12^{th} International Conference on Fracture Mechanics of Concrete and Concrete Structures
FraMCoS-12
B.L.A. Pichler, Ch. Hellmich, P. Preinstorfer (Eds)

https://doi.org/10.21012/FC12.1321
MS05-2:2

NUMERICAL SIMULATION OF FLEXIBLE ROCKFALL BARRIER POST FOUNDATIONS SUBJECTED TO IMPACT

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Key words: Flexible rockfall barrier, Impact, Concrete, Gradient-enhanced continuum

Abstract. Rapidly changing climatic conditions confront communities in alpine regions with great challenges and necessitate the continued development and optimization of geohazard mitigation solutions. One such solution is presented by flexible rockfall barriers, which constitute a frequently employed system for mitigating the risk posed by rockfall. Despite their widespread use, the design of their foundations is characterized by great uncertainty. Moreover, the construction of such foundations requires heavy machinery and constitutes a major part of the cost of the completed barrier. To optimize the design of rockfall barrier foundations, a better understanding of their mechanical behavior is needed. To this end, a three-dimensional finite element model of a rockfall barrier foundation subjected to impact is developed and implicit dynamic mechanical simulations are carried out. Results indicate that the developed finite element model is capable of predicting the forces acting on the foundation and its components during an impact event. A detailed investigation of the interaction of the individual components reveals that damage to the foundation's concrete plinth is accurately captured by the model if an over-nonlocal gradient-enhanced constitutive model for concrete is employed. Moreover, the contact between the concrete plinth and the surrounding soil is identified as a key parameter influencing the mechanical behavior of the concrete plinth during impact. This work highlights key factors influencing the mechanical behavior of rockfall barrier foundations during impact and directly complements recent full-scale impact testing of such foundations performed at the Unit of Geotechnical Engineering, University of Innsbruck.

1 Introduction

Anthropogenic climate change and the accompanying increase in air temperatures result in widespread thawing of permafrost and melting of glaciers in mountain regions. Directly linked to this degradation of the earth's cryosphere is an increased risk of geohazards like landslides, rockfall events and flooding. This effect is expected to accelerate in the coming decades and poses a serious threat to communities in mountain regions [8].

Arising from this increasing risk of geohazards is a growing demand for the development and optmization of risk mitigation measures.

Flexible rockfall barriers present a common solution for mitigating the risk posed by rockfall. Although the erection of such barriers frequently presents the most economical solution, their construction in remote areas is a challenging and costly task. Especially the construction of their deep foundations, which are required for resisting large forces during a rockfall impact event, requires heavy machinery and constitutes a major part of the construction costs of the completed barrier [3].

To develop an improved understanding of rockfall barrier foundations subjected to impact loading and to optimize their design, multiple research projects were carried out at the Unit of Geotechnical Engineering, University of Innsbruck, encompassing both experimental and numerical investigations. The present contribution aims at outlining the conducted numerical simulations and presenting their results.

2 Background

To ensure the effectiveness of a rockfall barrier, all components must withstand the high dynamic impact forces that occur during a rockfall impact event. The safety requirements and testing criteria for rockfall barriers are specified in the European Assessment Document *EAD 340059-00-0106 – Falling Rock Protection Kits* [2], which serves as the basis for product-specific European Technical Assessment (ETA) documents needed to freely market rockfall barriers in the European Economic Area. The EAD also lists the main parts of rockfall barriers, namely

- an *interception structure*, i.e., wire nets responsible for catching rockfall, dissipating energy and transferring forces to other components;
- a *support structure*, i.e., metallic posts, which are either hinged or clamped to foundations, and base plates responsible for transferring forces to other components and maintaining the position of the interception structure;
- and *connection components*, i.e., metallic support ropes, fitted with energy dissipation devices, responsible for transmitting forces to the foundations in a controlled fashion and maintaining the position of the interception structure.

However, *foundations*, which are responsible for connecting the rockfall barrier to the ground, are beyond the scope of the EAD, entailing that their design is left to the responsible geotechnical engineer. The document merely demands that the time history of the forces acting on the foundation is measured during full-scale testing and provided in the product-specific ETA along with their maximum values.

