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# ROCKERY DESIGN AND CONSTRUCTION GUIDELINES

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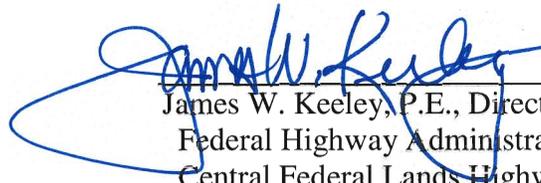


**Central Federal Lands Highway Division  
12300 West Dakota Avenue  
Lakewood, CO 80228**

## FORWARD

The Federal Lands Highway (FLH) of the Federal Highway Administration (FHWA) promotes development and deployment of applied research and technology applicable to solving transportation-related issues on Federal lands. The FLH provides technology delivery, innovative solutions, recommended best practices, and related information and knowledge sharing to Federal agencies, Tribal governments, and other offices within the FHWA.

The objective of this study is to review existing analytical methods and construction techniques currently in use for design and construction of rockeries and to develop a unified framework for design and specification of rockeries in modern highway construction. The ultimate goal of the project is to provide designers, inspectors, and contractors with a basis for evaluating existing rockeries and specifying and constructing new rockeries.



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James W. Keeley, P.E., Director of Project Delivery  
Federal Highway Administration  
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16. Abstract Rockeries consist of earth retaining and/or protection structures comprised of interlocking, dry-stacked rocks without mortar or steel reinforcement. They have been used for thousands of years and rely on the weight, size, and shape of individual rocks to provide overall stability. Some of the earliest rockeries constructed by the Federal Government date back to 1918. Within the private sector, commercially built rockeries have been constructed in the Pacific Northwest for at least the last four decades and in Northern California and Nevada for at least the last 10 years. As rockery design procedures tend to vary regionally, studies were performed to determine the methods by which rockeries are designed and constructed in various regions throughout the western United States. These design methods were then compared using several typical rockery design loading conditions to determine how the resulting rockery designs differ and which methods are most appropriate for a proposed design for the FHWA's FLH Divisions. Based on the research performed, a rational design methodology, which evaluates rockery stability as a function of the rockery geometry (height, base width, and batter), rock properties and placement, and lateral pressure imposed by the backfill materials, was developed. A sample design problem is included. Recommendations for specifying and constructing rockeries that are consistent with the design methodology are also provided.			
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# SI\* (MODERN METRIC) CONVERSION FACTORS

## APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
<b>LENGTH</b>				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
<b>AREA</b>				
in <sup>2</sup>	square inches	645.2	square millimeters	mm <sup>2</sup>
ft <sup>2</sup>	square feet	0.093	square meters	m <sup>2</sup>
yd <sup>2</sup>	square yard	0.836	square meters	m <sup>2</sup>
ac	acres	0.405	hectares	ha
mi <sup>2</sup>	square miles	2.59	square kilometers	km <sup>2</sup>
<b>VOLUME</b>				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft <sup>3</sup>	cubic feet	0.028	cubic meters	m <sup>3</sup>
yd <sup>3</sup>	cubic yards	0.765	cubic meters	m <sup>3</sup>
NOTE: Volumes greater than 1000 L shall be shown in m <sup>3</sup>				
<b>MASS</b>				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
<b>TEMPERATURE (exact degrees)</b>				
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
<b>ILLUMINATION</b>				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m <sup>2</sup>	cd/m <sup>2</sup>
<b>FORCE and PRESSURE or STRESS</b>				
lbf	poundforce	4.45	newtons	N
lbf/in <sup>2</sup>	poundforce per square inch	6.89	kilopascals	kPa

## APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
<b>LENGTH</b>				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
<b>AREA</b>				
mm <sup>2</sup>	square millimeters	0.0016	square inches	in <sup>2</sup>
m <sup>2</sup>	square meters	10.764	square feet	ft <sup>2</sup>
m <sup>2</sup>	square meters	1.195	square yards	yd <sup>2</sup>
ha	hectares	2.47	acres	ac
km <sup>2</sup>	square kilometers	0.386	square miles	mi <sup>2</sup>
<b>VOLUME</b>				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m <sup>3</sup>	cubic meters	35.314	cubic feet	ft <sup>3</sup>
m <sup>3</sup>	cubic meters	1.307	cubic yards	yd <sup>3</sup>
<b>MASS</b>				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
<b>TEMPERATURE (exact degrees)</b>				
°C	Celsius	1.8C+32	Fahrenheit	°F
<b>ILLUMINATION</b>				
lx	lux	0.0929	foot-candles	fc
cd/m <sup>2</sup>	candela/m <sup>2</sup>	0.2919	foot-Lamberts	fl
<b>FORCE and PRESSURE or STRESS</b>				
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in <sup>2</sup>

