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Back-calculation of pillar foundation for Skjeggestad Bridge

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Abstract. In 2015, a landslide incapacitated the expressway E18 in southern Norway because one of the large foundation pillars of the southern lane of Skjeggestad Bridge near Mofjellbekken failed. This accident immobilised throughway traffic between Oslo and southern Norway for 17 months until the bridge became fully operational again. This paper focuses on the adjacent bridge pillar of the northern lane which experienced substantial displacements during the landslide. The paper presents the back-calculated reserve capacity of the pile foundation immediately after the landslide occurred. This was essential to establish the impact of the slide on the foundation capacity and evaluate the safety of the northern lane for traffic. The paper is a companion of the paper presenting the two- and three-dimensional analyses of the landslide that occurred in Mofjellbekken. The software Trimble NovaPoint Geosuite was used for an integrated design approach examining the soil layering, selecting the soil parameters and doing capacity calculations on the pile foundation (and calculating the stability analyses in the companion paper). The soil parameters were selected based on a statistical analysis of the key parameters and for cases with insufficient information correlations on similar soils are used. Two modelling approached were investigated to simulate the slide impact on the piles. The sensitivity of the lateral performance of the piles to soil stiffness was investigated based on recommendations from different guidelines. The implication of these analyses on pile cross-sectional utilization is discussed.

1. Introduction

In February 2015, an unexpected quick clay slide occurred near Mofjellbekken, causing failure of pillars of the Skjeggestad bridge. This large bridge is located along the main transportation corridor between Oslo and southern Norway. Traffic chaos ensued for a period of 17 months. The pillar in Axis 4 of the southern lane failed due to the landslide causing complete damage of the bridge superstructure. However, damage to the adjacent pillar of the northern lane was limited to horizontal displacements of 3-6 cm towards the south and 2-4 cm towards the west. It is important to study the consequence of this displacement on the remaining pile foundation capacity after the landslide, so that safety of this pillar foundation for further traffic is ensured. This paper focuses on the back-calculation of the capacity of the bridge pillar of the northern lane that was displaced 5 cm due to the landslide. A companion paper, presented at this conference, examines in detail the stability of the slope with two- and three-dimensional stability analyses[1].

Engineering design on soft Norwegian clay usually presents a challenge. For the stability and pile foundation analysis, including the selection of the soil parameters, Trimble NovaPoint GeoSuite [2] was used to do an integrated study of all design aspects of slope stability [1] and pile design. The two papers illustrate the common ground used for the interpretation of the soil parameters which were determinant for each analysis., The two papers also illustrate the advantages such unified approach provides.

This paper briefly describes the failure that occurred, presents the interpretation of the soil parameters in Trimble NovaPoint GeoSuite at the location of the bridge pillar and the results of the back-calculated pile capacity analyses after the landslide occurred. The pile analyses used two approaches from two design guidelines, the API-93 guideline [3] and the Norwegian pile design guideline, Peleveiledningen 2019 [4]. The analyses focused on sensitivity of the pile analysis to the input parameters for each recommended method in the guidelines. The soil investigations combined with a statistical analysis of key parameters were used to select the strength parameters and soil layering. The pile analysis was conducted as a function of variations in the soil stiffness and lateral pile behavior modelling approach.

2. The 2015 pillar failure of the Skjeggestad Bridge

The landslide occurred on February 2, 2015 and damaged the Mofjellbekken Bridge (also called Skjeggestad Bridge). Fortunately, the traffic was stopped immediately, and no lives were lost. Figure 1 shows the extent of the 10 000 m^3 landslide [5] and its effect on the bridge. The southern bridge got heavily damaged and had to be demolished and rebuilt. This paper focuses on the northern bridge and especially the bridge pillar at Axis 4 (red circle in Fig. 1), which was exposed to horizontal displacement, of 3-6 cm towards the south and 2-4 cm towards the west, as measured after the landslide. Based on an evaluation of the reserve capacity of the piles, the pillar was reinforced with jet grouted columns in 2015-2016 as permanent rehabilitation [7].

Figure 1. Quick clay landslide that caused failure of the Skjeggestad Bridge pillar. The investigated pillar, Axis 4 bridge, lies slightly outside the failure footprint and is circled in the figure (Photo: SVV 2015)

The Norwegian Water Resources and Energy Directorate (NVE) established an independent commission to investigate the cause of the landslide. Possible triggering actions investigated were erosion in the stream, rainfall events, traffic vibrations and the placement of new fill material on the slope. The inquiry concluded that the fill material placed at the slope crest caused the landslide [5].

The topography and variation of soil strength in the area was complex, demanding a vast number of soundings. The depth to bedrock also varies significantly at the site. Figure 2 summarizes the zonation selected for this study based on the available piezocone tests. These tests were carried out after the slide. Axis 4 is indicated with the green area and the soil layering is given in the green box. This soil profile is used in the pile analysis. The extent of the landslides and the cross-sections U-U and Q-Q used for the early stability analyses ([5]; [6]) are also shown in Figure 2. Further discussion of other areas in Figure 2 can be found in [1].