The foundation of rockfall barriers usually comprises post foundations and the anchorage of the support ropes. The latter often consist only of a single micropile or a flexible cable anchor loaded predominantly by tensile forces. An individual micropile consists of a steel bar which is grouted along its entire length. Due to lateral forces induced during a rockfall impact event, the design of the post foundations is more elaborate and comprises multiple micropiles. In addition, a concrete plinth is often needed to allow the correct positioning of the base plate in uneven terrain [6].

3 Experimental investigations

The numerical investigations are based on an experimental campaign carried out at the University of Innsbruck in 2021 [7]. The investigations comprised full-scale testing of multiple types of rockfall barrier foundations subjected to impact. In 2021, six individual micropiles and 26 individual post foundations were tested.

The experimental setup is illustrated in Fig. 1. It consists of two steel frames: the primary frame, which carries a pendulum ($m = 1050\,\mathrm{kg}$ and $2860\,\mathrm{kg}$), and the secondary frame, which is used for lifting the pendulum to the desired drop height. The pendulum is connected to the individual foundations via a steel rope, which halts the pendulum's fall and transfers the impact force. Utilizing a pulley, multiple foundations can be tested without repositioning of the pendulum.

One of the experimentally investigated types of post foundations is illustrated in Fig. 2. It comprises two grouted micropiles, which are connected by a steel base plate. To transfer horizontal forces that arise during an impact event, the rear micropile is inclined with an angle of $45°$. The vertical front micropile is responsible for transferring compressive forces. Both micropiles are four meters long. To facilitate the installation of the base plate, a weakly reinforced concrete plinth is provided. In addition, the concrete plinth acts as a stiffener and protects the micropile heads from corrosion [13].

4 Methods

4.1 Finite Element Discretization

A finite element model is developed for the type of post foundation described in the previous section. Symmetry is exploited. The developed model is illustrated in Figs. 3 and 4 and comprises

- a section of the surrounding soil,
- the two micropiles,
- the steel base plate including a hinge plate,
- and the concrete plinth.

For modeling impact loading, a point mass m is included in the finite element model. It is connected to the base plate by means of a connector element with stiffness k.

Three-dimensional continuum elements with triquadratic shape functions and reduced integration are employed for discretizing the base plate, the surrounding soil and the upper parts of the micropiles, which interface with the concrete plinth. The lower parts of the micropiles, which interface with the surrounding soil, are discretized using beam elements with quadratic shape functions. For discretizing the concrete plinth, the employed type of finite element depends on the constitutive model (s. Section 4.3).

4.2 Interface Modeling

To account for the mechanical behavior of the various interfaces, we consider the following contact and interface properties (cf. Fig. 4):

(a) rigid coupling between the hinge plate and the connector element
(b) frictional contact between the base plate and the concrete plinth
(c) rigid coupling between the rear micropile and the base plate (identical nodes)
(d) constraint of the head of the front micropile to the the base plate (slotted hole)
(e) cohesive contact of the pile necks with the concrete plinth
(f) rigid coupling of the upper and lower parts of the micropiles
(g) embedding of the lower parts of the micropiles in the surrounding soil
(h) frictional contact between the concrete plinth and the surrounding soil

4.3 Constitutive Modeling

To obtain a linearly elastic embedding of the micropiles in the surrounding soil, its constitutive response is treated as linearly elastic. Moreover, linear elastic constitutive behavior is assumed for the steel base plate, since considerable plastic strains are not expected. Two variants are considered for modeling the constitutive behavior of the micropiles. Namely, a linearly elastic variant and a linearly elastic-perfectly plastic variant are investigated.

The focus of the present investigation lies on investigating failure modes of the concrete plinth. To this end, a constitutive model is employed, which adequately describes the nonlinear behavior of concrete. The model, originally proposed by Grassl and Jirásek [4], incorporates in an elastoplastic-damage constitutive framework both inelastic strains and material degradation effects. To accurately model the strength of confined concrete, its volumetric response is controlled by a *non-associated* plastic potential function.

