Natural experiments in rock mechanics using high precision monitoring of chalk sea cliffs

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ABSTRACT: Prediction of rock slide events remains a difficult task for geoscientists. Kinematic analysis provides an indication of possible modes of failure at a site. However, the highly variable nature of chalk compressive strength due to variations in water content and salt weathering is such that parameterizing models of slope stability can result in large variations in the resultant factor of safety. In this work, we use high-precision monitoring data of an eroding coastal cliff to characterize the geometry a large wedge failure in chalk. We use these data in conjunction with published material properties to model the joint compressive strength of the chalk at the time of failure through back analysis. Results indicate a strength of 7.19 KPa for the chalk suggesting that the joint surface was close to saturation at the time of failure.

1 INTRODUCTION

The structural geology of cliffs provides primary control over the type of slope failure that may occur. This is because the uniaxial compressive and shear strength of penetrative discontinuities are generally lower than those of the intact rock (Terzaghi, 1962). Slope stability modelling of brittle rock is typically undertaken using a limit equilibrium approach whereby stability is a function of body forces and the shear strength of discontinuities (joints) that bound the potential failure mass (Wyllie & Mah, 2004). Several models have been proposed to quantify the shear strength of joints, the most commonly used being the Mohr-Coulomb and Barton-Bandis models (Hencher & Richards, 2015). Research into the strength properties of medium-hard chalk indicate compressive strength can vary from 5 to 15 MPa based on the water content of the sample (Mortimore et al. 2004a), with a further reduction of up to 54.1% due to sea salt saturation and the growth of salt crystals (Lawrence et al.,

2013). Operationally, it is difficult to apply these relationships as sampling changing water content and salt concentrations across joint surfaces prior to failure is cost prohibitive. Consequentially, the ability to mitigate against potential slope failures is often based on incomplete information.

Tucker (2009) defines a natural experiment in landform development as a case study in which the initial/boundary conditions are well constrained and only one parameter varies significantly within the system. A common approach to this within landslide research is back analysis, in which the ratio of driving and resisting forces is assumed to be 1 at the point of failure (Wyllie & Mah, 2004). We apply this concept to eroding sea cliffs in order to examine the strength of chalk at a scale that cannot be replicated through laboratory testing. The Barton-Bandis model of shear failure calculates the shear strength of joints (τ) based on the residual friction angle (ϕ_r), the joint-wall compressive strength (JCS), the joint roughness coefficient (JRC), and the normal force (σ_n) such that (Barton 2013):

$$\tau = \sigma_n \tan \left[JRC \log_{10} \left(\frac{JCS}{\sigma_n} \right) + \varphi_r \right] \quad [1]$$

Equation 1 assumes a planar shear surface with second-order asperities based on JRC. However, many joint surfaces exhibit first-order asperities or waviness. This waviness is defined as the maximum difference between the slope of asperities and the mean slope. These act to increase τ in equation 1 according to (Miller, 1988):

$$\tau = [1] + \sigma_n \tan[\text{waviness}] \quad [2]$$

In this research, we use well-established material properties for chalk in conjunction with failure geometry data derived from high-precision monitoring to establish the JCS of Newhaven Chalk through back analysis.

2 STUDY AREA

Telscombe Cliffs are one of the few remaining undefended sections of coastline between Brighton and Newhaven in East Sussex, UK (Figure 1). Rising to a maximum elevation of 49 m, the cliffs are 750 m

Figure 1: Telscombe study area with cross sectional profile of the cliffs provided below. Red area indicates the location of the wedge failure used in this study.

long and are formed from Cretaceous Chalk of the Newhaven and Culver Formations (Mortimore, 1997). The cliffs are orientated to the south-west facing the dominant wave direction. To the east, the site is bounded by the sewage outfall pipe, which is protected by a concrete groyne and provides an artificial barrier

to the movement of beach sediments. As a consequence of this, the eastern portion of the cliff line is protected at the toe by a substantial pebble beach which tapers over approximately 300 m to the west (Figure 1). At both ends of the site, the cliffs have been artificially regraded and disconnected from marine interaction through the construction of a sea wall and promenade. The site is macro tidal with an average spring tidal range of 6.1 m (CCO, 2017), submerging the shore platform and allowing wave interaction with the base of the cliff. Significant wave heights (H_s) measured from the wave buoy at Seaford, show an average of 0.64 m in summer and 1.04 m in winter (CCO, 2017). Average annual precipitation is 720 mm with the majority falling in the winter months. The combination of winter storm waves and wet weather weaken the chalk such that rockfalls are most common in winter months (Mortimer et al., 2004a; Brossard and Duperré, 2004).

