Environ Geol (2005) 48: 1002–1013
DOI 10.1007/s00254-005-1323-1

ORIGINAL ARTICLE

Gorka Uribe-Etxebarria
Tomás Morales
Jesús A. Uriarte
Valentín Ibarra

Rock cut stability assessment in mountainous regions

Received: 14 March 2005
Accepted: 25 April 2005

G. Uribe-Etxebarria (✉) · T. Morales
J. A. Uriarte
Department of Geodynamics, Science and Technology Faculty, University of the Basque Country, B°Sarriena s/n,
48940 Leioa, Spain
E-mail: gppurgoj@lg.ehu.es
Tel.: +34-94-6015404
Fax: +34-94-6013500

V. Ibarra
TEAM SL, Parque Tecnológico,
Edif. 207-B, 48170 Zamudio, Spain

Abstract Ensuring stability of rock slopes is an essential requirement in the progress of our societies today. Rock determined to be loose or with potential for failure must be removed or restrained in some way. In our work, after doing an inventory of the instabilities that occurred in the last 5 years in the Basque Country, we analyse the different factors, in slope stability. The potential for failure is evaluated for different classes of rock mass, characterized previously by their geomechanical properties. The characterization of potential risk of each one is undertaken by considering 10 parameters that define the nature of mass rock, relative orientation and morphological features of the slope (interaction rock massif-slope) and infrastructure features (interaction rock massif-slope-infrastructure). Each of these parameters is evaluated separately and a Risk Factor (RF) is determined. The RF reaches a maximum value of 10,000 and allows to differentiate four categories of slopes; each category has its own priority. Rock mass characteristics also determine the potential damage from instability and the associated correction measures. The systematic evaluation of instabilities must allow establishing a priority in the correction measures and thus optimise the available economic resources.

Keywords Rock cut stability · Linear infrastructures · Risk Factor · Mountainous regions

Introduction

The development of instabilities in rock cuts in linear infrastructures is a serious problem with significant economic and social impact. Catastrophic failures of rock cuts can result in property damage, injury and even death.

In the Basque Country this situation appears to be related to three fundamental circumstances: the large amount of tectonically disturbed sedimentary rocks and flyschoid materials, the complex topography of the Basque Country, with significant differences between elevations, and its abundant rainfall.

The development of instabilities depends on the combination of the rock massif characteristics (strength, lithology, structure and degree of weathering), the preservation of the slope and how water enters into the system, depending on the relationship between the rainfall-runoff and the flowing groundwater.

Unforeseen occurrences and derivate damage also depend on the slope morphology and linear infrastructures features.

This combination of factors is the source of a large number of accidents occurring both during construction work and afterwards, leading to the loss of both material resources and lives.

In this context this paper attempts to develop a rock cut stability evaluation system, which allows to identify particularly hazardous slopes and establish a priority for the correction measures. With this aim we did a systematic analysis of the instabilities that happened in linear infrastructures in the Basque Country during the last 5 years.

Geological context

The territory of the Basque Autonomous Community lies, from a geological perspective, in what is called the Basque or Basque–Cantabrian basin, which is the western prolongation of the Pyrenean mountain range (Fig. 1a and b). This basin extends from the Cap Breton submarine canyon in the north to the Duero and Ebro depressions in the south; it is bounded on the west by the Asturian massif, Palaeozoic igneous-metamorphic materials, and on the east, by the Cinco Villas and Alduides Basque massifs.

Inside the Basque Basin, from the work of different authors (Feuillé and Rat 1971; Ramírez del Pozo 1973; Martínez-Torres 1989) and based on structural and sedimentological criteria, four major units or domains are identified: the Basque Arc, the Bilbao Anticlinorium and Alava Platform, the Cantabrian Sierra and the Santander Block. This study focuses on materials within the Basque Arc.

Apart from the Palaeozoic sediments in the Cinco Villas massif, the oldest sediments in the Basque Arc are of Mesozoic Age. The opening of the Atlantic Ocean and the changes in the relative position of Iberia and Europe during this period fundamentally controlled the sedimentation in the basin (García Mondejar et al. 1985).

Initially, during the Triassic in the Basque–Cantabrian basin, continental to epicontinental conditions prevailed forming predominantly sandstones, siltstones, claystones and microconglomerates overlain by brightly coloured clays and evaporites (Keuper facies) with subvolcanic intrusions (ophytes).

The Jurassic deposits consist mainly of limestones, dolomites and marly limestones with layers of siltstones and claystones, deposited on a shallow marine platform. In the Late Jurassic, the rejuvenation and emergence of significant relief occurred in the basin, which favoured the deposition of large quantities of clastic materials. During the Neocomian, sandstones, siltstones and claystones interbedded with marls and limestones were deposited.

A significant marine transgression during the Aptian, enabled the development of reef and parareef constructive complexes. As a result, the most characteristic deposits in the basin during the Aptian-lower Albian are massive limestones with clayey, silty and sandy materials (Urgonian Complex).

In the Albian, there was an episode of major subsidence throughout the basin, coinciding with the rejuvenation of relief in the surrounding continental areas. Deposition of clastics consisting of dark claystones rhythmically interbedded with sandstones (Durango Formation and Black Flysch) occurred.

In the Late Cretaceous, within the Basque Arc domain, the basin generally opened and deepened. Sedimentation is chiefly of a turbiditic nature and is characterised by marls interbedded with marly limestones in the lower part (calcareous Flysch), evolving to more sandy facies in the upper part (detrital-calcareous Flysch). These deposits include large quantities of volcanic materials, mainly basalts, in the form of lava flows and volcaniclastic layers.

A major change from extension to compression in the Basque–Cantabrian basin occurred in the Palaeocene-lower Eocene. The deposition of marls and marly limestone materials continued, typical of a deep marine environment. In some areas, breccioid and conglomeratic materials were deposited. In the Eocene, detrital or calcareous turbidites were again deposited.

Tectonic pulses, which lasted until the Middle-Upper Miocene period, led to compression of the area and the emergence of the basin complex; a rejuvenated relief appearing as the final response to the Alpine orogeny.