However, the incorporation of material degradation and a non-associated plastic potential function entail unstable softening material behavior, which is responsible for mesh-sensitive results in finite element simulations. Given fine meshes, numerical difficulties due to spuriously brittle material behavior can also be observed. Therefore, regularization approaches must be incorporated in such models.

The most basic regularization approach is given by the *crack band approach* [1]. Regularization of the structural softening behavior is achieved by adjusting the softening behavior for each finite element based on a characteristic length of the element and the specific mode I fracture energy of concrete. Estimating the characteristic length has been the focus of intense research; however, a general expression does not exist [9]. Herein, the characteristic length l_{char} is calculated using the volume of a finite element V_{e} as $l_{\mathrm{char}} = \sqrt[3]{V_{\mathrm{e}}}$ – the resulting model is denoted as the CDP model. To prevent localization in sub-element domains [9], 8 node hexahedral finite elements with trilinear shape functions are used with the CDP model.

More advanced regularization approaches are given by nonlocal continuum formulations. These formulations can be categorized into two classes: *integral-nonlocal* approaches based on local averaging of internal variables [11] and *gradient-enhanced* approaches based on gradients of internal variables [10]. In contrast to integral-nonlocal formulations[1], the implementation of gradient-enhanced formulations in finite element frameworks is straightforward. Gradients of internal variables are incorporated by considering additional degrees of freedom, the so-called nonlocal variables. An over-nonlocal gradient-enhanced formulation of the aforementioned constitutive model for concrete is given in [12]. In the present contribution, this model is denoted as the GCDP model. For simulations with the GCDP model, 20 node hexahedral finite elements with triquadratic shape functions are used to discretize the concrete plinth.

4.4 Actions

The impact velocity of the pendulum (cf. Fig. 1) can be written in terms of the drop height H as $v_0 = \sqrt{2gH}$. Therein, $g = 9.81\,\mathrm{m/s^2}$ is the gravitational acceleration. Using the pendulum mass m, the impact energy is obtained as $E_0 = \frac{1}{2}mv_0^2$. In Table 1, the impact velocity and the impact energy are given for multiple values of the drop height and the pendulum mass.

5 Computational remarks

The implicit transient dynamic analyses are performed using *SIMULIA Abaqus/Standard (Abaqus)*. The CDP model is implemented in Abaqus using the *UMAT* interface. Due to the additional degrees of freedom for the nonlocal variable, the *UEL* interface is required for implementing the GCDP model in Abaqus.

6 Results and discussion

6.1 CDP model

The simulation results obtained with the CDP model for three values of the drop height, namely, $H = 2\,\mathrm{m},\ 3\,\mathrm{m}$ and $4\,\mathrm{m}$, are illustrated in Fig. 5 by means of displacement-time curves and force-time curves. The chosen values for the drop height correspond to the values employed in the experiments. However, due to significant losses – such as dissipation in the steel rope, the pulley, and the connections between the steel rope, the pendulum, and the foundation – the impact energy is overestimated in the model. As a result, the drop height in the model cannot be directly related to the drop height in the experiments. The pendulum mass is chosen as $m = 2860\,\mathrm{kg}$.

Regardless of the drop height, the impact duration is obtained as approximately $175\,\mathrm{ms}$. The experimentally determined impact duration of approximately $300\,\mathrm{ms}$ is significantly underestimated. Accordingly, the dynamic forces acting on the foundation are overestimated, which is especially true for larger drop heights. The obtained lateral displacements are in agreement with the values determined experimentally.

For $H = 2\,\mathrm{m}$, elastoplastic behavior of the micropiles has negligible effect on the results. For larger drop heights, yielding of parts of the micropiles is observed and the obtained lateral displacements increase accordingly. For $H = 4\,\mathrm{m}$, plastic deformations in the micropiles exceed the fracture strain of steel.

Damage to the concrete plinth is observed in all simulations. However, the amount of damage is greatly influenced by modeling as-

[1]An integral-nonlocal extension of the constitutive model for concrete proposed in [4] is given in [5].