\*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised March 2003)

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**LIST OF ABBREVIATIONS, ACRONYMS, AND SYMBOLS**

- A – Acceleration Coefficient per AASHTO Division I-A, Section 3
- A\* – Amplified peak acceleration
- A<sub>a</sub>, A<sub>v</sub> – Acceleration coefficients per ATC-3-06
- AASHTO – American Association of State Highway Transportation Officials
- A.D. – Latin, *Anno Domini*, current calendar epoch
- AOS – Apparent Opening Size
- a<sub>peak</sub> – Peak spectral acceleration, gravity (g)
- ARC – Associated Rockery Contractors
- ASTM – American Society of Testing and Materials
- ATC – Applied Technology Council
- B – Base rock width, m
- BMP – “Best Management Practices” as related to site erosion, sediment, and runoff control and reduction
- c, c’ – Total and effective soil cohesion values, respectively, in kPa. Although effective stresses are most typically used, the term “c” is commonly used throughout this report.
- ca. – Latin, *circa*, meaning “about” for dates that are approximately known
- CalTrans – State of California, Department of Transportation
- c<sub>F</sub>, c’<sub>F</sub> – Total and effective soil cohesion values in the foundation soil, in kPa
- CFLHD – Central Federal Lands Highway Division
- C<sub>IE</sub> – Seismic inertial coefficient
- CO – Contracting Officer
- d – Depth of soil to neglect when computing passive resisting force, m
- d<sub>peak</sub> – Peak spectral displacement, cm
- D – Embedment depth at toe of rockery, m
- D\* – Amplified peak displacement
- e – Moment eccentricity relative to center of base rock, m
- e<sub>s</sub> – Moment eccentricity relative to center of base rock due to seismic forces, m
- EFP – Equivalent fluid pressure applied by soil, kN/m<sup>3</sup>
- EPA – Effective peak acceleration, g
- EPV – Effective peak velocity, cm/s

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**LIST OF ABBREVIATIONS, ACRONYMS, AND SYMBOLS**

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- $F_A$  – Active earth pressure force acting on the back of the rockery
- $F_{A,H}$  – Horizontal component of active earth pressure force kN (per meter of rockery)
- $F_{A,V}$  – Vertical component of active earth pressure force, kN (per meter of rockery)
- $F_{AE}$  – Total static plus seismic thrust acting on rockery, kN (per meter of rockery)
- $\Delta F_{AE}$  – Incremental seismic thrust acting on rockery, kN (per meter of rockery)
- $F_H$  – Sum of static horizontal driving forces acting on rockery, kN (per meter of rockery)
- $F_{H,S}$  – Sum of static and seismic horizontal driving forces acting on rockery, kN (per meter of rockery)
- FHWA – Federal Highway Administration
- FLH – Federal Lands Highway
- Foundation Fill – Material conforming to Section 704.01 of the FHWA Standard Specifications (FP-03)
- $F_p$  – Passive resisting force on toe of rockery, kN (per meter of rockery)
- $F_s$  – Horizontal resultant force due to application of vertical surcharge load, kN (per meter of rockery)
- $F_\mu$  – Static resisting friction force of bottom of base rock, kN (per meter of rockery)
- $F_{\mu,S}$  – Static and seismic resisting friction force of bottom of base rock, kN (per meter of rockery)
- FS – Factor of safety
- FS<sub>BC</sub> – Factor of safety with respect to bearing capacity
- FS<sub>OT</sub> – Factor of safety with respect to external overturning
- FS<sub>OT\_INT</sub> – Factor of safety with respect to internal (inter-rock) overturning
- FS<sub>seismic</sub> – Factor of safety with respect to required rockery weight to limit rockery displacement to  $\Delta$  or less
- FS<sub>SL</sub> – Factor of safety with respect to base sliding
- GPR – Ground penetrating radar, a type of geophysical evaluation method
- H – Rockery height, m
- H/B – Height-to-base-width ratio
- HDPE – High density polyethylene, usually referring to a type of plastic pipe
- HEC – Hydraulic Engineering Circular