Analysis of rockfall hazards

As Hoek (2000) points out "Rockfalls are generally initiated by some climatic or biological event that causes a change in the forces acting on rocks. These events may include pore pressure increase due to rainfall infiltration, erosion of surrounding material during heavy rain storms, and freeze-thaw processes in cold climates, chemical degradation or weathering of the rock, root growth or leverage by roots moving in high winds".

The nature and characteristics of the intact rock, together with characteristics of joint sets and degree of weathering, determine the mechanical behaviour and type of instability that can be developed in the rock cut.

Once the movement of a rock has been initiated, the most important factor controlling its fall trajectory is the geometry of the slope, in particular the dip of the slope faces. Height and dimensions of the slope also determine the probability of rockfall events and their magnitudes.

Finally, the possibility that falling rocks can reach a roadway, railway or any linear infrastructure, and the damage they can produce, complete the rockfall risk analysis.

Thus, the evaluation of the risk associated with a particular slope is generally estimated by the product of two components: probability of instability to occur and consequence analysis.

For a specific site, the slope stability can be analysed determining the corresponding Safety Factor (SF). The frequency distribution of SF can be characterized in

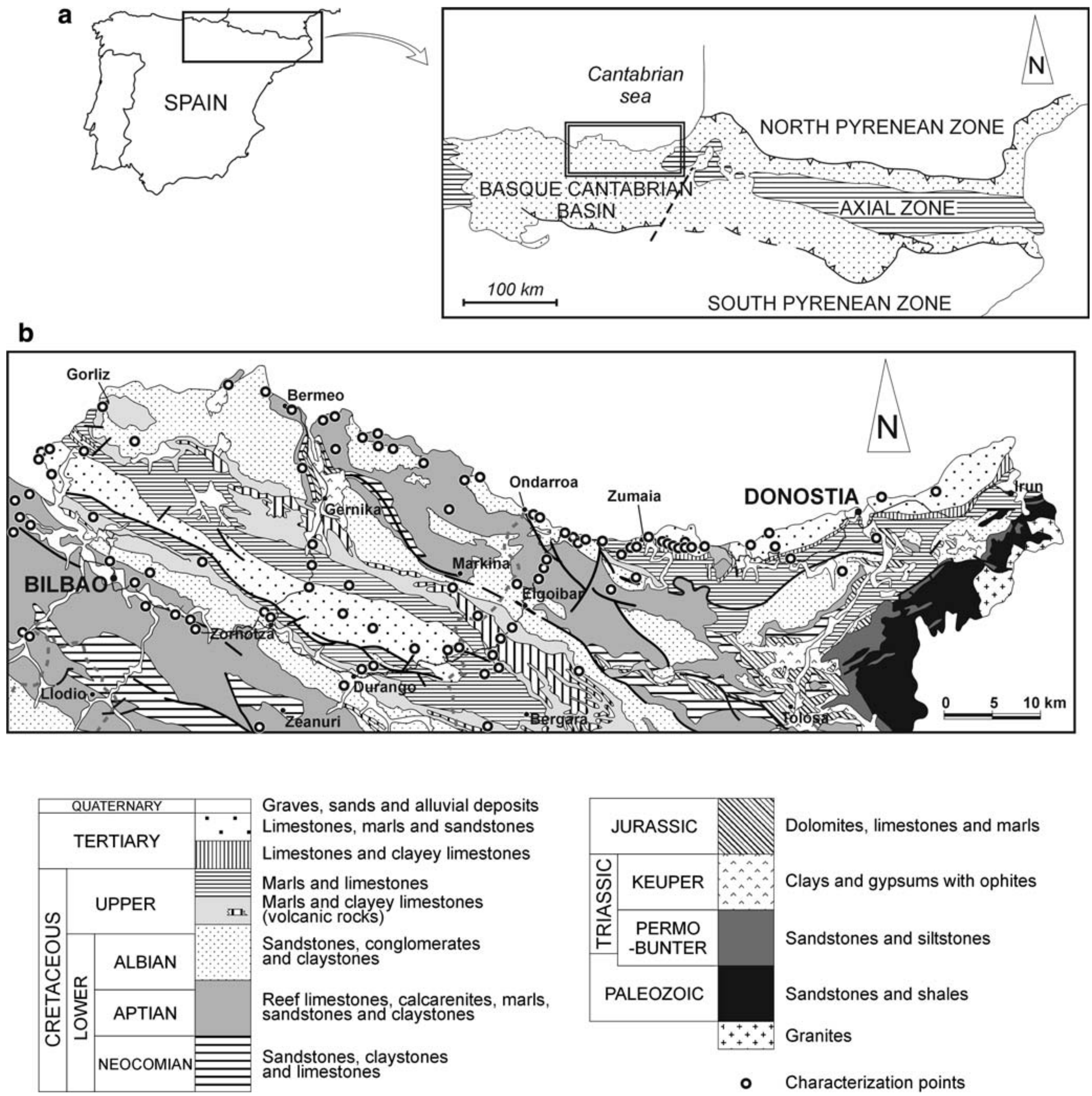


Fig. 1 **a** Sketch map of the main structural features of the Pyrenean belt (modified from Boillot and Capdevilla1977) and **b** localization map for rock cuts characterized in the Basque Country and geological framework

terms of its mean value and standard deviation (Morgenstern1997).

Extended length of linear infrastructures makes it difficult to obtain sufficient information for the stability assessment for all slopes along the route (Hoek2000). As a result, its necessary to develop classification schemes to allow particularly hazardous slope identification. These slopes require urgent remedial works prioritising associated risks and optimising economic resources. This scheme must disclose information for further detailed studies to be undertaken.

In this context, and in terms of rockfall hazard assessment, pioneering work was done by Brawner and Wyllie (1975) Wyllie (1987), Hungr and Evans (1989), Badger and Lowell (1992). Presently, the most widely accepted system is the Rock Fall Hazard Rating System (RHRS) developed by Pierson et al. (1990) and Pierson and Van Vickle (1993). This system and modifications to it have been implement by different authors in different contexts (Franklin and Senior 1997; Miller 2003).

Rockfall hazard evaluation

In our work, rockfalls that occurred in linear infrastructures of the Basque Arc in the last 5 years were studied. The study included geological, geotechnical, morphological, hydrological and hydrogeological characterization, and a historical review.

