockfall protection galleries.

The analytical model for the design of galleries proposed by Schellenberg [1] is based on estimating the force-displacement-relations for each of the three springs of a system of multiple degrees of freedom. The springs describe the properties of the cushion layer, the punching and the bending structural behavior, respectively. The force-displacement-relations contain simplifying assumptions that combined with overall idealizations of the impact process, e.g. rock shape, allow for estimating the dynamic impact resistance of protective structures. The model contains physical parameters only and tests carried out in 2007 in Walenstadt, Switzerland, have been used for verification [2].

The selection of the parameters describing the force-displacement-relationships is the main difficulty for the application of the model. For this purpose test data beyond the boundary conditions of the validated ranges is crucial for further improvements of the model.

In November 2009, a large-scale falling weight test has been carried out in Sapporo, Japan [3] and blind predictions were requested as part of a round robin test program. The here presented predictions were the only ones that have been submitted.

Fig 1: Falling weight impact test [3]

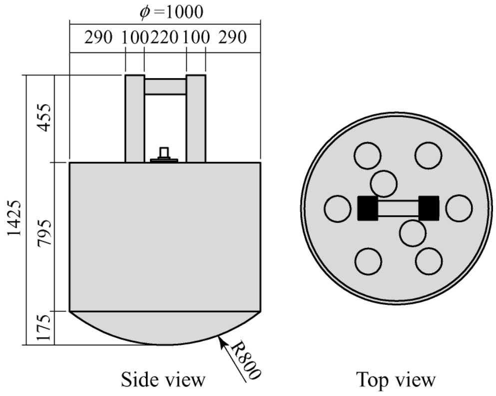

Fig 2: Geometry of falling weight [3]

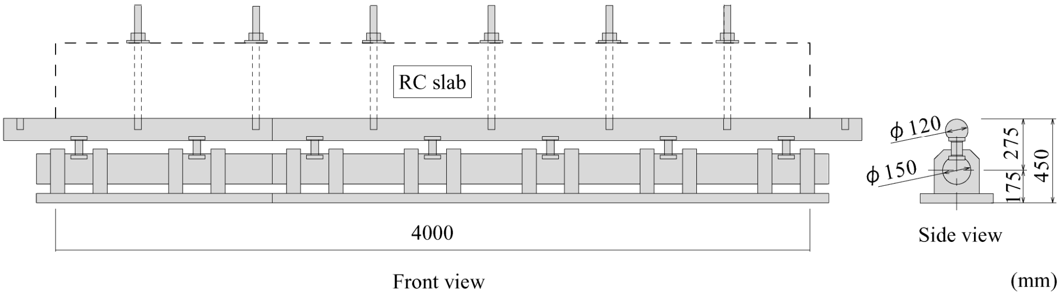

Fig. 3: Detail of hinged line support with tie down connections [3]

Setup of Falling Weight Test

The impact loading test was conducted at the test site of CERI in November 2009. The falling weight test consisted of a mass of 5000 kg, which had been dropped from a height of 10 m on a 0.4 m thick reinforced concrete slab with the side dimensions of 5 x 4 m. The concrete slab had been covered by a 0.5 m thick cushion layer of sand.

The reinforced concrete slab was reinforced with diameter 19 mm bars in spacing of 125 mm and had an effective depth of 340 mm. The slab was simply supported on two opposite sides with a clear span length of 4 m. The line supports allowed for rotations and the slab was tied down in order to avoid uplift after the impact. A detail of the support beams is given in Fig. 3.

The cylindrically shaped weight had a diameter of 1 m and a spherical bottom surface with a radius of curvature of 0.8 m (Fig. 2).

The compressive strength of the concrete was 34.2 MPa. Yield strength of the bending reinforcement was 393 MPa.

Analytical Model

The results of the falling weight impact test are predicted using a System of Multiple Degrees of Freedom (SMDF) as proposed in [1] for modeling the dynamic behavior of rockfall protection galleries covered by a cushion layer. The model consists of three masses and three nonlinear springs (Fig. 4). The three masses are defined as follows: M_1 is the mass of the impacting block, M_2 is the mass of the assumed punching cone under the loading location, and M_3* is the modal mass of the surrounding of the structure.