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**LIST OF ABBREVIATIONS, ACRONYMS, AND SYMBOLS**

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- HP – Ultraseismic Horizontal Profiling Method, a type of geophysical surface wave method
- i – Slope inclination, measured up from horizontal, per A. J. Hendron, Jr., methodology
- IR – Impulse Response, a type of geophysical surface wave method
- $K_A$  – Coefficient of active earth pressure
- $K_{AE}$  – Lateral earth coefficient for computation of static plus seismic thrust force
- $k_h$  – Horizontal seismic coefficient
- $K_p$  – Coefficient of passive earth pressure
- $k_v$  – Vertical seismic coefficient
- LTDS – Long term design strength of geogrid reinforcement
- MARV – Minimum Average Roll Values
- $M_o$  – External overturning moment about toe of rockery imposed by lateral earth pressure (horizontal component) and surcharge loads, kN – m (per meter of rockery)
- $M_{o,s}$  – External overturning moment due to static and seismic loads, kN – m (per meter of rockery)
- $M_{o\_int}$  – Internal (inter-rock) overturning moment about toe of intermediate rock imposed by lateral earth pressure (horizontal component) and surcharge loads, kN – m (per meter of rockery)
- $M_r$  – External resisting moment about toe of rockery imposed by rockery weight, lateral earth pressure (vertical component), and passive pressure, kN – m (per meter of rockery)
- $M_{r,s}$  – External resisting moment about toe of rockery imposed by rockery weight and vertical components of static and seismic lateral earth pressures kN – m (per meter of rockery)
- $M_{r\_int}$  – Internal resisting moment about toe of intermediate rock imposed by partial rockery weight, lateral earth pressure (vertical component), and passive pressure, kN – m (per meter of rockery)
- MSE – Mechanically stabilized earth or embankment
- OSHA – Occupational Safety and Health Administration
- PCR – Poorly constructed rockery as defined by A. J. Hendron, Jr.
- PGA – Peak ground acceleration, g
- PS – Parallel Seismic, a type of geophysical cross-hole logging method
- PVC – Polyvinyl chloride, a type of plastic pipe

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**LIST OF ABBREVIATIONS, ACRONYMS, AND SYMBOLS**

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- $q_{\max}$  – Maximum applied bearing pressure at toe of base rock due to moment eccentricity, kPa
- $q_{\max,s}$  – Maximum applied bearing pressure at toe of base rock due to moment eccentricity under seismic loading, kPa
- $q_s$  – Vertical surcharge pressure acting on the ground surface behind the rockery, kPa
- RSP – Rock slope protection, e.g., riprap
- RSS – Reinforced soil slope
- SAGE – Sanders & Associates Geostructural Engineering, Inc.
- SE – Sonic Echo, a type of geophysical surface wave method
- $T_a$  – Allowable tensile strength for geotextile soil reinforcement, equal to LTDS/FS
- UBC – Uniform Building Code
- U.S. – United States
- UV – Ultraviolet light, such as sunlight
- $v_{\text{peak}}$  – Peak spectral velocity, cm/sec
- $V^*$  – Amplified peak velocity
- $W$  – Total weight of rockery (unit width basis), kg
- WCR – Well-constructed rockery as defined by A. J. Hendron, Jr.
- $W_i$  – Weight of rockery component (unit width basis), kg
- $x_i$  – Moment arm from toe of rockery to centroid of mass for rockery component, m
- $z$  – Point of application of  $F_{AE}$ , measured vertically from base of rockery, m
- $\Delta$  – Tolerable rockery displacement for Richards and Elms analysis, in
- $\Delta F_{AE}$  – Incremental seismic thrust acting on rockery, kN (per meter of rockery)
- $\alpha$  – Angle, measured up from the horizontal, to the back cut or rear face of rockery, degrees. Positive angle is defined as face of back cut sloping up and away from the base of the rockery, starting from the bottom of the cut.
- $\alpha_A, \alpha_V, \alpha_D$  – Newmark and Hall amplification factors
- $\beta$  – Retained ground surface inclination (“backslope”), degrees. Positive angle if slope increases in height with increasing distance from the back of the rockery.
- $\delta$  – Interface friction angle between retained soil and back of rockery/crushed rock backfill zone, degrees

- $\phi, \phi'$  – Total and effective soil friction angle / angle of internal friction, respectively. Although effective stresses are most typically used, the term “ $\phi$ ” is commonly used throughout this report.
- $\phi_{CR}$  – Friction angle (effective or total) of crushed rock backfill
- $\phi_F$  – Friction angle (effective or total) of foundation soil and soil at toe of rockery
- $\gamma, \gamma_s$  – Density of retained soil,  $\text{kN/m}^3$
- $\gamma_{CR}$  – Density of crushed rock backfill (net density, including voids),  $\text{kN/m}^3$
- $\gamma_R$  – Density of rockery facing (net density, including voids),  $\text{kN/m}^3$
- $\gamma_F$  – Density of foundation soil,  $\text{kN/m}^3$
- $\mu$  – Friction factor for sliding
- $\psi$  – Back cut inclination, degrees



## EXECUTIVE SUMMARY

A rockery is a retaining or protection structure that consists of stacked rocks without mortar, concrete, or steel reinforcement. Although the rocks are stacked in an “interlocking” pattern, there are no mechanical connections made between the individual rocks. Rather, these structures rely on the weight, size, shape, and interface friction of the rock elements to provide overall stability. Other “structures,” such as riprap revetments, dry-stack rock walls, and grouted rock walls, use rocks as a primary construction material, although they are not considered rockeries.