Rock massif's characterization refers to both intact rock and joints set. Rock cut morphology, stretches, height and dip are determined in the field. Analyses of the shape and size of detached elements, reach, damage and antecedents complete the inspection of the zone.

The stability of rock slopes and damage are recognized to be determined by the following main parameters: rock mass characteristics, joint orientation, hydraulic conditions, durability, slope history, detached rock block size/rock mass volume, slope height, communication line (CL) reach probability, sight distance/decision sight distance and way category. These 10 parameters are grouped into two categories: Occurrence Factor (OF) and Consequence Factor (CF).

The OF refers to the possibility of detachment of rock fragments from the slope. The CF refers to the potential risk of rockfall to impact on the CL.

Parameters for Occurrence Factor

Rock mass characteristics

Behaviour of a slope excavated in rock depends first on geomechanical characteristics of the rock mass, its strength characteristic, nature, structure and degree of weathering.

In this work, characterization is approached by a graphic classification procedure (Morales et al. 2004) from Geological Strength Index (GSI) values, which provide basic information on the structure of the massif, and from uniaxial compressive strength (σ_{ci}) values which provide information of the strength of the materials. The GSI is established using the method proposed by Hoek (2000) and adapted by Marinos and Hoek (2001) for heterogeneous rock masses such as flysch.

We also measured uniaxial compressive strength (σ_{ci}) on-site, using a Schmidt hammer. The process, according to different authors (Deere1964; Haramy and De Marco 1985; Romana1992; Kolaiti and Papadopoulus1993), consists of performing 20 tests on smooth areas, free of cracks and joints, discarding abnormal results and taking the mean of half the tests with the highest results as the *N* index. Where drilling was possible, paraffined rock samples were collected for laboratory testing. We should mention here that, in practice, it is difficult to obtain representative samples of the worst quality materials, especially when there are alternating weak and strong materials. The laboratory tests consisted of failure tests on 76-mm-diameter rock samples to determine the uniaxial compressive strength (σ_{ci}) of the intact rock.

The relation between the value of Schmidt hammer rebound number (N) and the uniaxial compressive strength (σ_{ci}) is shown in Fig. 2 (Morales et al. 2004). Table 1 shows the results of the regression analysis applied to both parameters. The corresponding empirical equation is $\sigma_{\mathrm{ci}} = \exp(1.332 + 0.053\ N)$. The test results show a very good correlation with a correlation coefficient (R) of 0.941. This relationship is statistically significant according to the Student's *t*-test at the 95% confidence level. The residuals show a distribution between ±2 standard deviation (SD) with positive and negative residuals balanced. The distribution of the residuals also indicates that the ratio is statistically significant.

The comparison of GSI and σ_{ci}values allows distinguishing rock masses according to their geotechnical properties and grouping rock masses into eight classes

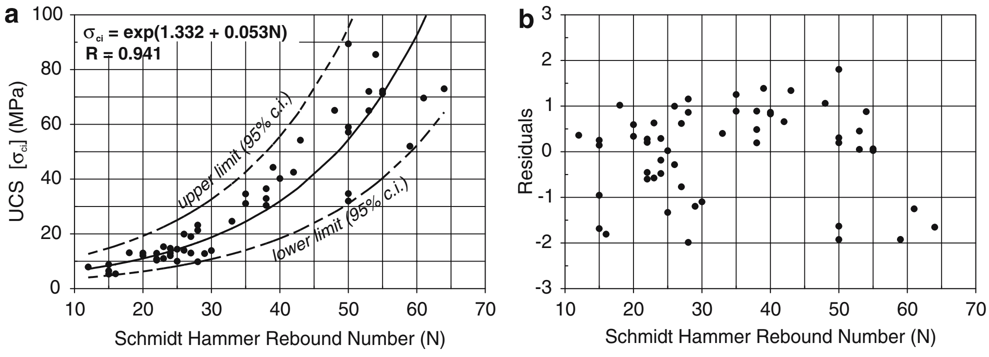


Fig. 2 **a** Relationship between uniaxial compressive strength (σ_{ci}) and Schmidt hammer rebound number (N) and **b** standardised residuals (Morales et al. 2004)

Table 1 Regression analyses results of uniaxial compressive strength (σ_{ci}) and Schmidt hammer rebound number (N) (Morales et al. 2004)

Coefficients						
Parameter	Estimate	Standard error	T value	Probability level	95% confidence interval	
					Lower limit	Upper limit
Exponential model: $\sigma_{ci} = \exp(a + bN)$						
a	1.332	0.097	13.690	0.000	1.137	1.527
b	5.329E-02	0.0031	30.031	0.000	0.048	0.059
Model	Sum of squares	d.f.	Mean square	F-ratio	Probability level	
Analysis of variance						
Regression	30.048	1	30.048	401.258	0.000	
Residual	3.894	52	0.075			
Total	33.942	53				
Correlation coefficient (R)	Regression coefficient (R^2)	Standard error of estimate				
Model summary						
0.941	0.885	0.27365				

using a graphic classification procedure (Fig. 3; Morales et al. 2004).

Joint orientation

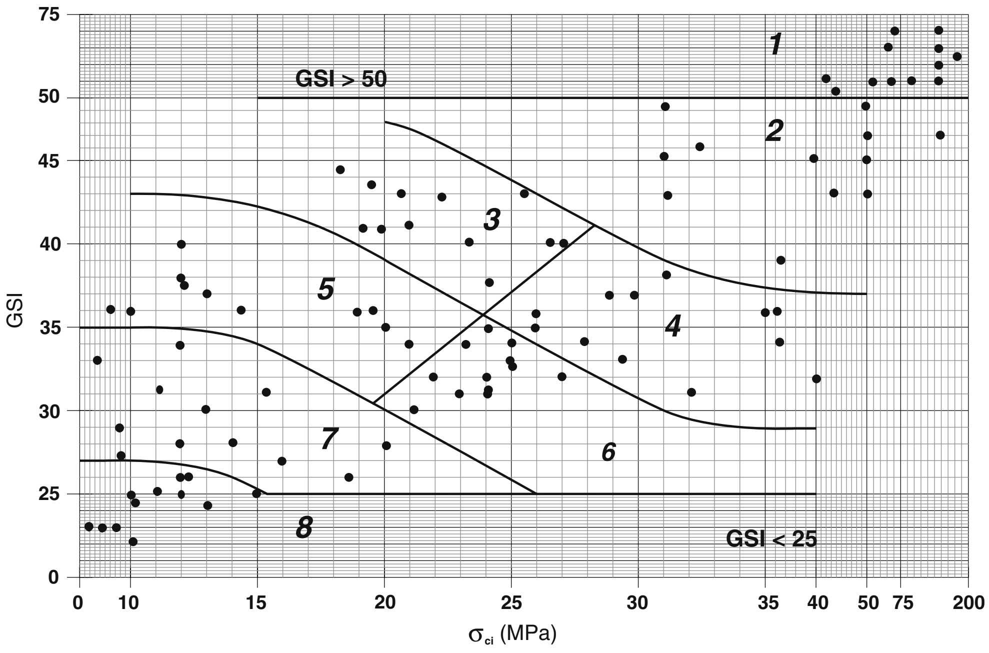
The relation between the orientations of the joint and the slope permits identifying potential instability elements and shows their degree of development. In our case, four categories are established: very unfavourable, unfavourable, favourable and very favourable orientation.