H (m)	v_0 (m/s)	E_0 (kJ)	
		1050 kg	2860 kg
2.0	6.26	20.6	56.1
3.0	7.67	30.9	84.2
4.0	8.86	41.2	112

Table 1: Impact velocity v_0 and impact energy E_0 for multiple values of the drop height H and the pendulum mass.

sumptions. For instance, damage to the concrete plinth is only weakly pronounced if the interface between the concrete plinth and the surrounding soil ((h) in Fig. 4) is considered (Fig. 6, left). If, however, this interface is omitted, the rear part of the concrete plinth is severely damaged (Fig. 6, right). While the obtained lateral displacements are larger in this case, no influence on the dynamic forces acting on the foundation is found (Fig. 7). Nevertheless, a shift of axial forces from the rear micropile to the front micropile can be observed if the interface is omitted (Fig. 7).

Modeling the connection of the front micropile to the base plate, which allows for relative lateral displacements by means of a slotted hole in the base plate (cf. Fig. 2), also significantly impacts damage to the concrete plinth. If a rigid coupling of the base plate and the front micropile is considered instead of the constraint (d), the front part of the concrete plinth is severely damaged (Fig. 8).

Closer examination of the damaged parts of the concrete plinth in Fig. 6 (right) and Fig. 8 reveals undesired features of the predicted damage patterns. As localization of damage is not prevented by regularization via the crack band approach, the predicted damage patterns are closely aligned with the mesh lines of the finite element discretization.

6.2 GCDP Model

Simulation results obtained with the GCDP model for two values of the drop height, namely, $H = 2\,\mathrm{m}$ and $3\,\mathrm{m}$, are illustrated in Fig. 9. Qualitatively, the results are comparable to the results obtained with the CDP model. While the dynamic forces acting on the foundation are largely unaffected by the choice of constitutive model for concrete, larger lateral displacements are obtained by means of the GCDP model, indicating a softer structural response.

The difference between the CDP model and the GCDP model are most obvious when considering the predicted damage patterns. The damage pattern obtained by means of the GCDP model is illustrated in Fig. 10. In contrast to the CDP model, alignment of the damage patterns with the mesh lines of the finite element discretization is not observed with the GCDP model. Moreover, the maximum values of damage predicted by means of the GCDP model are significantly smaller than for the CDP model.

7 Conclusion

We carried out implicit transient dynamic analyses of a rockfall barrier post foundation subjected to impact. The finite element model, which was developed to this end, encompasses the interactions between the components of the foundation as well as the interaction of the foundation with the surrounding soil. The simulations indicated damage to the rear part of the concrete plinth, which was also observed during the experimental investigations. Furthermore, we identified the contact to the surrounding soil as an important factor influencing the mechanical behavior of the foundation.

The surrounding soil is treated as an elastic continuum, representing a drastic simplification of the actual constitutive behavior. Moreover, the interaction of the grouted micropiles with the surrounding soil is not adequately modeled. Therefore, statements regarding the energy dissipation capacity of the system are beyond the scope of the present study.

Additional research is needed for modeling the energy dissipation capacity of rockfall barrier foundations. In this context, the experimental investigations of individual micropiles subjected to impact, conducted as part of the 2021 experimental campaign, could play an important role in the future.

8 Acknowledgments

The first author acknowledges financial support provided by the *Tyrolean Science Fund (TWF; grant № F.45186/8-2022)*.

The second and fourth authors acknowledge the excellent cooperation with project partners, which made the experimental work possible. In particular, we would like to thank:

- *ÖBB Infrastruktur AG* (Dipl.-Ing. Markus Schuch),
- *Zentralstelle Ingenieurbauwerke und Georisiken (ZIG), Landesbaudirektion Bayern* (RD Dipl.-Geol. Andreas Koch),
- *SBB AG* (Florian Hofstetter, MSc ETH),
- *Rhätische Bahn AG* (Dipl.-Ing. Dr. Martina Sättele),
- *Wildbach- und Lawinenverbauung — Sektion Tirol* (Ing. Michael Posch, Mag. Michael Mölk, Ing. Mag. Dr. techn. Thomas Sausgruber),
- *Mair Wilfried GmbH* (Friedrich Mair).