For the purpose of this study, a rockery is defined as an engineered system of stacked angular rocks placed without mortar in an approximate “running bond” pattern. Rock dimensions are generally greater than 450 mm (18 in) and rock weights generally greater than 90 kg (200 lb). Stability of the system is achieved through the mass of the rocks and inter-rock friction. A rockery can be defined as either **protecting** (i.e., it only supports itself and armors the underlying slope) or **retaining** (i.e., it supports itself and resists lateral earth pressures).

Worldwide, unreinforced stone structures have been constructed for thousands of years. In the United States, rockeries still exist that were constructed in the late 1800s. However, it is doubtful these historic rockeries were “engineered” in the current sense of the term. In addition, rockeries were constructed along many Forest Highway and National Park roads by manual labor in the 1930s with various levels of quality. Many of these roads have subsequently become part of the national highway system, and some of the rockeries are still in use today. The Federal Lands Highway (FLH) Division of the Federal Highway Administration (FHWA) continues to find situations where new rockery construction would be advantageous or where repair or modification of existing historic rockeries is required. In 2005-06, rockeries were designed and constructed by the FLH for the Guanella Pass Road project in the Pike and Arapaho National Forests in Colorado. Commercially, rockeries have been constructed in the Pacific Northwest for the past four decades, and have seen increasing use in northern California and Nevada over the last 10 years.

The objectives of this study are to review existing analytical methods and construction techniques currently in use and to develop a unified framework for design and specification of rockeries in modern highway construction. The ultimate goal of the project is to provide designers, inspectors, and contractors with a basis for evaluating existing rockeries and specifying and constructing new rockeries.

The recommended design procedure is analogous to the design of gravity retaining walls. The rockery is designed to resist static and seismic earth pressures and lateral pressure surcharges that are defined using traditional retaining wall design concepts. Accordingly, the rockery is checked for an adequate factor of safety against sliding, overturning, and bearing capacity failure. Additionally, an allowable displacement method for seismic design proposed by Richards and Elms (1979) for gravity walls is provided for cost-effective rockery design.

Using the recommended design procedure, gravity rockeries can generally be constructed to between 2.4 m (8 ft) and 3.7 m (12 ft) tall in fill conditions without geogrid reinforcement, and up to 4.6 m (15 ft) as a facing for a mechanically stabilized earth (MSE) fill. Regardless, a maximum single-tier height of 4.6 m (15 ft) should be used in cut or fill conditions. Taller rockeries should be tiered.

For most civil works, the performance of a structure is directly related to the quality of construction. For a rockery, this concept is magnified several times by the fact that rockeries are constructed from irregularly shaped, naturally occurring materials. Therefore, this study provides recommendations for constructing and specifying rockeries consistent with the recommended design procedure. Standard rockery plans and specifications, compatible with the Federal FP-03 specification format, have been developed in conjunction with FLH. Field inspection checklists and forms are also included to aid in construction observation services.

## CHAPTER 1 – INTRODUCTION

### BACKGROUND

A rockery is a retaining or protection structure that consists of stacked rocks without mortar, concrete, or steel reinforcement. Although the rocks are stacked in an “interlocking” pattern, there are no mechanical connections made between the individual rocks. Rather, these structures rely on the weight, size, shape, and interface friction of the rock elements to provide overall stability.

Other “structures” exist that use rocks as a facing element, although they are not considered rockeries. Rock slope protection (RSP), also known as riprap, is similar in concept to rockeries, although it is usually placed over much shallower slopes and designed to resist hydraulic flow forces rather than lateral earth pressures. Grouted rock walls are also used in many landscape applications, although they derive their strength from the grout fill between the individual rocks and, therefore, are also not considered rockeries.