This parameter can be estimated by using the chart proposed by Moon et al. (2001). Having sufficient information, this parameter can be evaluated by a cinematic analysis of instabilities and the determination of the corresponding SFs (Fig. 4).

Hydraulic conditions

Possible water input into the system is a very important factor for the global slope stability. Water contributes to weathering and movement of rock materials. In our work we consider the five categories established by Bieniawski (1989).

Fig. 3 Distribution of characterized rock masses in a GSI versus σ_{ci} graph (modified from Morales et al. 2004)



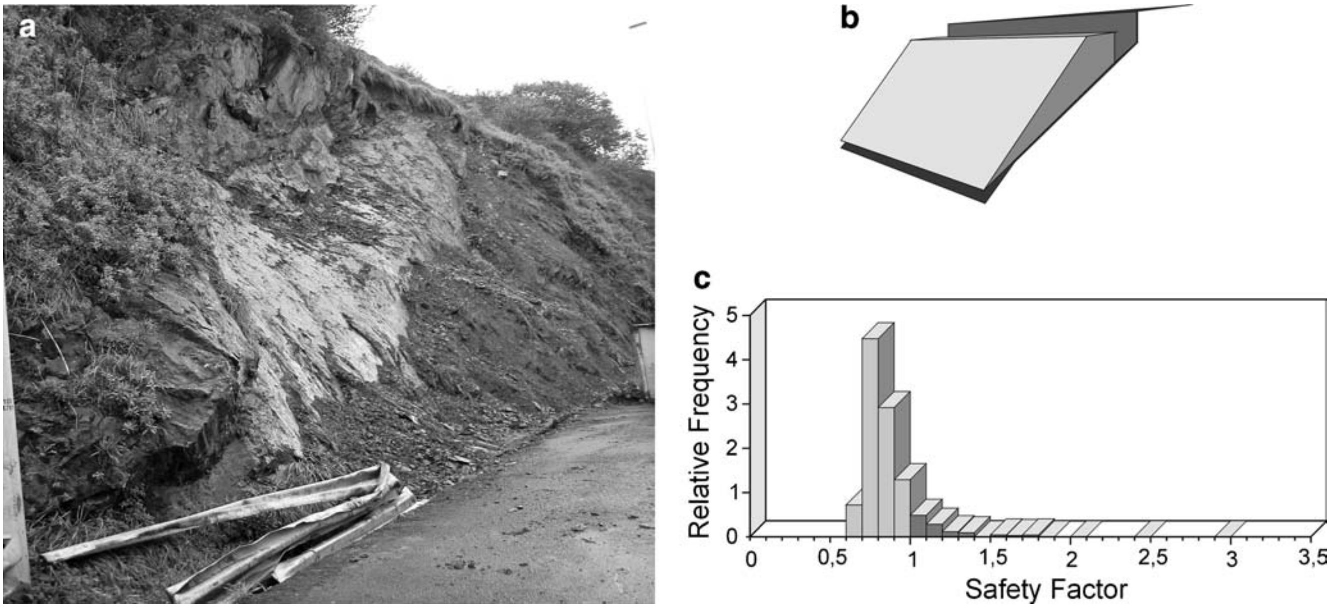


Fig. 4 **a**Photography of planar failure, **b** planar failure and **c** statistical distribution of Safety Factor for the planar failure

A detailed analysis of the watershed above the slope and its hydrogeological context may help in a better fitting of this parameter.

Durability of the intact rock

Stability of slopes can be affected in time by the susceptibility of rock massifs to weathering depending on their lithological and mineralogical features. Atmospheric exposure or water presence can reduce the strength of the rock mass with the corresponding loss of their geomechanical properties particularly in slopes and embankments (Bell and Pettinga1988; Read and Millar1990).

Susceptibility of intact rock to weathering can be assessed by the Slake Durability Index (ISRM1979). In our work, the score of Slake Durability Index has been established from the classification of Gamble in Goodman (1989).

Fig. 5 **a**Photography of rockfall, **b** analysis of blocks trajectories and **c** horizontal locations of block end-points