The soil layering in the area designated as "Pillar axis 4" consists of dry crust underlain by a soft to medium soft silty clay, sensitive and, locally, quick. Bedrock or a stiff till underlies the clay. Detailed information about the site and the spatial variation of the soil parameters can be found in [1]. Detailed information about the slide and earlier back-calculations can be found in [5], [6] and [7]. The most recent stability calculations are however found in [1].

3. Soil parameters for pile analysis

The interpretation of the soil parameters was done in the Trimble NovaPoint GeoSuite module "Soil Data Interpretation" (SDI). Earlier site investigations conducted during construction of the bridge and those carried out after the landslide were used. When enough data were available, a statistical analysis of the soil parameters was done. However, except for cone resistance data from piezocone tests (CPTU) after the landslide (in the five colored areas in Fig. 2), there were very little data on several of the required properties for pile design analysis. Thus, when data were not available, soil parameters were deduced from correlations available in the SDI module. Table 1 presents the parameters established for characterization of the soil at the location close to axis 4. Key soil parameters for the Geosuite PileGroup analyses with the API soil model were the strength parameters, stiffness and unit weight.

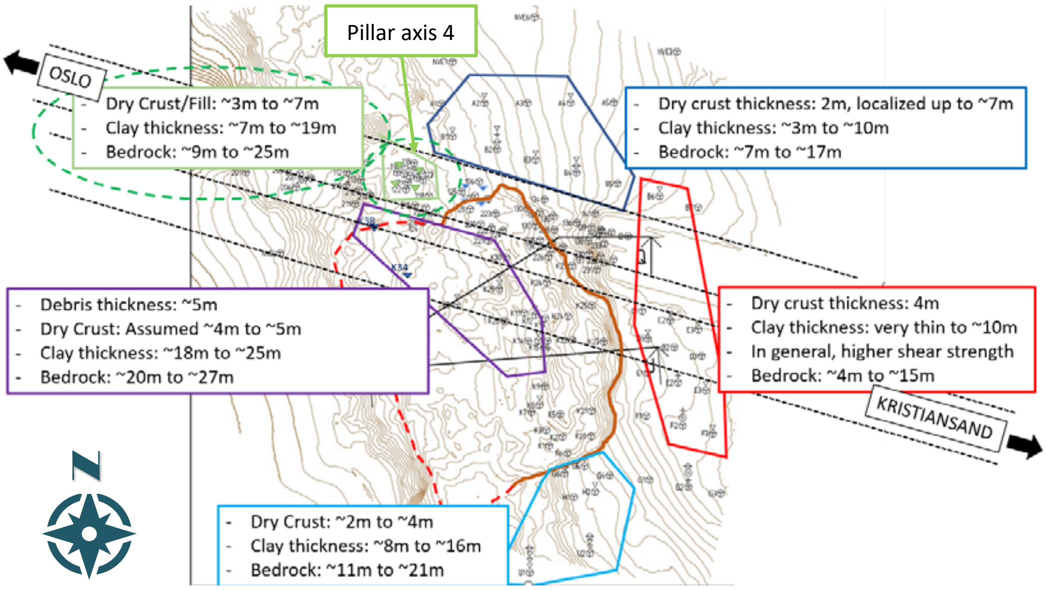

Figure 2. Overview on the different zones identified for soil layering at the landslide. East- and westbound bridge lanes illustrated with dotted lines.

3.1. *Index properties*

All the index properties were obtained from the site investigation (both *in situ* and laboratory tests) prior to the construction of the bridge [8]. Except for the sensitivity, the measured index properties were essentially constant with depth. Noticeably, the plasticity index was extremely low at 5 to 8%. This suggests very high stress-induced anisotropy in the quick clay.

3.2. *Undrained shear strength*

Figure 3 presents the results of the undrained shear strength interpreted from the piezocone tests (CPTU), using a cone factor N_{kt} between 10 and 14. The undrained shear strength interpreted corresponds to the triaxial compression strength ($s_{u\ TC}$). The figure on the right also shows the results of one *in situ* vane shear (VS) test run before construction of the bridge, but only in the top 10 m of the deposit (small inset in right of Fig. 3). A statistical analysis of the CPTU results was carried out in the SDI (right figure) showing mean ± one standard deviation. The coefficient of variation (COV, equal to the ratio of one standard deviation to the mean) was 15% below 7 m but could be as high as 30 % above 7 m. More discussion on the variation of the undrained shear strength across the site can be found in the companion paper [1]. For the present paper, the mean shear strength was adopted as a representative profile.