Similarly, dry-stacked, unmortared rock walls, although not considered rockeries as defined herein, have been constructed in the United Kingdom for over 100 years. Many are still in use along the highway system. These walls are generally constructed of 1 to 20 kg (2 to 45 lb), hand-placed stones stacked like bricks by masons.<sup>(Claxton, et al., 2005)</sup>

Worldwide, unreinforced stone structures have been constructed for thousands of years. Examples of early rock structures include Hadrian’s Wall in Scotland (ca. 122 A.D.) and the Incan city of Machu Picchu in Peru (ca. 1470 A.D.). Similarly, the city of Great Zimbabwe, in Zimbabwe, was constructed with unmortared, brick-sized stones ca. 1200 A.D.<sup>(Global Heritage Fund, unpublished Internet reference)</sup> Although not examples of rockeries per se, they are examples long-lasting uses of unmortared stone as a building material. In the United States, rockeries still exist that were constructed in the late 1800s. It is doubtful these historic rockeries were “engineered” in the current sense of the term.

Rockeries were also constructed along many Forest Highway and National Park roads by manual labor in the 1930s. Many of these roads have subsequently become part of the national highway

system. Little is known about the design of many of these rockeries, although it is suspected many were constructed with little or no engineering. Nevertheless, while some have failed, many are still in use today. These older rockeries, as well as more recent rockeries, may need to be evaluated by the FLH for conformance with current design standards as current and future transportation needs depend on their continued usage.

Although there is evidence the public sector was building rockeries in the 1930s, the private sector appears to have been somewhat slower to adopt commercial rockery construction. Rockeries have been constructed in the Pacific Northwest for the past four decades, and have seen increasing use in northern California and Nevada over the last 10 years. Because rockeries are a relatively inexpensive engineered retaining alternative with a natural aesthetic appeal, they continue to gain in popularity throughout the western United States.

The FLH continues to find situations where new rockery construction would be advantageous or where repair or modification of existing historic rockeries is required. However, conventional highway design standards are not available to confirm adequate internal stability or factors of safety, even where rockeries have performed adequately for decades.

Moreover, there is limited coverage of dry-stacked rockeries in engineering textbooks and literature. Although attempts have been made to develop guidelines for construction, these are typically local efforts and tend to be more procedural than analytical. For example, in 1992 the Associated Rockery Contractors (ARC) <sup>(1)</sup> developed construction guidelines based on local experiences in the Pacific Northwest. However, while the ARC guidelines provide general “rules-of-thumb” for use by contractors during construction, they do not provide a rational basis for design. As a result, individual designers are left to develop rockery design and construction standards based on their personal experience.

The lack of statewide or national design standards and construction guidelines has sometimes resulted in permitting and performance problems. Many municipal agencies are slow to accept rockery plans because they are unfamiliar with the design concept and/or do not have accepted guidelines to use as a basis for review. In the absence of standard guidelines, some municipal agencies require conservative designs and rockeries become prohibitively expensive. In other cases, poor rockery design and construction procedures, as well as a lack of understanding of the

nature of rockeries during the review process, have resulted in poor performance, including failures during or after construction. In many cases, the lack of quality assurance or construction oversight guidelines exacerbates problems related to poor design or construction practices. For example, the failed wall shown in Figure 1 generally has rocks of inadequate size and which are poorly stacked. In addition, a backdrain does not appear to have been installed.



**Figure 1. Photograph. Failure of non-engineered rockery in El Dorado Hills, California, 2004.**

## **PURPOSE OF STUDY**

The objectives of this study are to review existing analytical methods and construction techniques currently in use and to develop a unified framework for design and specification of rockeries in modern highway construction. The ultimate goal of the project is to provide designers, inspectors, and contractors with a basis for evaluating existing rockeries and specifying and constructing new rockeries.

Based on review of available literature and the evaluation of several existing design procedures, the authors have developed a unified analysis and design framework that can be used in modern highway engineering. The framework is rational and follows recognized engineering principles derived from analysis procedures for gravity retaining walls.

Because rockeries are field assembled with natural and variable materials, careful field observation is essential to confirm that the as-built structures are consistent with the intent of the design. Therefore, guidelines have also been developed to provide designers, inspectors, and contractors with a basis for specifying, evaluating, and constructing rockeries.

## **ROCKERY DEFINITIONS**

Rockeries can be generally defined as rough rocks stacked in an “interlocking” pattern without concrete, mortar, or steel reinforcement. Neither mechanical nor physical connections are made between the individual rocks; “interlocking” is accomplished through proper rock layout, rock weight, and frictional interaction. Various terms have been used to describe rockeries, including “rock breast walls,” “rockery walls,” “dry-stack walls,” “stone walls,” and “rock walls.” The terms used to describe rockeries often reflect the intended use, and, in some cases, preconceptions regarding rockeries.