Newhaven Chalk is characterized by steeply inclined conjugate joint sets (Mortimore et al., 2004b). These, in conjunction with various bedding planes provide discernible controls on the type and magnitude of failures along this stretch of coastline (Mortimore et al., 2004b). Barlow et al. (2017) undertook kinematic analysis of these cliffs. Results indicate two major joint sets, joint set 1 (JS1) and joint set 2 (JS2) with mean dip/dip direction of 75°/169° and 78°/233° respectively. The mean dip/dip direction of the cliff face is 76°/204° such that wedge failure is indicated (Barlow et al. 2017). Indeed, a large wedge failure involving sliding across both joint sets occurred at Telscombe on March 1st 2017 (Figure 1) and forms the basis for our analysis. Antecedent rainfall for the two days leading up to failure was 17.9 mm and mean H_s for the same period was 2.75 m during periods of high tide (CCO, 2017).

3 METHODS

This research forms part of an on-going monitoring project into coastal erosion along Telscombe Cliffs using UAV photogrammetry. Monthly surveys have been undertaken from July 2016 to present. Our methods involve the use of dense point clouds derived from the UAV photogrammetry data to constrain the geometry of the wedge failure. This section is subdivided

based on the relevant stages of the workflow, these were: installation of ground control, UAV survey, photogrammetric processing, change detection, extraction of joint properties, and back analysis.

3.1 Ground Control

Georeferencing of our models involved 23 ground control points (GCPs), 5 at the cliff top and 18 at the cliff toe (Figure 1). Differential global positioning system (dGPS) was used to locate cliff top GCPs at 5 locations and to set up total station survey markers on the shore platform. Ground control at the cliff toe required the use of a total station in order to overcome the reduction in dGPS accuracy due to the ‘shadowing’ effect of the cliff, a common issue seen in complex terrain morphologies (Young, 2012). After processing (Awange & Kiema, 2013) the dGPS points were accurate to 0.02m in the x , y and z planes with PDOPs varying between 1.2 and 1.5. Total station surveying was used to obtain the cliff toe positions at equidistant locations along the base of the cliff, back sight locations were dual measured pre and post survey to attain an uncertainty value. The maximum deviation from the known dGPS back sight coordinates was 0.006 m such that the maximum error of our GCPs was within about 0.03 m.

3.2 UAV survey

The characterization of sub-vertical slopes using digital photogrammetry is typically successful if a fine (cm) to very fine (mm) resolution is obtained (Sturzenegger and Stead, 2009b). Birch (2006) noted that a ground pixel size of 0.01 m, together with an expected image accuracy of 0.5 pixels is a good conservative value for engineering photogrammetric planning. Mapping at this resolution makes it possible to detect and measure low to extremely high persistence discontinuities (Sturzenegger and Stead, 2009a,b).

Total flight time for each survey was 8 minutes. A full frame Nikon D810 36 mega-pixel camera was used for the surveys with a Nikkor 24mm f/2.8D prime lens. Image capture was achieved at a shutter speed at 1/5000 s in order to minimize distortion associated with camera motion. The aircraft used was a DJI S1000 octocopter flying an automated path that maintained a distance of about 50 m between the camera and the cliff face at a flying height of 21 m. During

flight, live streaming video was used to orient the camera orthogonal to cliff face. The UAV was set to fly at a constant speed of 3 m s⁻¹ while image capture was set to a time interval of five seconds resulting in a photobase of 15 m following a strip plan. This technique is an efficient means for mapping a long stretch of cliff line from a relatively close distance with a short focal length (Birch, 2006). Ground control survey markers were installed on the GCPs in order to georeference the models and test model accuracy.

3.3 Photogrammetric processing

Photogrammetric modelling was undertaken using ADAM 3DM Mine Mapping Suite (Adam Technology, 2010). The interior orientation is based on camera calibration prepared prior to survey. An exterior orientation was undertaken by digitizing the location of the 23 GCPs to an image accuracy of 0.1 pixels based on the center of the survey control markers. Relative Only (RO) points were automatically generated and a bundle adjustment was run based on these points. Resulting exterior orientations had standard errors ranging from 0.03m to 0.06 m. The models had a point density of ~ 351 points/m².

Figure 3: Volumetric analysis of the wedge failure. Negative values indicate erosion.

Erosion along the cliff face was mapped using 2.5 D change detection between sequential surveys (e.g. Rosser et al. 2005). Point cloud data were rasterized to a resolution of 0.1 m such that each grid cell is based on ~3.5 points. A subtraction between sequential raster layers was then undertaken as illustrated in Figure 3. A 10 cm difference threshold was used in order to eliminate false positives.