3.3. Stress history and shear modulus

There were very few oedometer tests to estimate the preconsolidation stress and overconsolidation ratio (OCR). As a result, the OCR was established from the undrained shear strength in triaxial compression normalized with the effective overburden stress. This is evaluated in light of values for both Norwegian clays and clays in the literature [9]. The inferred OCR is therefore only approximate. Nonetheless, it was possible to obtain a reasonably looking approximate OCR profile with depth, as shown in Figure 4.

There were no site data to determine the shear or Young's modulus. The values in Table 1 were obtained from the correlations in the SDI module, based on the results for Norwegian clays and other clays in the literature (e.g. [9]; [10]; [11]).

Table 1. Summary of interpreted soil parameters for pile design analysis at Pillar axis 4

Soil property	Crust (5 m)	Soft clay (5-19 m)	Source of data
Index properties			
Water content, w	25%	30%	Initial investigations, 3 boreholes
Plasticity index, I_p	10%	5-8%	Initial investigations, 3 boreholes
Sensitivity, S_t	4-10	75-150 (30 from 17m)	Initial site investigations, VS & fall cone
Clay content	--	30-35%	Initial investigations, 2 boreholes
Total unit weigh, γ_t	20 kN/m^3	19.5 kN/m^3	Initial investigations, 3 boreholes
Undrained shear strength, s_u			
TC (from CPTU), $s_{u\ TC(CPT)}$	--	Fig. 3	5 CPTU tests, N_{kt} = 10-14
Average (from VS), $s_{u\ VS}$	30-60 kPa	Fig. 3	One VS, not at location; stops at 10m
Anisotropy ratios, s_{uTC}/s_{uDSS}, s_{uTE}/s_{uDSS}	--	Very high anisotropy	Very low I_p: used correlations in SDI (see also [2]; [9]; [10])
Stress history			
Overconsolidation ratio, *OCR*	>3	3 to 1	From s_u/p'_0 ratios and correlations in SDI
Deformation modulus (normalized to undrained shear strength)			
$G_{50}/s_{u\ TC}$	--	300	From correlations in SDI
$G_{50}/s_{u\ DSS}$	--	150	Fro m correlations in SDI
Max shear modulus, G_{max}/s_u		900-1000	From correlations in SDI
Young's modulus, E $(2G$ $(1+\nu)$	kPa	---	From correlations in SDI

Notation TC = Triaxial compression TE = Triaxial extension DSS = Direct simple shear
CPTU = Piezocone test VS = Vane shear
G_{50} = Secant shear modulus at 50% peak shear stress
p'_0 = In situ vertical effective overburden stress ν = Poisson's ratio

4. Design approaches for lateral behaviour of piles

4.1. Background

The soil/structure interaction was modelled based on a solution found by an iterative procedure. The lateral interaction between a pile and the surrounding soil adopted in this paper was based on the Winkler model, and representing the soil resistance with independent springs.

Load-deformation curves (*p-y*), describing the relationship between lateral deflection and mobilized resistance in each spring, were used for the analysis of the lateral behaviour of the piles. The API-93 [3] and Peleveiledningen 2019 [4] were used to evaluate the lateral performance of the pile group.

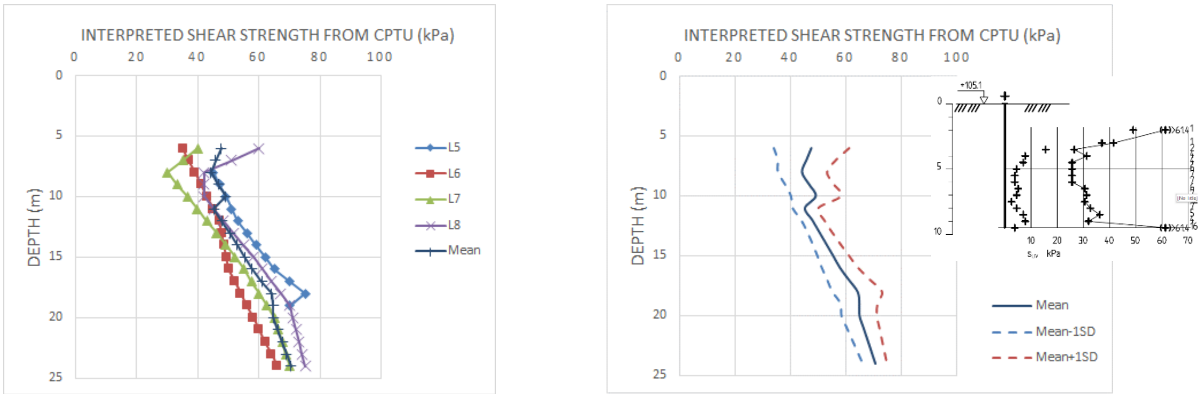

Figure 3. Active undrained shear strength at Bridge Pillar 4 from CPTU (left: 5 CPTU tests, right: mean ± one standard deviation (SD); inset: result of one vane shear (VS) test close to Pillar 4)