There is some disagreement within the engineering community as to whether rockeries should be considered earth retaining structures. The City of Seattle, Washington, specifically states rockeries should not be used for earth retention purposes,<sup>(2)</sup> but can be used as an aesthetic treatment for an otherwise stable slope or to provide erosion protection (slope armor). The City of Seattle rockery guidelines appear to have been adopted by Mason County, Washington, and the City of Sparks, Nevada, both of which do not allow the use of rockeries for earth retention.<sup>(unpublished Internet references)</sup> On the other hand, municipalities such as the City of Brier, Washington, allow the use of rockeries as retaining structures, although they require engineered design for any rockeries over 0.9 m (3 ft) tall.<sup>(3)</sup>

Some researchers have acknowledged that rockeries can serve as retaining structures and have developed equations especially designed to evaluate the stability of rockeries retaining both native soils and fills.<sup>(4)(Dale C. Hemphill, unpublished)</sup> Others practitioners have taken the middle ground, conceding that while rockeries are best implemented as decorative architectural features or as slope protection for stable slopes, there is an increasing tendency to use rockeries for stabilization of oversteepened cut slopes or for retention of fill slopes.<sup>(5)</sup> Despite the different definitions and attitudes toward rockeries, they have been successfully designed and constructed

to heights up to 7.6 m (25 ft) in the Pacific Northwest and northern California over the last decade.

For the purpose of this study, a rockery is defined as an engineered system of stacked angular rocks placed without mortar in an approximate “running bond” pattern. Rock dimensions are generally greater than 450 mm (18 in) and rock weights generally greater than 90 kg (200 lb). Stability of the system is achieved through the mass of the rocks and inter-rock friction. A rockery can further be defined as either **protecting** (i.e., it only supports itself and armors the underlying slope) or **retaining** (i.e., it supports itself and resists lateral earth pressures). The average thickness of protecting rockeries is generally less than retaining rockeries.

Rockeries are typically specified by their:

- Height (H).
- Base Width (B).
- Face batter angle, typically between 4V:1H and 6V:1H.
- Individual rock weight and/or size.

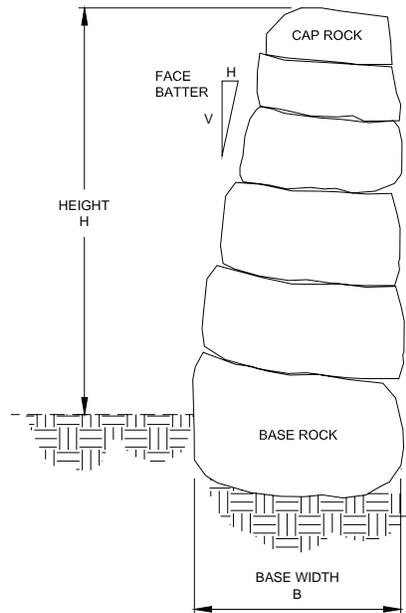
A diagram showing the rockery parameters defined above is presented as Figure 2.

Small landscaping walls comprised of cobble-sized rounded rocks can be often be found retaining 0.3 to 0.6 m (1 to 2 ft) of soil, often in garden or landscaping applications. These walls are typically not engineered and are often constructed mainly for aesthetic purposes; therefore, they are not considered rockeries for the purposes of this study.

### **Man Rocks**

For many years, it was common to describe the facing rocks used for rockery construction in terms of “man rocks.” While the rockery design practice generally no longer uses this term for reasons described later in this report, many references and specifications still exist that refer to “man rocks”. In general, a “man rock” is defined by the number of men it would take to manually move a rock into place using steel pry bars,<sup>(5)</sup> and, therefore, the term qualitatively

describes the weight of a rock. For example, a “two-man rock” is defined as the largest rock two men could move into place; a “four-man rock” would require four men, and so on. Man rock sizes are generally capped at “six-man rocks.”



**Figure 2. Graphic. Diagram showing definitions of height (H), base width (B), face batter, and relative rock sizes.**

Because this unit of measure is subjective, definitions of man rock sizes often vary between municipalities, as shown in Table 1. Therefore, the classification of facing rocks into man rock groups in the field requires knowledge of the definitions being used. For example, Table 1 shows that while the maximum weight for a six-man rock is generally considered about 3,600 kg (8,000 lb), at least one classification system considers the typical weight to be 1,090 kg (2,400 lb). Similarly, while some of the municipalities use a range for typical man rock sizes, the City of Seattle uses a single number for 5- to 6-man rocks that appears to be an average size.