3.4 Extraction of wedge geometry

The use of dense point clouds derived from close-range digital photogrammetry and terrestrial laser scanning in the characterization of rock slope morphology is well established within the research literature. These methods have the advantage of allowing measurements to be taken from the entire rock surface rather than only those sections accessible to manual measurement (e.g. Sturzenegger & Stead, 2009a; Salvini et al. 2013). The geometry of shear surfaces was extracted directly from the point cloud data by best fitting a plane to the surface points using least squares regression as shown in Figure 4. The same approach was used to extract a generalized geometry for the tension crack at the rear of the failure and the pre-failure cliff face.

Figure 4: Mapping of the principle shear surfaces involved in the wedge failure. JS1 indicated in yellow and JS2 in orange.

3.5 Back analysis

Back analysis was undertaken using a limit equilibrium approach within the Swedge software environment. A detailed description of the numerical formulation is provided in Wyllie & Mah (2004). The failure mass was generalized based on the two principle shear surfaces bounded by the tension crack at the rear and the cliff face at the front as discussed above. However, the density of the point cloud was insufficient to assess

JCR such that this was done manually in the field using a set of reference profiles (Barton, 2013). Joint waviness was extracted by rasterizing joint surfaces to a resolution of 10 cm and taking cross sections parallel to the direction of sliding. Three cross sections were taken from each joint surface and the maximum divergence from mean joint slope was extracted as illustrated in Figure 5 and used in the back analysis.

Figure 5: Cross sectional profiles of joint surfaces used to evaluate joint waviness with maximum divergence from mean slope indicated.

4 RESULTS AND DISCUSSION

Based on the change detection, the total failure volume was estimated at 2547 m³. However, the lower part of the failure scar was obscured by debris (Figure

3) such that this is an underestimate. This volume includes a large section on the eastern side of the wedge. A fractographic examination of river lines evident in the chalk at the top of this section indicates failure radiated outwards horizontally from JS2 (Figure 4) suggesting that the failure of this mass was triggered by the initiation of motion in the wedge. Based on the geometries extracted from the point cloud data, the wedge volume is estimated at 669 m³ or 26% of the total volume.

Profiles extracted from the surfaces of joint sets 1 and 2 are shown in figure 5. The grey dashed lines indicate the mean slope face across the raster with the black dashed lines showing the slope of large scale undulations on the surface. For the JS1 and JS2, the maximum difference measured was 18° and 20° respectively. Joint waviness increases τ in equation 1 by an amount equal to $\sigma_n \tan(\text{waviness})$

The parameters used in Swedge and their assigned values are listed in table 1. As indicated, most of these were extracted from the point cloud data. However, residual friction angle and the specific weight were derived from measurements reported in the literature.

Table 1: Parameters used in back analysis

Parameter	Value	Source
JS1 (dip/dip dir)	74°/165°	point cloud
JS2 (dip/dip dir)	75°/253°	point cloud
Cliff (dip/dip dir)	79°/200°	point cloud
Tension Crack (dip/dip dir)	82°/202°	point cloud
JS1 waviness	18°	point cloud
JS2 waviness	20°	point cloud
JRC (JS1 & JS2)	11	field
JCS (JS1 & JS2)	7.19 MPa	back analysis
Residual friction	30°	Lord et al. 2002
Specific Weight	0.02MN/m ³	Mortimore et al 2004a

The back analysis indicates sliding across both JS1 and JS2 with the failure mass moving outwards along azimuth 207° along the line of intersection between the two joints at an angle of 69°. Back analysis indicates a JCS of 7.19 MPa at the time of failure. This is at the lower end of the range of values reported by Mortimore et al., 2004a which suggests that the chalk surrounding the joint surface was near saturation at the time of failure. Antecedent precipitation in the preceding two days measured 17.8 mm. Infiltration of this water along joint surfaces may explain this low JCS value.

5 CONCLUSIONS

The large range in chalk strength caused by variations in water content and salt weathering are such that it is difficult to transfer laboratory results into models of slope stability. Back analysis of rock slides in chalk is one means of assessing strength at the time of failure. High precision monitoring data derived from UAV photogrammetry can be a useful tool in back analysis as the failure geometry and joint waviness can be extracted from the point cloud data. The rapid rate of data collection is such that large sections of cliff can be monitored (Barlow et al. 2017). For actively eroding sea cliffs, this suggests that many failures can be sampled over a monitoring period of just a few years.

Our results indicate a JCS of 7.19 MPa for the wedge failure. This value is close to the lower end of the range reported by Mortimore et al (2004) suggesting that the chalk along the shear surfaces was close to saturation prior to failure. Although only a single failure is discussed here, with continued monitoring it will be feasible to build up a database of multiple examples in order to better understand the variability of JCS, JRC, and joint waviness. These data can then inform statistical models for each parameter to facilitate better estimates for predicting a factor of safety for potential wedge failures identified in the survey data.

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