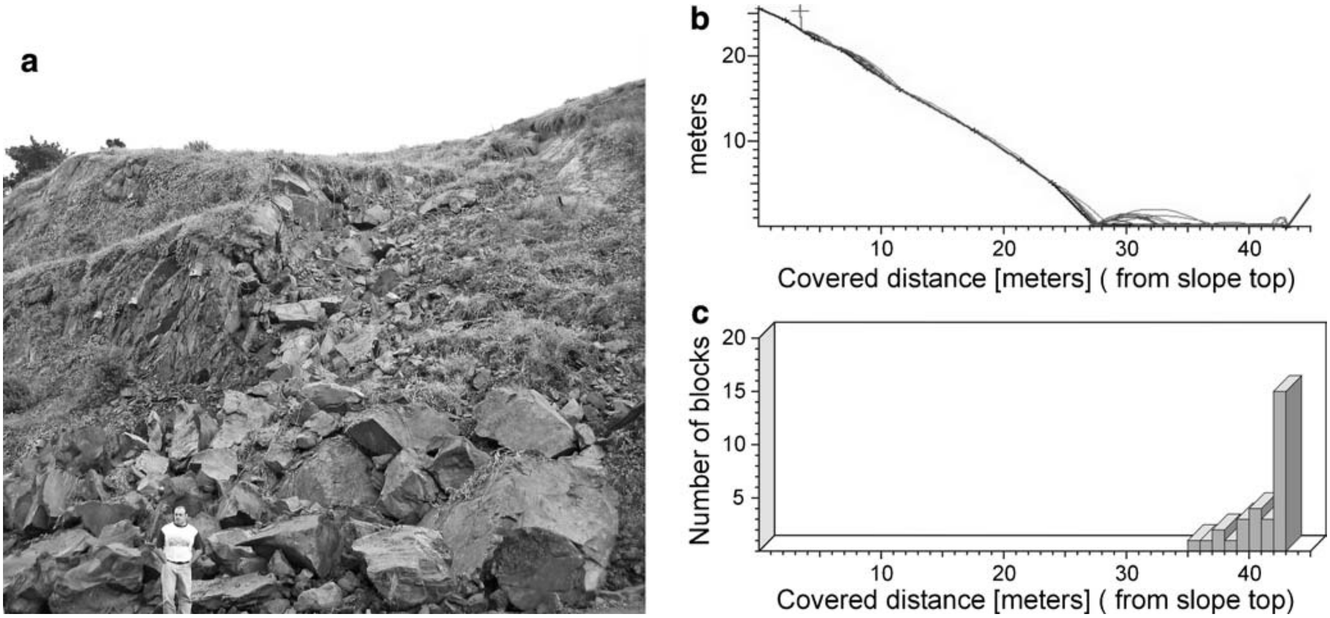


Table 2 Determination of decision sight distance as function of maximum speed limit (modified from Hoek1998)

Speed limit (km/h)	Decision sight distance (m)
50	140
60	170
70	200
80	230
90	275
100	315
120	350

Slope history

This refers to rockfall history of the slope in time. This information is an important check on the potential for future rockfalls (Hoek2000).

Rockfall activity can be established from documentation on maintenance costs and from the managers of the infrastructure.

Parameters for Consequence Factor

Rock block size/rock mass volume

Rock mass characteristics determine the type of rockfall event that is most likely to occur: individual blocks or mass of materials. In the first case, block size determines the maximum energy that can be developed. In the second case, the volume of material mobilized in an event is determined from the maintenance history or estimated from the observed conditions.

Slope height

Slope height assesses the system's potential energy (Pierson et al. 1990; Hoek2000). There is thus, a direct relationship between slope height, maximum reach distance of rock fragments, maximum impact energy and, therefore, maximum hazard and potential damage.

Probability of reaching CL

Previous parameters being the same, the probability of rock fragments reaching CLs depends on CL-slope distance, dip of slope face and ditch effectiveness.

This parameter can be evaluated by experienced geologists or by the analysis of rockfall trajectories (Fig. 5).

Sight distance/decision sight distance

The sight distance is defined as the minimum distance required for detecting a hazard to the CL. Minimum decision sight distance, as function of vehicular speed (Table 2), is included in Willie (1987) and Hoek (1998).

The ratio between the sight distance and the minimum decision sight distance determines the ability to notice an obstacle in the CL.

Communication line (CL) category

The last parameter considered is the CL category. The CL category is set according to the traffic volume and hierarchy of the CL in the communication system of the region.

Results obtained

The rock cut stability was characterized from 96 slopes on different materials and locations in the Basque Country. The features, properties and slope behaviour allow the development of a system to quantify each

Table 3 Parameters for OF and their ratings

Parameters		Range of Values							
1	*Material Class	I	II	III	IV	V	VI	VII	VIII
	Rating	1	5	10	15	18	22	27	30
2	Joint orientation	Very favourable		Favourable		Unfavourable		Very unfavourable	
	Rating	1		5		10		20	
3	Hydraulic conditions	Dry	Damp	Wet		Dripping		Flowing	
	Rating	1	3	5		10		20	
4	Durability	Very low (**$Id_2 < 30\%$)		Low (30–60%)		Medium (60–85%)		High-very high (>85%)	
	Rating	10		5		3		1	
5	Slope history	No evidence of instabilities	Few instabilities	Sporadic instabilities		Frequent instabilities		Continual instabilities	
	Rating	1	5	10		15		20	

Table 4 Parameters for CF and their ratings

	Parameter	Range of Values					
1	Rock block size (m) or soil mass volume (m^3)	< 0.3 < 0.25	0.3–0.5 0.25–1	0.5–1 1–8	1–2 8–60	> 2 > 60	
	Rating	1	5	10	20	30	
2	Slope height (m)	< 2	2–5	5–10	10–15	15–20	> 20
	Rating	0	5	10	13	17	20
3	CL reach probability	Low	Medium	High	Sure		
	Rating	5	10	15	20		
4	Sight distance/decision sight distance	> 2	1.5–2	1–1.5	< 1		
	Rating	5	10	15	20		
5	CL category/CL	Low traffic	Medium traffic	High traffic	With top priority		
	Rating	2	5	8	10		

considered parameter incidence in the global massif stability (Tables 3 and 4). The sum of the parameters that make up each of the considered factors reaches a maximum punctuation of 100. The maximum RF, as the product of the two previous factors, reached a total of 10,000.

Figure 6 shows the graphic representation of the 96 rock cuts considered, comparing OF values with CF values. This representation allows us to distinguish four different levels of risk associated with each slope: very high risk (RF > 4,500), urgent remedial action is required; high risk (4,500 > RF > 3,000), remedial action is required; moderate risk (3,000 > RF > 1,500), monitoring and control is recommended; low risk (RF < 1,500), low priority of action.

This characterization permits us also to perform a comparative study of rock cut stability as function of the geomechanical properties of the rock mass. Thus, there is a direct relationship between the geomechanical properties of the rock masses and the type and entity of the instability events most likely to occur. Table 5 shows the geomechanical properties of the different rock mass classes and the types of associated instabilities.

In general, instability problems in rock cuts on very competent rock masses (class 1) are local and related to the detachment of blocks, often of very large size. The larger fracturation rate of class 2 results in more frequent rockfalls, particularly during periods of heavy rain. Structural failures are the most common type of instability in rock masses of class 3. In rock masses of class 4, the development of instabilities is hardly determined by the orientation of the discontinuities, and large instabilities can appear.