**Table 1. Comparison of man rock definitions.**

Rock Size (Man Rocks)	Source									
	ARC <sup>(1)</sup>		City of Seattle, WA <sup>(2)</sup>		City of Brier, WA <sup>(3)</sup>		Gifford & Kirkland <sup>(5)</sup>		FHWA <sup>(6)</sup>	
	Weight, kg	Average Dimension, m	Weight, kg	Volume, m <sup>3</sup>	Weight, kg	Diameter, m	Weight, kg	Surface Area/ Man Rock, m <sup>2</sup>	Weight, kg	Average Dimension, m
1	23 – 91	0.30 – 0.46	26 – 95	0.011 – 0.036	23 – 91	0.30 – 0.46	182	0.33	20 – 90	0.300 – 0.450
2	90 – 318	0.46 – 0.71	120 – 263	0.045 – 0.10	90 – 318	0.46 – 0.71	363	0.48	90 – 300	0.450 – 0.710
3	318 – 908	0.71– 0.91	345 – 831	0.13 – 0.31	318 – 908	0.71– 0.91	545	0.61	300 – 900	0.710 – 0.900
4	908 – 1,816	0.91– 1.22	1,362 – 1,816	0.52 – 0.67	908 – 1,816	0.91– 1.22	726	0.63	900 – 1,800	0.900 – 1.200
5	1,816 – 2,724	1.22– 1.37	2,270	0.86	1,816 – 2,724	1.22– 1.37	908	0.65	1,800 – 2,700	1.200 – 1.350
6	2,724 – 3,632	1.37– 1.52	3,178	1.20	2,724 – 3,632	1.37– 1.52	1,090	0.67	2,700 – 3,600	1.350 – 1.500



CHAPTER 2 – SURVEY OF EXISTING DESIGN METHODS

**HISTORIC ROCKERIES**

The use of dry stacked rocks to build walls is one of the earliest construction technologies used by man. It is doubtful that any early examples of rock construction were “designed,” other than by trial and error, to produce a structure that was useful for a period of time acceptable to the builder. As a result, rockeries that survive today are only those that, by chance or through the experience of the builder, have certain characteristics that have allowed them to stand the test of time. These characteristics might include the use of large rock-mass-to-rockery-height ratios, tabular rock shapes, and well-drained retained materials.

One of the earliest surviving examples of a dry stacked wall is known as Hadrian’s Wall, which is shown in Figure 3. Built in 122 A.D. by the Roman Emperor Hadrian, the wall served as a fortification against invasion by “barbarians.” The wall is 6 m (20 ft) tall and 3 m (10 ft) wide, and stretches 117.5 km (73 mi) across the southern border of what is now Scotland.<sup>(About Scotland, unpublished Internet reference)</sup> The wall generally does not retain soil or protect a soil slope, and, therefore, is not directly analogous to rockery design or performance. Nevertheless, it is an example of the durability and permanence of historic mortarless rock construction.



**Figure 3. Photograph. Hadrian’s Wall, Scotland.**

Similarly, dry-stacked stone construction was used for the construction of the city of Great Zimbabwe in Zimbabwe ca. 1200 A.D. The city, which is thought to be the ruling city of the Queen of Sheba, has many surviving structures constructed of brick-sized, unmortared blocks.

The largest structure at the site, the Great Enclosure, is the largest ancient structure in Africa south of the pyramids in the Sahara Desert. The building has a circumference of about 255 m (840 ft), 10-m-high (33-ft-high) walls, and a maximum wall thickness of about 5 m (17 ft). It is estimated that about one million blocks were used to construct the Great Enclosure.<sup>(Global Heritage Fund, unpublished Internet reference)</sup>

Another example of early rock construction is the dry stacked rock walls and structures constructed by the Incas at Machu Picchu in Peru, as shown in Figure 4. Constructed between 1460 and 1470 A.D., the construction consists of tightly fitting blocks stacked without mortar. It is reported that the joints were tool finished, and although some blocks have as many as 30 “corners,” the joints are so tight that a knife blade cannot be forced between the rocks.<sup>(University of Minnesota, Mankato, unpublished Internet reference)</sup> Many of these rock structures act as retaining walls. The structures were constructed using local rock materials and many blend into adjacent rock outcrops or underlying rock cliffs. In several cases, natural water features, such as springs, flow seamlessly from native rock through rockery-supported flumes.



**Figure 4. Photograph. Rockery construction at Machu Picchu, Peru.**

Historically, dry stacked rock walls have also been constructed in rural portions of the United States. As early as the 1700s, short rock walls were used to delineate property lines in the northeastern United States. During the nineteenth century, rockeries were also constructed in the western states, often by immigrant labor. Chinese laborers reportedly constructed numerous rockeries in California during the later stages of the gold rush in the late 1800s. Many of these

walls remain, particularly along railways and historic canals crisscrossing the Sierra Nevada mountain range. One example of this type of rockery is shown in Figure 5, which supported a flume for hydraulic mining outside of La Grange, California. The rockery is approximately 9.1 m (30 ft) tall and appears to have an overall width of less than 3 m (10 ft). While small areas at the flanks of the rockery have begun to deteriorate due to erosion, the majority of the rockery is still in good condition.