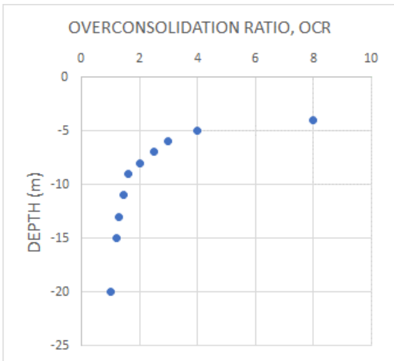

Figure 4. Approximate overconsolidation ratio inferred based on undrained shear strengths normalised with in situ overburden stress at location close to Pillar 4

4.2. API-93 (Matlock) guideline

The API-93 method is based on Matlock [12] and is valid for small diameter (*D*) piles ($D < 2.5$ m). The approach is considered common practice and used for both onshore and offshore pile design. Figure 5 shows the normalized API-93 curve.

The static lateral resistance P_{ult} is given by $N_{ru} \cdot s_u$, where the lateral bearing factor, N_{ru}, depends on whether the failure of the surrounding soil is shallow or deep. API-93 suggests N_{ru}-values between 3 and 9. Shallow failure (a wedge mechanism) occurs when the effective overburden stress, p'_0, is small. The transition between shallow and deep failure is estimated through calculation of a transition depth, X_r, which is a function of s_u, γ', diameter *D* and the constant *J*, with *J* vaying between 0.25 and 0.50.

An important parameter for the displacement ratio, δ/δ_{50}, is the strain which occurs at 50% of the maximum principal stress in an undrained triaxial compression tests, ε_{50}. The reason for using triaxial tests is that triaxial tests were used for the back-calculation of the pile load tests that form the basis of the model. For further details on the *P-y* model, reference is made to [3] and [12].

4.3. Peleveiledningen 2019 approach

Peleveiledningen 2019 is a Norwegian guideline considered as best practice for pile design. The *P-y* curve formulation for clay is based on the same principals as given in API-93, except for two additional stress ratio points at levels (P/P_{ult} = 0.23 and 0.33) (Fig. 6). This curve is the same as suggested in the newer revisions of API and ISO-19901-4:2016 [13]. In terms of static lateral resistance, the P_{ult} suggested by Peleveiledningen 2019 is slightly higher than that from the API-93 method. The lateral bearing factor, N_{ru}, varies between 5 and 10 depending on the failure mechanism, where deep failure starts at a depth of 8*D* (i.e. 8 pile diameters).

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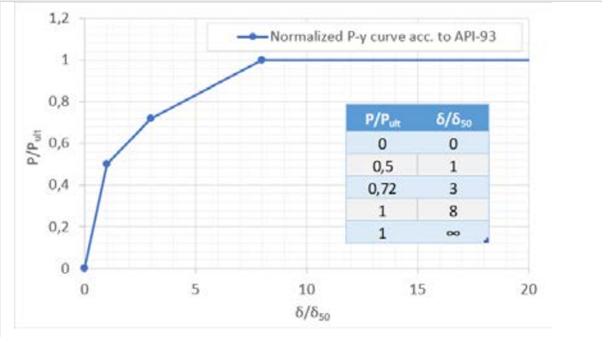


Figure 5. Normalized static *P-y* curve according to API-93 approach

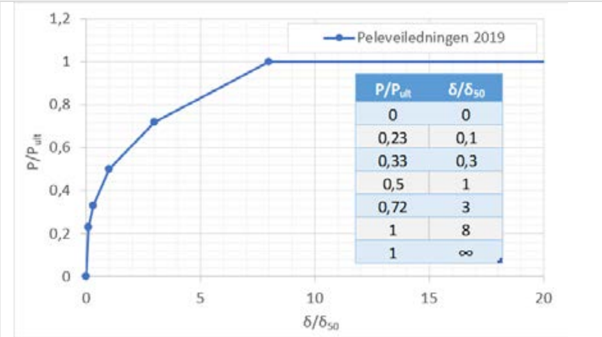


Figure 6. Normalized *P-y* curve according to Peleveiledningen 2019

5. Back-calculation for bridge pillar in axis 4

The Trimble NovaPoint GeoSuite Toolbox [2] assembles a series of computer programs especially developed for geotechnical design, including stability, settlement, bearing capacity, pile and excavation calculations. The module for pile calculations, GS PileGroup, was used for this study for simulating the interaction between piles in the pile group with the surrounding soil by means of different *P-y* models. The API-93 approach is a standard model in GS PileGroup, while the Peleveiledningen needs to be manually inserted at this time. The pile group in axis 4 was modelled 'as-built' with both vertical and inclined piles (Fig. 7).