Fig. 6 Distribution of characterized rock masses in an OF versus CF graph

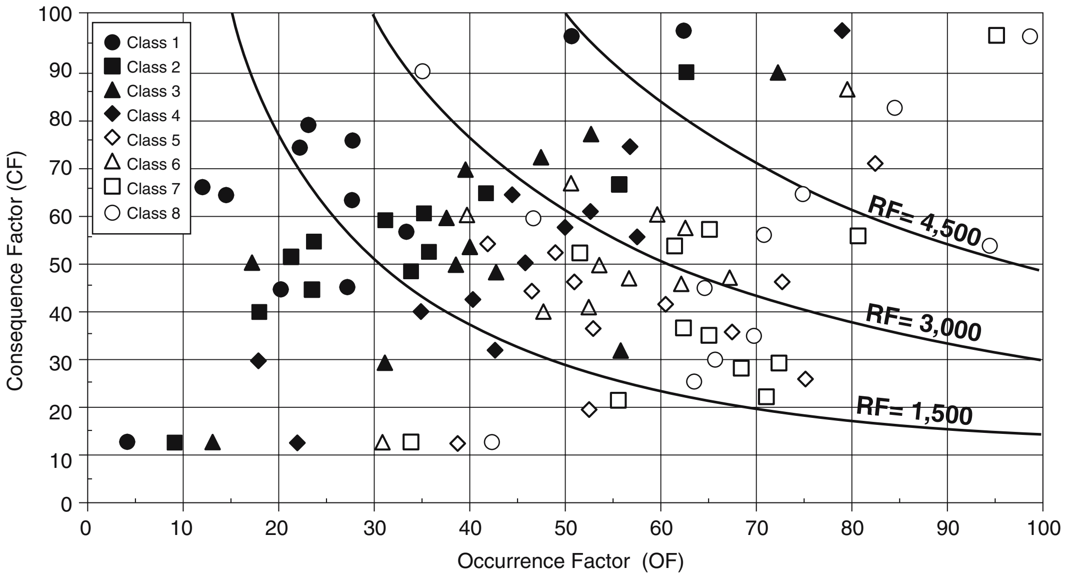


Table 5 Geomechanical properties, types of instabilities and reinforcement design for rock mass classes

Class	Geomechanical properties	Types of instability	Corrective measures
Class 1	Very competent rock masses, made up of high compressive strength (σ_{ci}) materials, massive or thick-bedded. The discontinuities affecting the rock mass do not constitute persistent and regular sets of weakness (GSI over 50). They are generally either very compact sandstone and limestone or materials of igneous origin	Overall instability is not to be expected. Locally structural failures can develop and very large blocks occasionally become detached. Reaching of these blocks may be important	Because of large mass of detached blocks, their energy is usually very high. Most suitable measure is spot bolting of potential unstable blocks andthe use of mesh dropped over the rock. The installation of dynamic fences in the slope would be suitable too
Class 2	Rock masses consisting of highly competent materials, with σ_{ci} values similar to class 1, affected by wide-spaced discontinuity sets (GSI values under 50). The rocks are basically thick-bedded blocky sandstones, limestones and igneous materials	Rock masses are competent overall. However, there are regular rockfalls, particularly during periods of heavy rain. The size of the detached blocks depends on the discontinuity pattern	The combined use of mesh dropped over the rock face and catch ditches, or dynamic fencing at toe of the slope is suitable. Locally, it may be necessary to resort to bolting
Class 3	The materials making up the rock mass have a σ_{ci} of less than 30 MPa and relatively high GSI values (from 35 to 45). They are generally very competent marls or claystones	Rock mass is generally competent. However, the development of structural failures is relatively common. Wedges and planar failures leads to the detachment of medium and large size blocks depending on the discontinuity sets	Systematic bolting, spaced from 1.5 to 2.5 m, and the installation of mesh is recommended. Locally, it may be advisable to protect the slope with shotcrete (50 mm)
Class 4	Rock masses with a GSI from 25 to 40 and a medium-high σ_{ci} value (over 25 MPa). This group largely consists of rock masses in which competent materials, predominant (generally sandstone or limestone) and alternate with less competent layers (argillaceous marls, siltstone or claystone)	Behaviour of rock cuts depend strongly on the orientation of discontinuities. When the orientation of discontinuities is favourable, blocks may frequently fall and their size mainly depends on the thickness of competent layer. When the orientation of discontinuities is unfavourable, the arrangement of competent materials above incompetent materials may cause large failures, depending on the strength of the plane of weakness	When discontinuities orientation is favourable, rockfall can be corrected by installing mesh and protective fencing or walls. Installation of bolts is also recommended to contain the fall of the larger rocks. When discontinuities orientation is unfavourable, the most effective solution may be stretch out the slope
Class 5	Rock masses consisting of materials with a σ_{ci} of less than 25 Mpa, basically marls and competent siltstone. Discontinuity rating is relatively high (GSI between 30 and 40)	Rock cuts are generally stable. The number of rock fragments detached may be high, but usually they are mainly small blocks. However, depending on its discontinuity rating and the durability of materials, the evolution of the slope over time could be problematic	Construction of ditch guards, ditches and mesh to hold the falling fragments is recommended. The application of a layer of shotcrete will prevent the slope from becoming degraded over time
Class 6	These are rock masses made up of materials of medium compressive strength (σ_{ci} over 20 MPa) or equal amounts of layers of very different strength. The GSI value is between 25 and 35	Rock mass may be relatively competent depending on its discontinuity rating and materials durability. If there are alternations in lithologies, the behaviour of the rock mass depends on the orientation of the discontinuity sets. If it is favourable, relatively small rock fragments could fall frequently; if the discontinuity orientation is unfavourable, important complex failures could develop	Slopes have to be protected with a layer of shotcrete(50–100 mm). It may be reinforced by systematic bolting (spacing 1–2 m) and mesh. When the discontinuity orientation is unfavourable, the most effective solution may be to extend the slope
Class 7	Rock masses consisting of siltstone and alternations with predominant siltstone (σ_{ci} under 20 MPa). Their discontinuity rating is high, with significant weathering (GSI ranging from 25 to 35)	Rock mass has low geomechanical properties. The most common forms of instability consist of the fall of small rock fragments, often in considerable number. Circular and irregular failures can occur in the most weathered areas	Falling material makes it advisable to install catch ditchs at toe of the slope and apply shotcrete (100–150 mm). Other effective method of stabilisation consists of the construction of gentle slopes with revegetation