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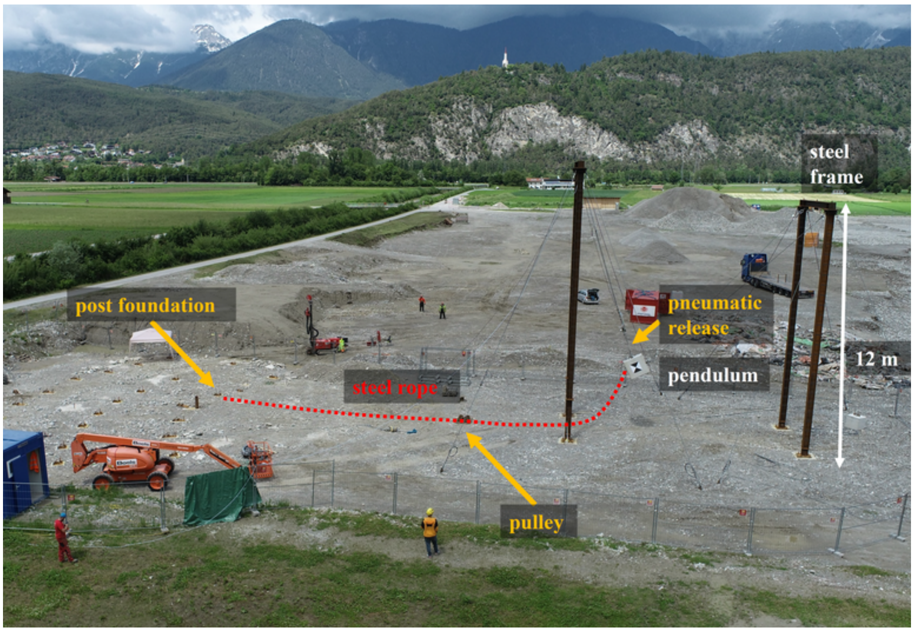

Figure 1: Experimental setup. Upon release, the pendulum swings downward from left to right.

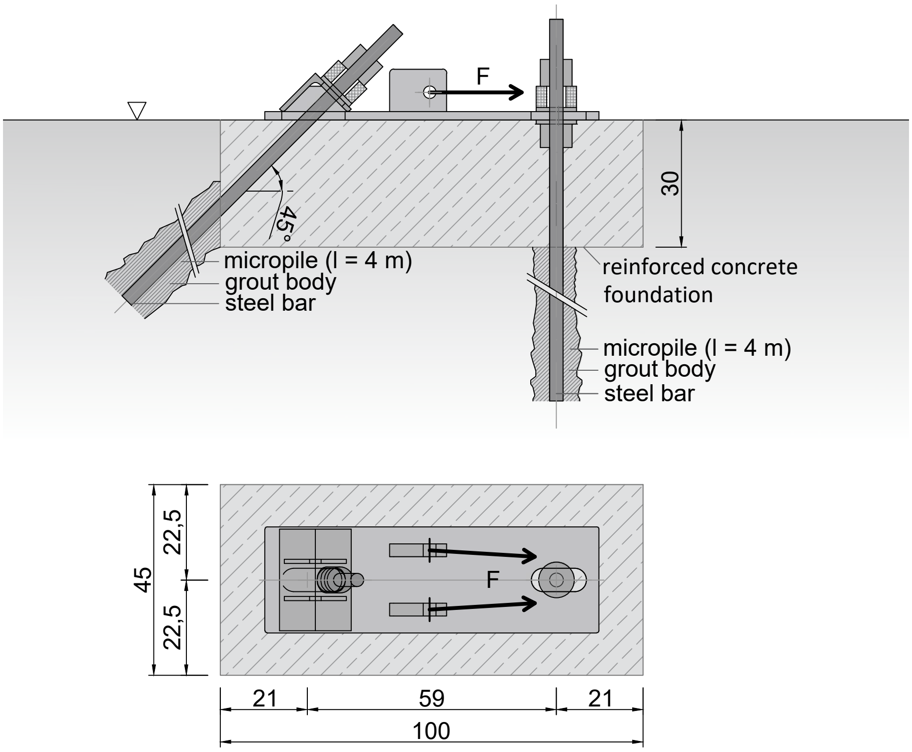

Figure 2: Schematic illustration of the investigated rockfall barrier post foundation.