Figure 7. Illustration of pile group in GS PileGroup (Left: plan view, Right: depth profile view)

After the landslide, the top of the pillar foundation was observed to have been displaced horizontally 3-6 cm towards the south and 2-4 cm towards the west. For the back calculations, a lateral 5 cm displacement at the pile top was used. The pillar foundation was outside the landslide front, an important assumption deducted from this fact was that the landslide caused soil movement around the pillar foundation in axis 4 only in the top 5 m thick dry crust, down to the transition between the dry crust and the quick clay. If the soil movement occurred in the quick clay layer, the pillar foundation in Axis 4 would have been exposed to significant movements. To identify the effect of the soil movement on the pile cross-sectional capacity (Axis 4 pillar foundation), two methods were used (Fig. 8) with the following procedure (for both methods, the analysis included dead weight of the bridge):

1) Apply horizontal force at pile top resulting in 5 cm displacement at the pile top. This is referred to as "Applied force" method.

2) Apply soil displacement in dry crust, resulting in 5 cm displacement at pile top/dry crust. This is referred to as the "Applied soil displacement" method.

Tables 2 and 3 summarize the characteristic input values used for the steel core pile and soil profiles. The unit weight and undrained soil strength were taken from the soil parameter study (Table 1; Fig. 3).

The mean value was used as a reference strength for the clay, and a cautious estimate of 75 kPa was assumed in the dry crust. The pile was modelled with fixed boundary conditions at top and bottom. However, to avoid numerical problems with long slender piles, a tip restriction was imposed, with a code referred to as ND in GS PileGroup was used at the bottom of the pile. The ND code at a specific depth gives moment fixities at the specified depth taking into account the representative axial stiffness of the rest of the pile below that depth. For the analyses in this study, ND was defined at 15 m below terrain level. Below this level, the moment and displacement were considered negligible.

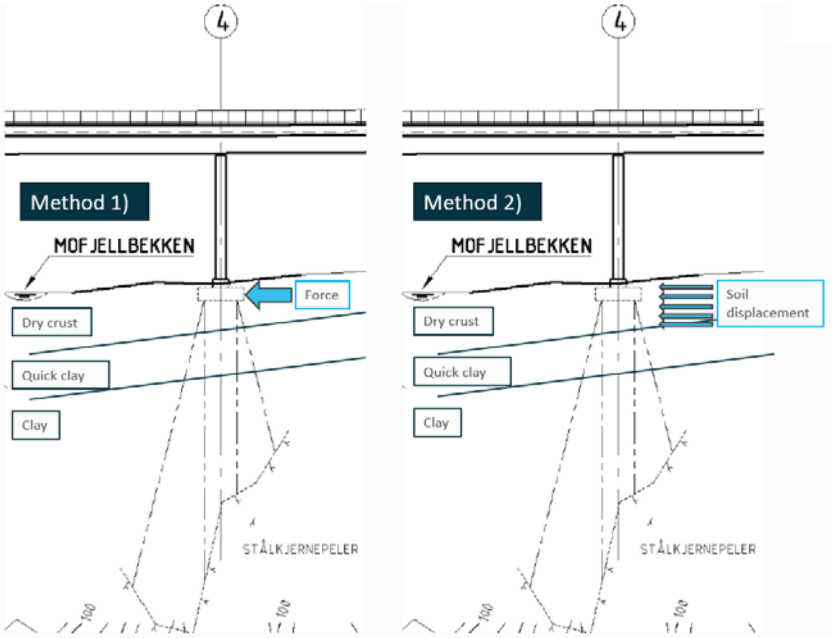


Figure 8. Illustration of bridge pillar, foundation system and analyses methods: left: Applied force method; right: Applied soil displacement method.

As a sensitivity study, the stiffness was modified by varying the axial strain ε_{50} parameter between 0.6% and 2%, as illustrated in Fig. 9. These curves were taken at a depth of 2.5 m below pile top. No sensitivity analysis was performed with respect to the undrained shear strength.

Table 2. Steel core data used in GS PileGroup

Diameter, D [m]	Weight [kN/m^3]	Yield strength [MPa]	EA [kN]	EI [kNm^2]
0.219 (steel core 0.150)	54.4	295	4 417 252	9267.5

Table 3. Soil data used in GS PileGroup

Soil layer [-]	Depth [m]	Soil weight, γ [kN/m^3]	Undrained shear strength, s_u [kPa]	Strain, ε_{50} [-]	API-J [-]	T_{res}/t_{max} [-]
Dry crust	0	19.5	75	0.006/0.02	0.5	0.7
	5	19.5	45	0.006/0.02	0.5	0.7
Quick clay	5	19.5	45	0.006/0.02	0.5	0.1
	11	19.5	45	0.006/0.02	0.5	0.1
Quick clay	11	19.5	45	0.006/0.02	0.5	0.1
	45	19.5	117	0.006/0.02	0.5	0.1

6. Results of analyses

The results of the GS PileGroup calculations are presented in three figures: (1) pile group utilisation with the API-93 and Peleveiledningen (2019) guidelines, with ε_{50}=0.02 (Fig. 10); (2) displacement of