Class 8	Weak rock masses result of intense jointing or because the rock material itself have low strength. This class also includes very weathered rock masses. The GSI is under 27 and the σ_{ci} is no more than 15 Mpa	Circular failures and rotating landslides are the most characteristic forms of instability in slopes. Depending on the fracture rating and weathering, these failures could be relatively deep	When failures are shallow, the most common corrective measure consists of smoothing the shape of the slope, reducing its gradient and revegetation. Whether failures are depth, it may be necessary to build rock fills at the toe of slope, to install anchored walls, or even to drive systematic piles in the slope

Class 5 is characterized by the detachment of a large number of blocks, usually of small size; the durability of materials can determine significantly the behaviour of the talus with time. Discontinuity orientation and materials durability are the main parameters controlling the development of failures in rock masses of class 6. Rock masses of class 7 have low geomechanical properties; the most common form of instability is the falling of small rock fragments; circular and irregular failures can occur in the most weathered areas. Finally, class 8 is a very weak rock mass resulting from intense jointing, weathering processes or low strength. Circular failures and rotating landslides are the most characteristics forms of instabilities (Fig. 7).

Consequences and damage depend not only on the rock mass characteristics, but on rock cut features and height also. Distance to the CL, sight distance and CL category complete the potential risk associated with a particular rock cut. The correction measures depend again on the class of the rock massif. Table 5 shows the corrective measures suitable for each class of rock mass.

Conclusions

In mountainous regions, where high rockfall hazard exists, prevention of slope instabilities is often considered. In order to optimise the existing resources, it is necessary to establish priorities in short and long terms. Afterwards, annual measures will be taken on those slopes, which show high potential risk. Therefore, it is necessary to design preventive programs, which allow us to identify, classify and resolve rockfall hazards.

Thus, in the present work, and from the information collected from 96 slopes of the Basque Country, we analyse the factors in their stability. Related to the possibility of falling material (OF), the basic parameters considered are: rock mass characteristics, joint orientation, hydraulic conditions, durability and slope history. Regarding the reach and the damage instabilities can produce (CF), parameters to consider are: rock block size/rock mass volume, slope height, CL reach probability, sight distance/decision sight distance and CL category.

The RF, determined as the product of the two previous factors, allows us to distinguish different levels of risk associated with different slopes: very high risk (RF > 4,500), urgent remedial action is required; high risk (4,500 > RF > 3,000), remedial action is required; moderate risk (3,000 > RF > 1,500), monitoring and control is recommended; low risk (RF < 1,500), low priority of action.

Comparative studies can be done on the rock cut stability as function of the geotechnical properties of massifs, and their behaviour, type and degree of associated problems, and type and effectiveness of solutions provided can be determined.