Three nonlinear springs define the force-displacement relationships, which represent the behavior of the cushion layer, the shear failure surface, and the global behavior of the structure, respectively.

For the cushion layer, a hardening soil behavior is assumed and approximated by a hyperbola with an initial stiffness K_{10} and a vertical asymptote, where the soil reaches complete compaction.

Spring K_2 describes the shear stiffness for the assumed punching shear failure of the concrete slab. With increasing relative displacements of the punching cone compared to the surrounding part of the slab, the behavior is controlled by three contributors: K_{21}: the contribution of the concrete, K_{22}: the contribution of shear reinforcement if existing, and K_{23}: the membrane effect of the bending reinforcement.

The third spring K_3 describes the global stiffness of the concrete slab taking into account the plastic behavior of the structure. The equation of motion is solved explicitly with the impact velocity as initial conditions.

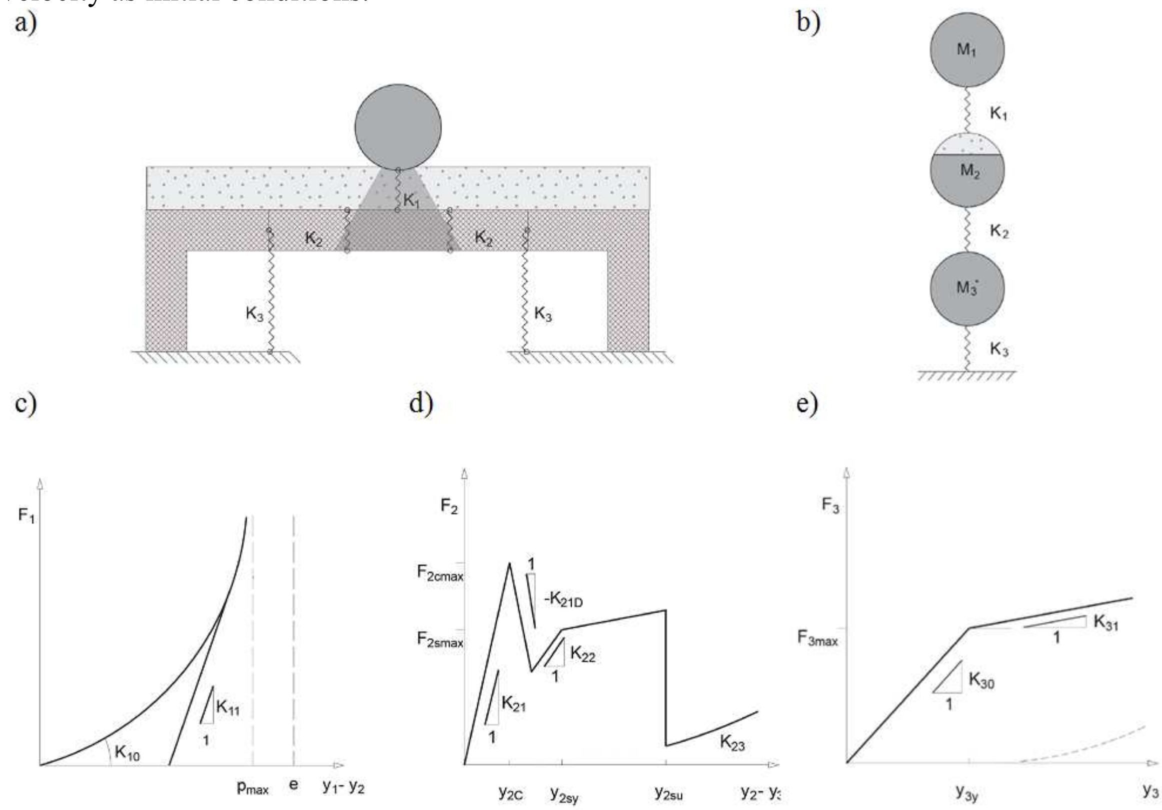

Fig. 4: System of multiple degrees of freedom (SMDF) a) and b) from the section of a gallery to the model definition and spring properties of c) cushion layer, d) for shear behavior and e) global bending stiffness [1].