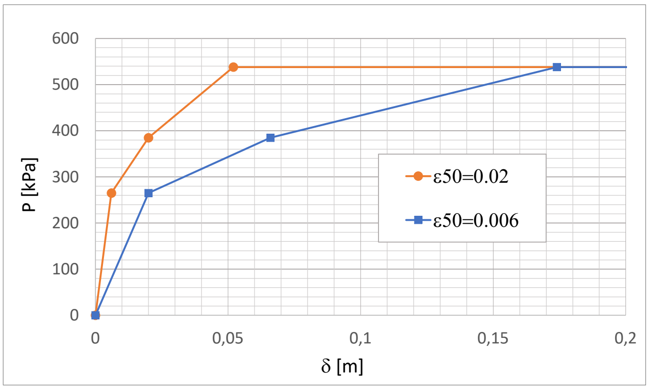


Figure 9. *P-y* curve for the API-93 approach, with ε_{50} set to 0.6 % and 2 %

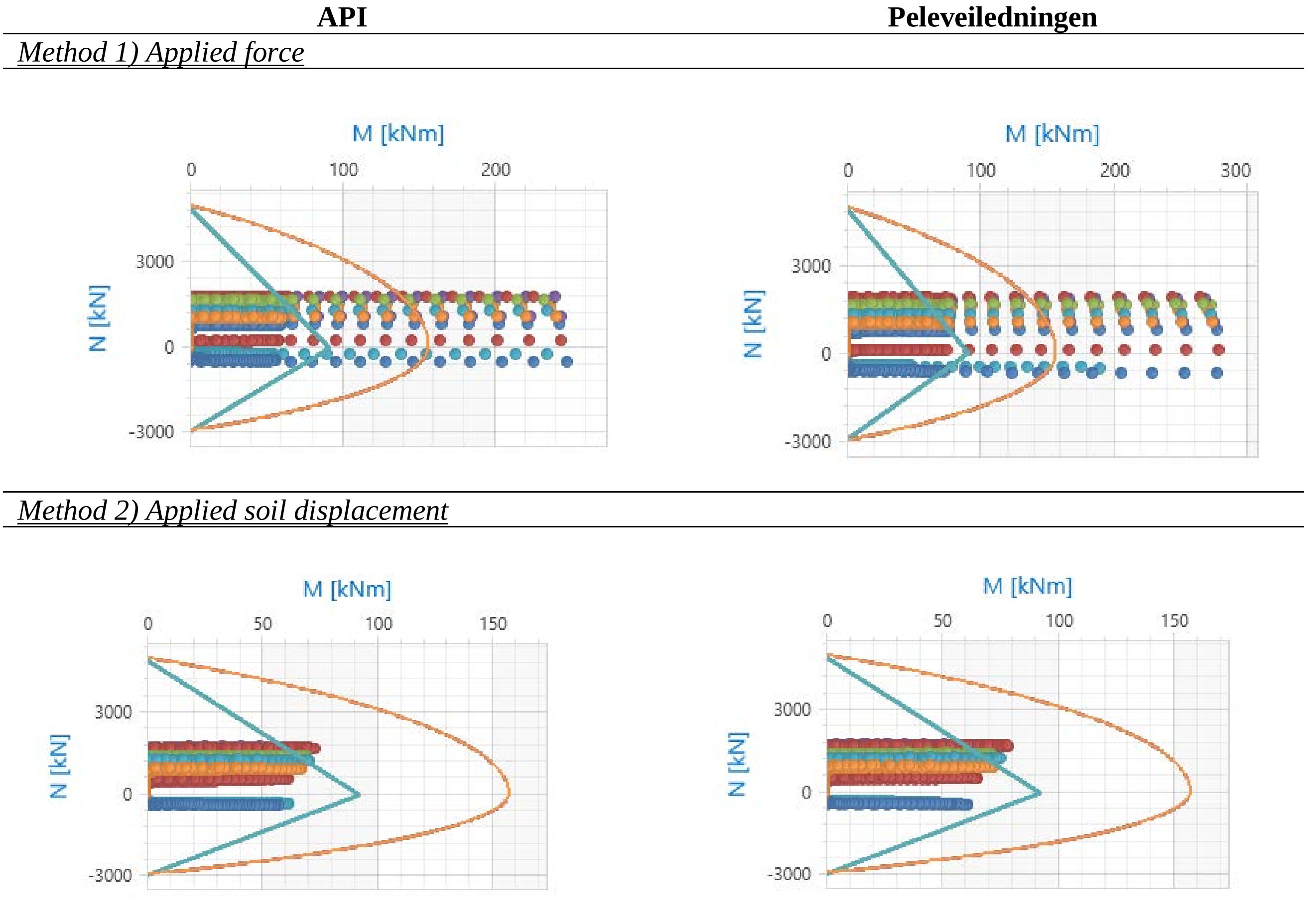
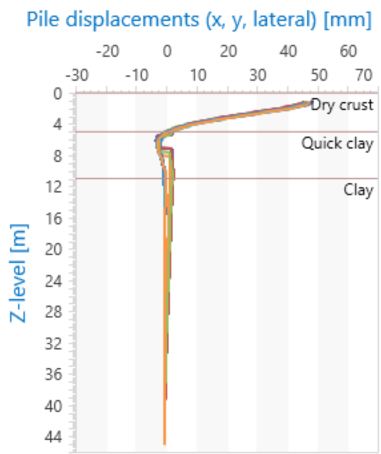