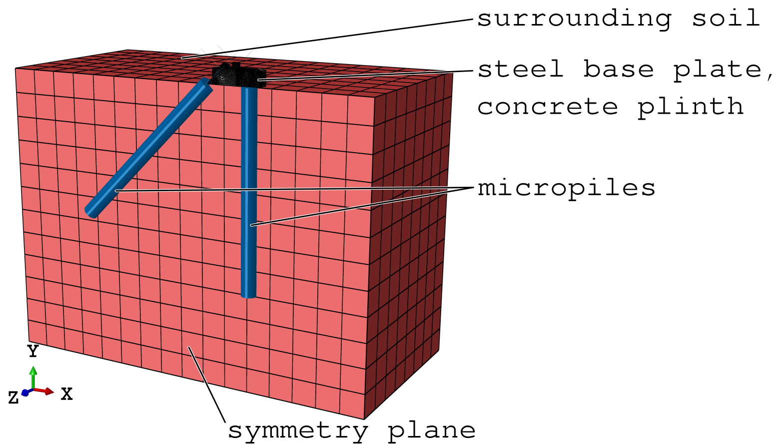

Figure 3: Illustration of the entire model. The diameter of the lower parts of the micropiles are exaggerated for illustrative purposes.

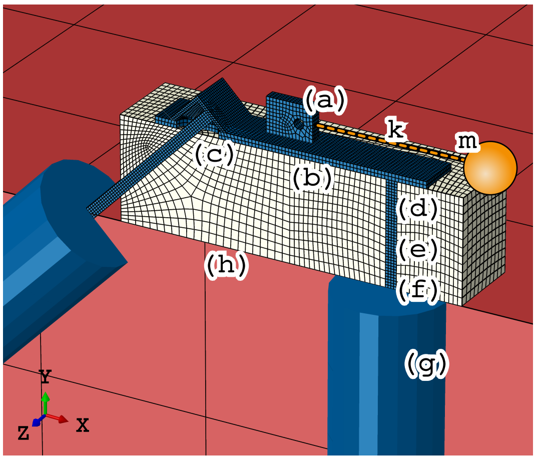

Figure 4: Detailed view of the model indicating the point mass m, the connector element k as well as the interfaces (a) to (h).

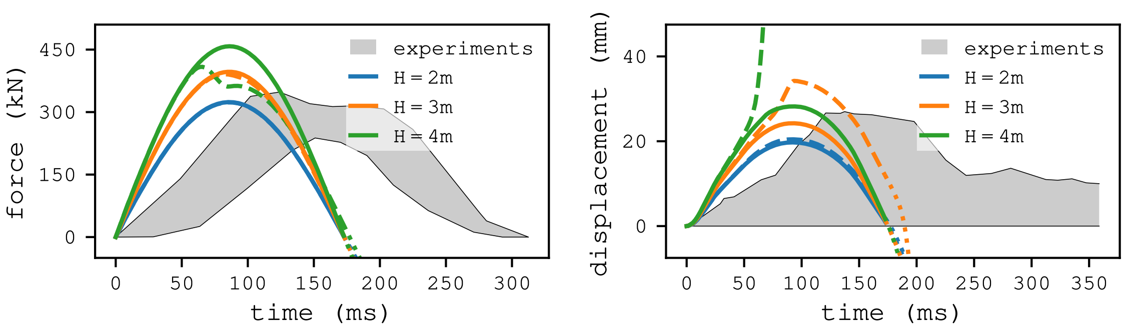

Figure 5: Displacement-time curves and corresponding force-time curves for simulations with the CDP model. Solid lines correspond to linearly elastic constitutive behavior of the micropiles. Dashed lines refer to linearly elastic-perfectly plastic constitutive behavior of the micropiles.