Selection of Model Parameters

The model determines the masses and spring stiffnesses based on geometry and material parameters. For the relations between the parameters and the calculated masses and stiffnesses, it is referred to Appendix C in [1]. The following geometrical parameters are used directly as input parameter in the model: span of supports $L_x = 4$ m, length of slab $L_z = 4$ m, slab thickness $d = 0.4$ m, effective depth $z = 0.34$ m, concrete cover $\ddot{u} = \ddot{u}' = 60$ mm, diameter and spacing of bending reinforcement $D_1 = D_1' = 19$ mm and $s = s' = 125$ mm, diameter and spacing of shear reinforcement $D_w = 13$ mm and $s_w = 500$ mm.

The hardening properties of the reinforcement after yielding have the largest influence on the predictions of the maximum reaction forces if the slab reaches the plastic range during the impact. Due to lack of information on the post-yield behavior of the applied reinforcement, a global stiffness equal to $\gamma_y = 10\%$ of the elastic stiffness has been assumed after yielding.

In order to account for the two sides line supported slab with a span ratio of 1.0, a coefficient for the global stiffness of $k_w = 41$ has been selected, according to Table 4-2 in [1]. The yield load of the slab is considered by the value $P_u/m_u = 4$ as given in Table 4-3 of [1].

The mass factor for the global response is selected as $\alpha = 0.33$, as given in Table 4-1 of [1] for a plastic deformation shape and a ratio of spans of 0.8, since other values are not included in the table. For a span ratio of 1.0, the mass factor should be slightly smaller.

The loading condition is considered by an initial impact velocity of v_{10} = 14.01 m/s, corresponding to a falling height of 10 m and by the mass of the impacting body of M_1 = 5000 kg with a diameter of the loaded of D_s = 1 m.

The cushion layer is modeled with a thickness e = 0.5 m and a dynamic soil stiffness of 88000 N/mm calculated based on the cone model of Wolf [4] using a density of 15.7 kN/m^3 and a modulus of elasticity of 80000 MN/m^2, the contact area 0.78 m^2 and the cone height z_0 = 0.71 m.

A maximum penetration of p_{max} = 0.3 m is selected, which corresponds to a compressibility of the cushion sand of 20%. This is based on the assumption that the volume of the compressed sand below the impact position (wedge V_2) after the impact is 80% of the wedge with the passive earth pressure (V_1) see Figure 4-5 in [1].

For the reinforcement a yield stress of f_s = 393 MPa is used and for the concrete compressive strength f_c = 34.2 MPa is used.

Applying all parameters as explained above, the model determines the following masses and the spring stiffnesses corresponding to Fig. 4 as follows: M_1 = 5000 kg, M_2 = 2662 kg, M_3* = 3376 kg, K_{21}= 146.8 MN/mm with y_{2c} = 0.036 mm and K_{30} = 410 kN/mm with F_{3max} = 1178 kN.

Prediction and Comparison with Test Results

The blind prediction should contain the time history of the acceleration in the impacting mass, the time history of the reaction forces at the supports and the time history of the slab displacement at the loading point.

The predictions with the SMDF are displayed in Fig. 5, where the force-time histories of the impacting mass F1 and the reactions forces at the supports F3 are shown in Fig. 5a) and the displacements at the center y_3 of the slab is shown in Fig. 5b).