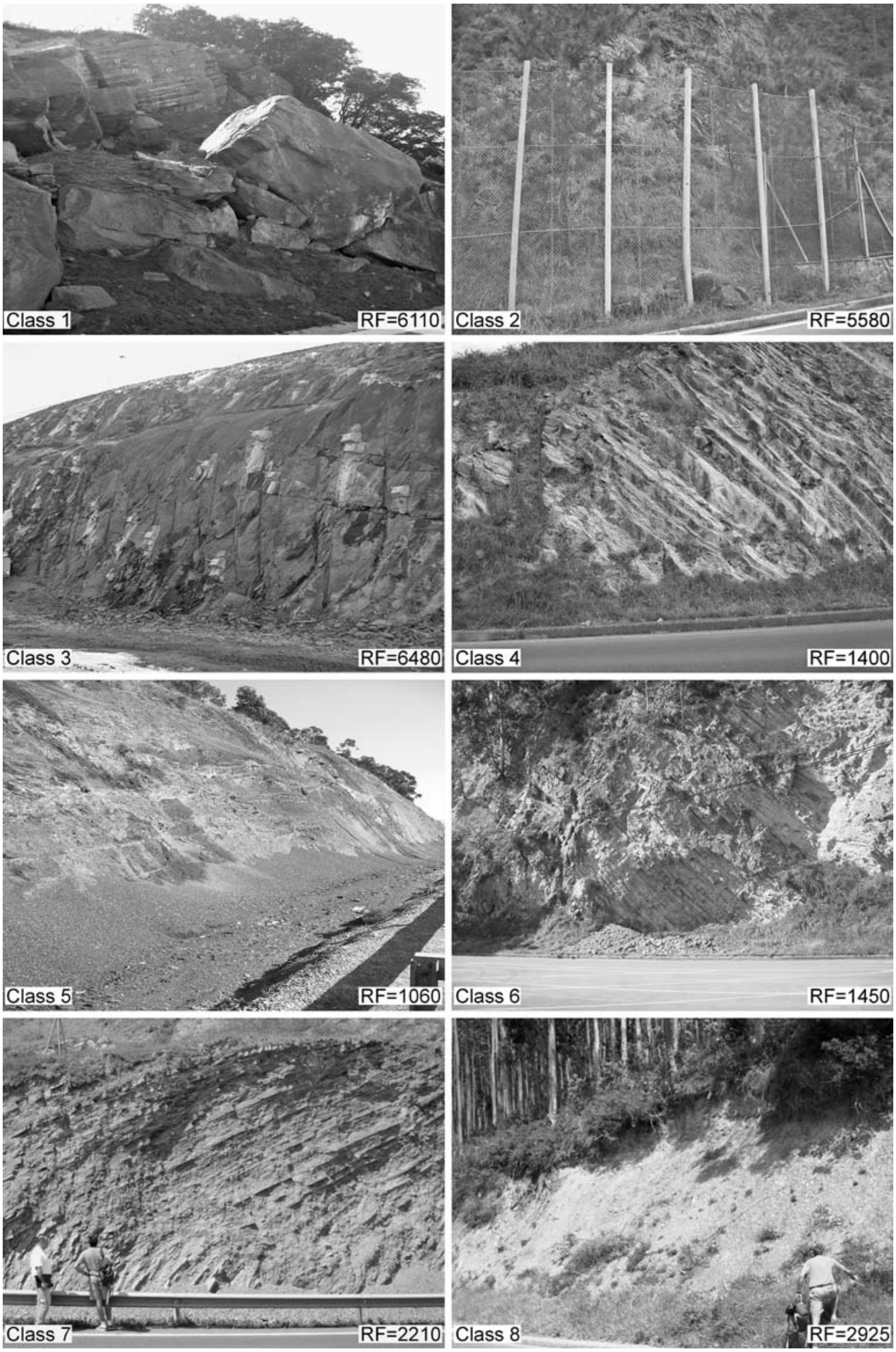


Fig. 7 Examples of rock mass classes

Thus, it is possible to arrange an effective tool for the systematic evaluation of slopes that allows particularly hazardous slopes to be identified, and to prioritise the repair work according to urgency and economic criteria.

Acknowledgements This work was supported by The University of the Basque Country. Project 1/UPV 00001.310-E-13915/2001: Caracterización de la inestabilidad de taludes en macizos interestratificados del País Vasco: aspectos geomecánicos, interacción precipitación-inestabilidad y análisis de riesgos.

References

Badger TC, Lowell S (1992) Rockfall control Washington State. In: Rockfall Prediction and Control and Landslide Case Histories, Transp. Research Record, National Research Council, Washington, 1342:14–19

Bell DH, Pettinga JR (1988) Bedding-controlled landslides in New Zealand soft rock terrain. In: Proceedings of the fifth international symposium on landslides, Lausanne, pp 77–83

Bieniawski ZT (1989) Engineering rock mass classifications. Wiley, NY, p 251

Boillot G, Capdevilla R (1977) The Pyrenees: subduction and collision? Earth Planet Sci Lett 35:151–160

Brawner CO, Wyllie DC (1975) Rock slope stability on railway projects. In: Proceedings of the American Railway Engineering Association Regional Meeting, Vancouver, BC; American Railway Engineering Association, Washington, DC

Deere DU (1964) Technical description of rock cores for engineering purposes. Rock Mech Eng Geol 1(1):16–22

Feuille P, Rat P (1971) Structures et páleogéographie pyréneo-cantabrique. Hist St du golfe de Gascogne Edn Technip 2:V-1-1–V-1-48

Franklin JA, Senior SA (1997) Outline of RHRON, the Ontario rockfall hazard rating system. In: Proceedings of the international symposium on engineering geology and environment, Athens, Greece, pp 647–656

García Mondejar J, Hines FM, Pujalte V, Reading HG (1985) Sedimentation and tectonics in the western Basque–Cantabrian area (northern Spain) during Cretaceous and Tertiary times. In: Mila MD, Rosell J (eds) Sixth European Regional Meeting. Excursion guidebook, pp 307–392

Goodman RE (1989) Introduction to rock mechanics. Wiley, NY

Haramy KY, De Marco MJ (1985) Use of the Schmidt hammer for rock and coal testing. In: Proceedings of the 26th U.S., symposium on Rock Mech. Rapid City, pp 549–555

Hoek E (1998) Reliability of Hoek–Brown estimates of rock mass properties and their impact on design. Int J Rock Mech Min Sci 35(1):63–68

Hoek E (2000) Practical rock engineering. Course notes. Evert Hoek Consulting Engineer Inc., Canada

Hungr O, Evans SG (1989) Engineering aspects of rockfall hazard in Canada. In: Geological Survey of Canada, Open File 2061

ISRM (international Society for Rock Mechanics) (1979) Suggested methods for determination of the slake durability index. Int J Rock Mech Min Sci Geomech Abstr 16:154–156

Kolaiti E, Papadopoulus Z (1993) Evaluation of Schmidt rebound hammer testing: a critical approach. Bull Int Assoc Eng Geol (IAEG) 48:69–76

Marinos P, Hoek E (2001) Estimating the geotechnical properties of heterogeneous rock masses such a flysch. Bull Eng Geol Env 60(2):85–92

Martinez-Torres LM (1989) El manto de los mármoles (Pirineo Occidental): Geología estructural y evolución geodinámica. PhD Thesis, University of The Basque Country

Miller M (2003) Development and implementation of the Idaho Highway Slope Instability and Management System (HISIMS). National Institute for Advanced Transportation Technology (NIATT), University of Idaho

Moon V, Russell G, Stewart M (2001) The value of rock mass classification systems for weak rock masses: a case example from Huntly, New Zealand. Eng Geol 61:53–67

Morales T, Uribe-Etxebarria G, Uriarte JA, Fernandez de Valderrama I (2004) Geomechanical characterisation of rock masses in Alpine regions: the Basque Arc (Basque–Cantabrian basin, Northern Spain). Eng Geol 71:343–362

Morgenstern NR (1997) Toward landslide risk assessment in practice. In: Cruden DY, Fell R (eds) Landslide risk assessment. A. A. Balkema, Rotterdam, Brookfield, pp 15–24

Pierson LA, Van Vickle (1993) Rockfall hazard rating system-participants manual. In: Federal Highway Administration Report FH-WA-SA-93-057, FHWA, Washington, DC

Pierson LA, Davis SA, Van Vickle R (1990) Rockfall hazard rating system implement manual. In: Federal Highway Administration (FHWA) Report FHWA-OREG-90-01. FHWA, U.S. Department of Transportation

Ramirez Del Pozo J (1973) Síntesis geológica de la provincia de Álava. Obra Cultural de la Caja de Ahorros de la Ciudad de Vitoria

Read SA, Millar PJ (1990) Characterisation and classification of New Zealand Tertiary age sedimentary soft rocks. Road Research Unit, Transit New Zealand, Bull 84:1–25

Romana M (1992) Métodos de corrección de taludes según la clasificación geomecánica SMR. In: III Simposio Nacional sobre Taludes y Laderas Inestables, La Coruña, pp 629–650

Wyllie D (1987) Rock slope inventory/maintenance programs. In: FHWA Rockfall Mitigation Seminar, 13th Northwest Geotechnical Workshop, Portland, Oregon