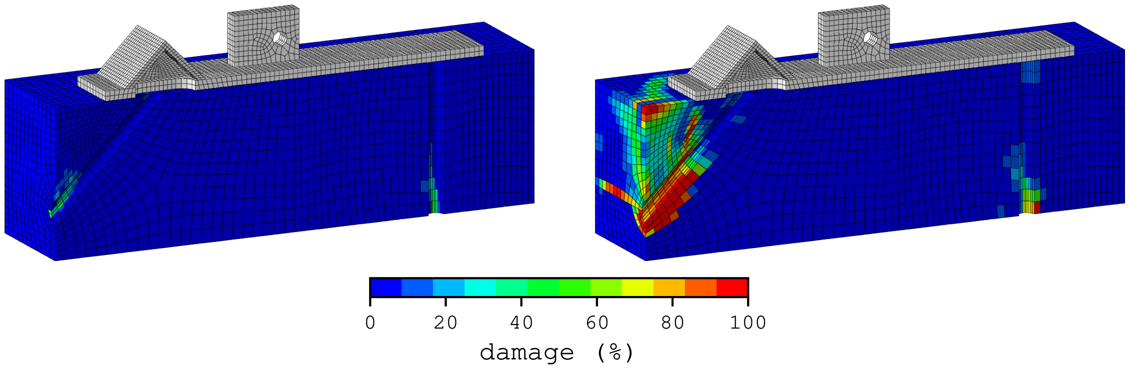

Figure 6: Contour plots of the damage of the concrete plinth predicted by means of the CDP model with (left) and without (right) consideration of the contact to the surrounding soil ($H = 2\,\mathrm{m}$; micropiles linearly elastic; $t = 150\,\mathrm{ms}$).

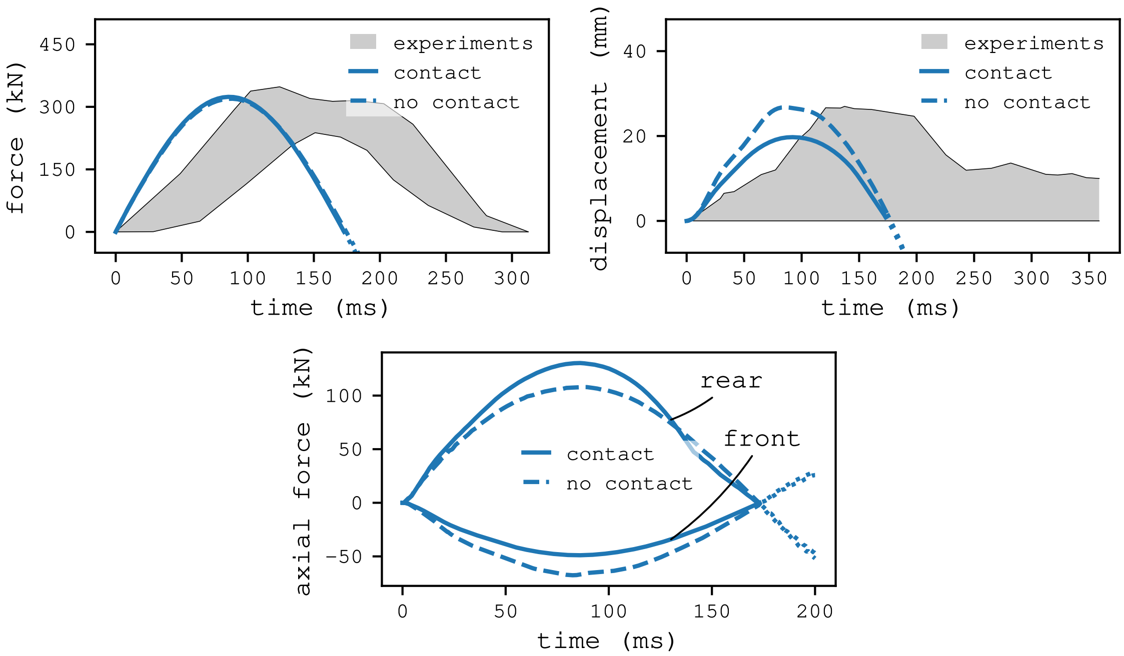

Figure 7: Displacement-time curves, force-time curves and axial force in piles vs. time for simulations with the CDP model with and without consideration of the contact to the surrounding soil.