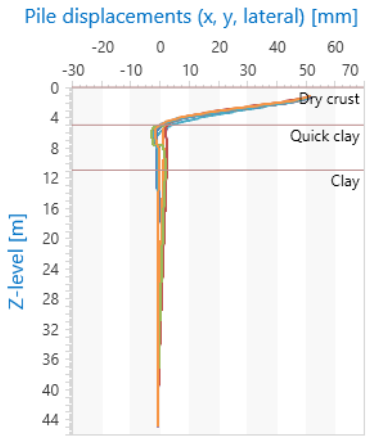
Figure 10. Utilization of pile group with API-93 and Peleveiledningen 2019, ε_{50}=0.02: Outer yield curve represents the plastic capacity, linear curve represents the elastic capacity of pile cross-section, are the utilization of each pile node from pile top to toe. Each pile in GS PileGroup have different colours.

pile group with API-93 and Peleveiledningen (2019) guidelines, with ε_{50}=0.02 (Fig. 11); and (3) pile group utilisation with the API-93 guideline, using ε_{50}=0.02 and ε_{50}=0.006 (Fig. 12). The two methods, the 'Applied force' and the 'Applied soil displacement method', are compared. The results suggest that horizontal forces of 3500 kN and 3880 kN are required to reach 5 cm displacement with the *p-y* curves from the API-93 and the Peleveiledningen 2019 approaches respectively. The required force to reach 5 cm displacement when using API-93 with ε_{50}=0.006, was 4303 kN.

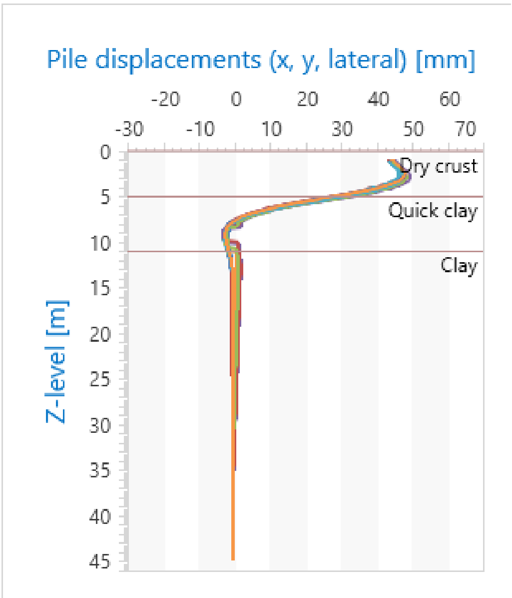
API	Peleveiledningen

Method 1) Applied force





Method 2) Applied soil displacement



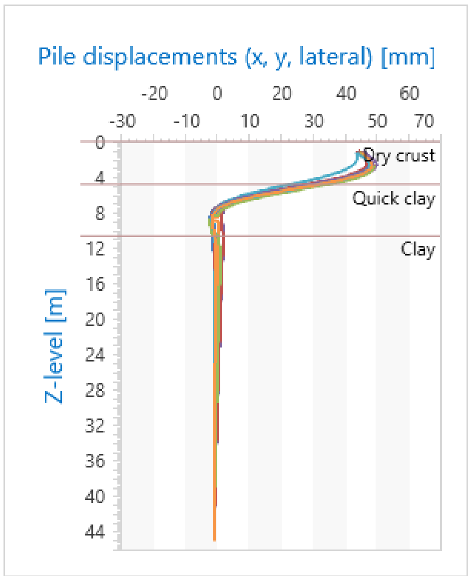


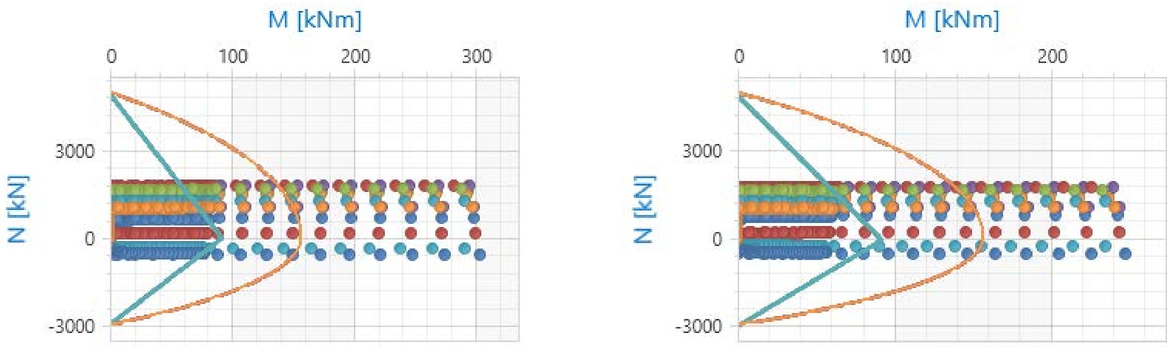
Figure 11. Displacement of pile group with API-93 and Peleveiledningen 2019, ε_{50}=0.02. The displacement of each pile is shown with different colours, as presented in GS PileGroup.