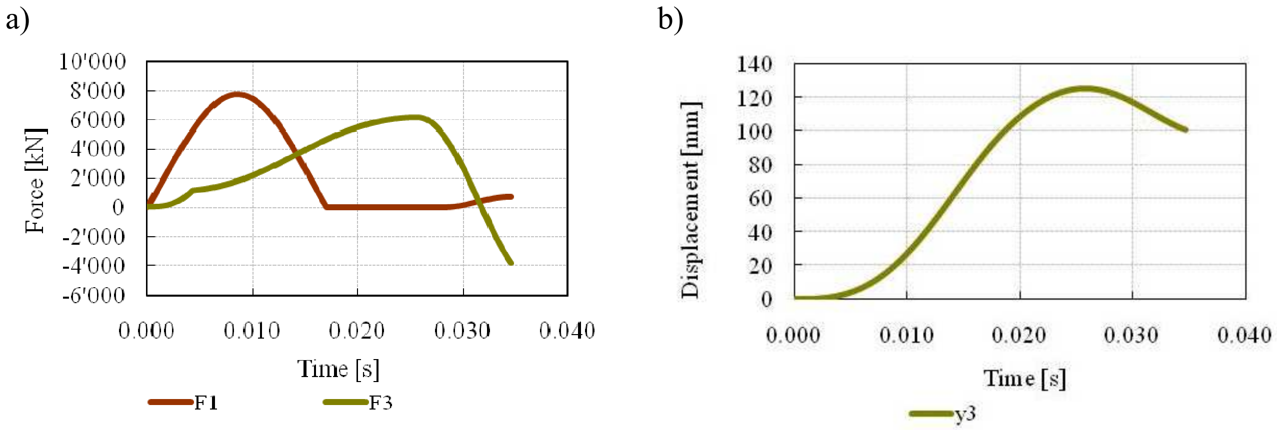

Fig. 5: Predicted time histories of: a) impact force (F_1) and reaction forces at support (F_3) and b) slab displacement at loading point y_3

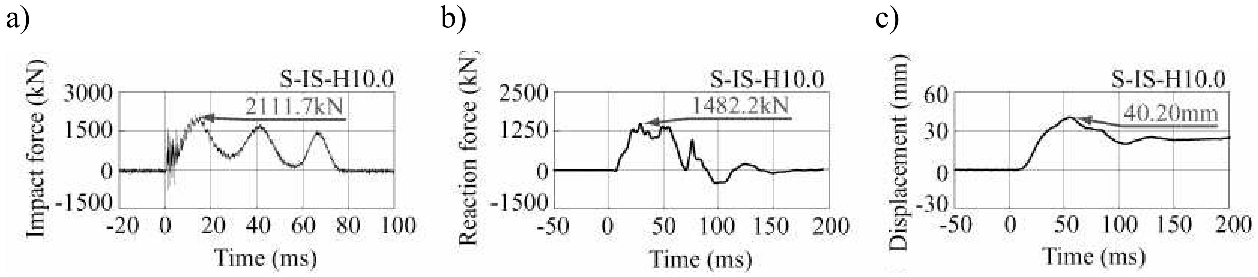

Fig. 6: Test results: a) impact force, b) reaction forces and c) slab displacement at loading point

Compared with the test results (Fig. 6), it is observed that the prediction overestimated forces by a factor of about 4 and underestimated the duration of the time histories significantly. Table 1 shows the comparison for impact force, reaction force and displacements, regarding the maximum value, the time of the maximum value and the total time.

Table 1: Comparison of test results and predictions

		Impact force	Reaction force	Displacements
Maximum	Test	2112 kN	1482 kN	40 mm
	Prediction	8000 kN	6000 kN	125 mm
	Ratio	380%	400%	310%
Time of maximum	Test	15 ms	30 ms	55 ms
	Prediction	10 ms	25 ms	55 ms
	Ratio	66%	83%	100%
Duration	Test	25 ms	90 ms	100 ms
	Prediction	17 ms	32 ms	70 ms
	Ratio	68%	35%	70%

Adjustments in Parameters and Interpretation

The disagreement between the prediction and the test results questions the definition of the input parameters for other test configurations than used for the model verification [2]. In this opportunity, curve fitting by adjusting the input parameters can be useful in attempting to identify the inappropriate assumptions within the physical model. By changing one parameter at the time the models sensitivity to the single input parameters can be observed. In order to obtain better agreement with the test results, the following four adjustments have been carried out and a physical meaning for change has been proposed. The changes in the force time histories due to the four adjustments in the parameters are documented in Fig. 7, which in respect of maximum forces and the duration of the impact are significantly closer to the test results than the predictions.