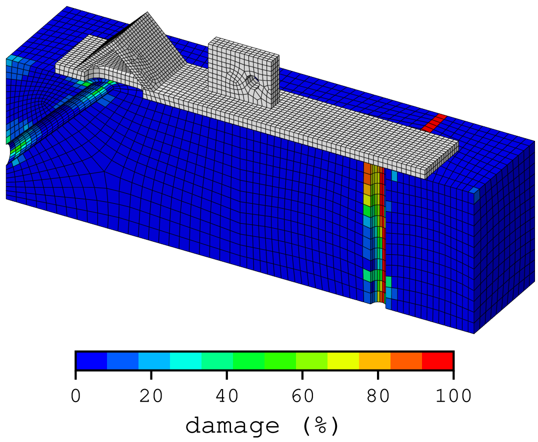

Figure 8: Contour plots of the damage of the concrete plinth predicted by means of the CDP model for a rigid coupling of the front micropile and the steel base plate ($H = 3\,\mathrm{m}$; micropiles linearly elastic; $t = 150\,\mathrm{ms}$).

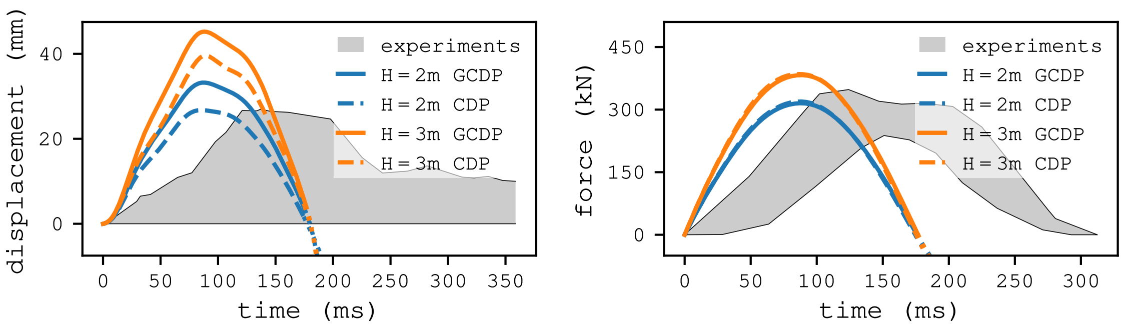

Figure 9: Displacement-time curves and corresponding force-time curves for simulations with the GCDP model (micropiles linearly elastic; no contact to surrounding soil; $t = 150\,\mathrm{ms}$).

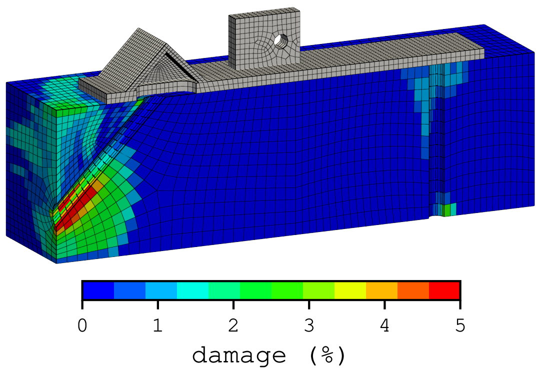

Figure 10: Contour plots of the damage of the concrete plinth predicted by means of the GCDP model ($H = 3\,\mathrm{m}$; micropiles linearly elastic; no contact to surrounding soil; $t = 150\,\mathrm{ms}$).