7. Summary and conclusions

The back-analyses of the pile group under the pillar Axis 4 of the northern Skjeggestad Bridge showed a significant difference in pile cross-section utilization depending on the method investigated. A higher utilization was achieved for the case where the 5 cm displacement of the bridge pillar was modelled by applying a shear force at pile top ("Applied force" method), compared to the case where the 5 cm foundation displacement was achieved by applying soil displacement in the dry crust soil layer ("Applied soil displacement" method). Applying a shear force on the pile top creates higher concentrated bending stresses in the uppermost part of the pile, due to the dry crust supporting the piles with rather high stiffness with the Winkler springs. This results in higher pile cross-section utilization and thus forms a more conservative estimate of the pile group utilization directly after the landslide.

API ε_{50}=0.006	**API ε_{50}=0.02**

Method 1) Applied force



Method 2) Applied soil displacement

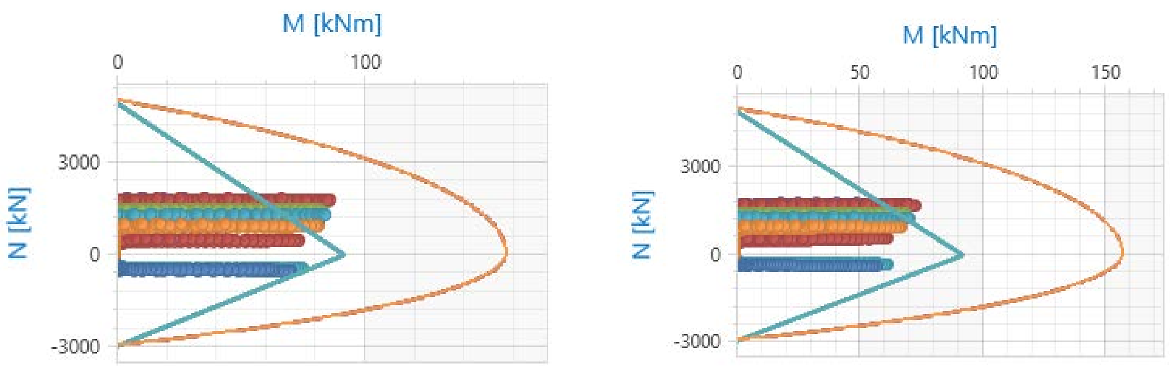


Figure 12. Utilization of pile group for API-93 using ε_{50}=0.02. and ε_{50}=0.006. The dots represent utilization of each pile node from pile top to toe.

The "Applied soil displacement" approach is, however, considered a more realistic approach bearing in mind the movement of the surrounding soil due to the landslide. It is reasonable to assume that the movement of the soil in the dry crust layer creates a large lateral pressure on the pile group. There are uncertainties related to depth at which the soil displacement actually occurred at the site. It is assumed that the soil displacements were very limited in the underlying quick clay layer. Otherwise, the pillar foundation in Axis 4 would have been exposed to significantly larger displacements.

For pile design in general, softer Winkler models typically provide a more conservative approach to pile group bearing capacity, whereas a stiffer model typically yields more conservative results for pile cross-section utilization.

The analyses also show that where soil displacement is considered as an action on the pile group, a stiffer Winkler spring gives higher pile cross-section utilization. Displacing a stiffer soil causes larger stresses in the pile compared to a softer soil. Since Peleveiledningen 2019 suggests stiffer springs at lower stress levels and slightly higher lateral bearing factor N_{ru} as compared to API, Peleveiledningen 2019 is believed to represent a more conservative model for the evaluation of reserve capacity after the landslide event.

The back-calculations reflect the foundation reserve capacity immediately after the landslide had caused a 5 cm displacement of the bridge pillar. The analyses highlight the importance of sensitivity analyses as part of design. In the Skjeggestad Pillar Axis 4 example, the sensitivity analyses were done for variations in soil stiffness and using two methods of lateral pile behavior (recommended by different design codes). For design, one should consider doing sensitivity studies of the parameters with significant uncertainty, including undrained shear strength. It is worthwhile to mention that results of sensitivity evaluations were used to select the rehabilitation measures. For the Skjeggestad Bridge, stabilizing the soil around the piles with jet grouted columns was the preferred mitigation method [7].

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