- Adj1: The maximum impact force is strongly overestimated and the impact time duration is underestimated. This means that the stiffness of the cushion layer has been overestimated. The applied model [4] contains the contact surface for defining the dynamic soil stiffness and the spherical shape of the bottom surface of the impacting body results in a much smaller area at the beginning of the impact and therefore a smaller stiffness can be justified. A reduction of the soil initial stiffness K_{10} from 88000 to 5000 N/mm results in a reduction of the maximum impact force from about 8000 kN to 4337 kN with an impact duration of 39.7 ms.
- Adj2: The maximum reaction force shows a strong increase after yielding. For a structural evaluation an overestimation of the hardening effect is on the safe side since failure is defined by an ultimate load bearing capacity. Therefore a more moderate value for the post yield hardening for the global stiffness of γ_y = 2% of the elastic stiffness is used instead of 10%. While the maximum of the impact force is slightly reduced from 4337 to 4124 kN, the maximum reaction force is significantly reduced from 3484 kN to 2359 kN.
- Adj3: The shape of the force time history of the impact force shows a progressive increase of the force development due to the hyperbolic shape of the assumed increase in stiffness of the soil for increasing penetration. In order to reduce this influence, which can not be found from the test results, the maximum penetration of the rock into the cushion layer $p_{\max}$ is increased from 0.3 to 0.4 m.
- Adj4: The damping coefficient of the cushion layer is submitted to a large uncertainty, where a value of 7000 Ns/m was applied in the model based on [5] and has not been further investigated during the verification phase of the model. In order to reduce the maximum impact and reaction forces, the damping coefficient is increased here by a factor of 5 from 7000 Ns/m to 35000 Ns/m. A physical model for defining the damping coefficient would be

required in order to improve the model. In general, the damping is required to dissipate the impact energy introduced into the structure, which is reflected by the integrated impact force through penetration. For the application in this model damping could be evaluated from future test results. This adjustment reduces the impact and the reaction force to F_{1max} = 2429 kN and F_{3max} = 2361 kN, respectively.

a) b)

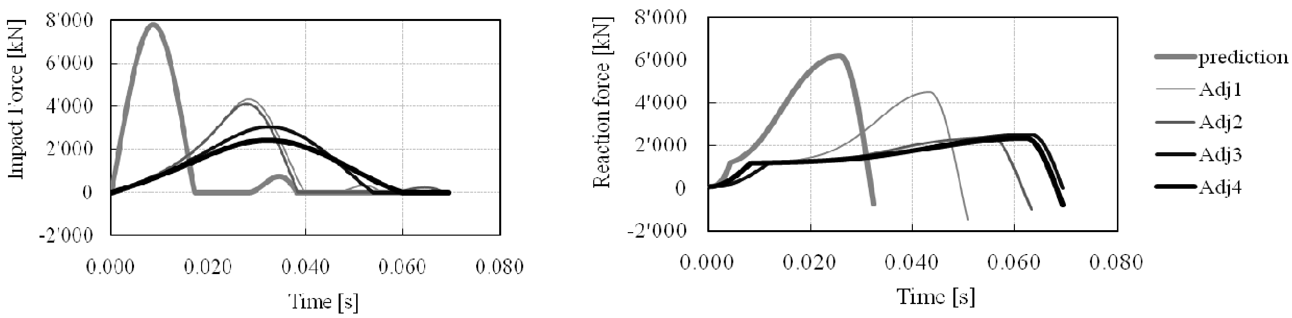

Fig. 7: Changes in the force time histories due to the adjustments in the single parameters a) Impact force (F_1) and b) reaction forces at support (F_3)

Conclusions and Outlook

This paper has shown an independent verification of a model following a blind prediction of test results. In particular due to different loading conditions and the properties of the cushion layer, compared with the tests used for the previous verification of the model, the predictions were rather dissatisfactory. The presented adjustments in the input parameters have demonstrated the importance of the proper definition of the properties of the cushion layer, whenever predicting potential structural failure of evaluated protective structure. The main model uncertainties are identified in the force-penetration-relationship of the rock into the cushion layer. It can be concluded that for a more reliable prediction of the structural performance of a protection gallery by means of the proposed analytical model, an accurate description of the cushion layer properties is crucial.

Thus further experimental research programs should also focus on determining the force-penetration-relationship of rocks into different cushion layers considering the involved parameters related to the cushion material, cushion layer thickness, compaction as well as the size and geometry of rock and others.

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