**Figure 5. Photograph. Approximately 9.1-m-tall (30-ft-tall) rockery outside La Grange, California, dating from the late 1800s.**

### **MODERN ROCKERY DESIGN**

The design of rockeries appears to have been an art rather than a science, with most rockery construction based on a trial-and-error process, until sometime in the 1960s or 1970s. About this time, suitable equipment and materials became available in the northwestern portion of the United States to produce rockeries economically and dependably. These rockeries began to routinely feature tabular, tightly placed stones of adequate size to resist destabilizing forces. In addition, wall designs began to specify that crushed rock infill material be used behind the rock facing to provide high-strength backfill material and adequate drainage. These construction materials continue to be specified for current-day rockeries and are consistently observed when reviewing historical rockeries that have lasted for a hundred years or more.

Another innovation in rockery construction was the use of hydraulic construction equipment with the capability to grasp individual rocks and set them accurately within the rockery matrix, which

helped speed construction. Furthermore, blasted and crushed rock with aesthetically pleasing characteristics has increased in commercial availability. When combined, these factors have all contributed greatly to the economy, popularity, and dependability of rockeries used for commercial construction in recent years.

However, while certain rockery components appear to be common to most modern rockeries, dimensioning, specifying, and engineering rockeries is an evolving process that can vary significantly between designers. The remainder of this chapter presents design and specification similarities and differences found between the different designers and governmental entities studied.

### **Federal Government**

The U.S. Department of Agriculture, Bureau of Public Roads showed typical rockery sections on their plans for the Clifton-Springerville Road in the Apache Nation Forest in Arizona,<sup>(unpublished, untitled drawing)</sup> which demonstrates rockeries were being constructed by some departments within the Federal government as early as 1918. On this project, “dry rubble walls” were apparently used as headwalls at pipe outfalls. The headwalls, which appear to vary between 990 and 1,600 mm (39 and 63 in) in height, have minimum base widths of 530 to 760 mm (21 to 30 in) and top widths of 380 to 460 mm (15 to 18 in). Details are also provided for a rockery that apparently retains a fill slope with an inclination of 1V:1.5H. The detail for the rockery simply specifies a design height “H,” a base width of “H/2,” and a minimum dimension at the top of the rockery of 600 mm (2 ft).

More recently, rockeries were designed and constructed by the FHWA for the Guanella Pass Road project in the Pike and Arapaho National Forests in Colorado.<sup>(6,7)</sup> The rockeries for Guanella Pass were specified with a maximum height of 3.5 m (11.5 ft). Tiered rockeries were limited to 3 m (10 ft) per tier. Specific rockery dimensions were described on the plans using the following design parameters, where H is the design height as measured from the bottom of the base rock (rock that bears on the soil subgrade) to the top of the cap rock (uppermost rock in the rockery, usually flush with the retained surface):

- Base width:  $0.5H$ , but not less than 1.2 m (4 ft). Rocks in each course are specified by a man rock designation, and a table was provided that presents different values for each course for different rockery heights. The maximum “average dimension” for a six-man rock is listed as 1.5 m (5 ft).
- Embedment: Minimum of 500 mm (19.5 in).
- Base surface: The base of the rockery should be sloped back into the slope such that the rocks are not inclined to fall out of the face of the rockery.
- Top width:  $0.3H$  or 500 mm (19.5 in), whichever is larger.
- Allowable steepness/face batter: 4V:1H or flatter.
- Soil cut: 6V:1H or flatter.
- Structural rocks shall not be loose or able to be moved by a pry bar after rockery completion.
- Non-structural chinking rocks shall not be loose or able to be removed by hand after construction is complete. Chinking rocks consist of small rocks used to fill voids and distribute loads between larger facing rocks.
- The longest rock dimension should be oriented perpendicular to face of rockery.
- The longest dimension shall not exceed three times the shortest dimension.
- Structural rocks shall bear on at least two rocks below, and shall be placed so that no continuous joints existing in either vertical or horizontal planes.

Typical wall details and schedules for the Guanella Pass bid documents are presented as Figures 6 and 7.<sup>(6)</sup> The schedule provides man-rock designations for each rockery lift with regard to overall rockery height. Man-rock sizes are also provided. An example of a completed rockery for the Guanella Pass project is presented as Figure 8.

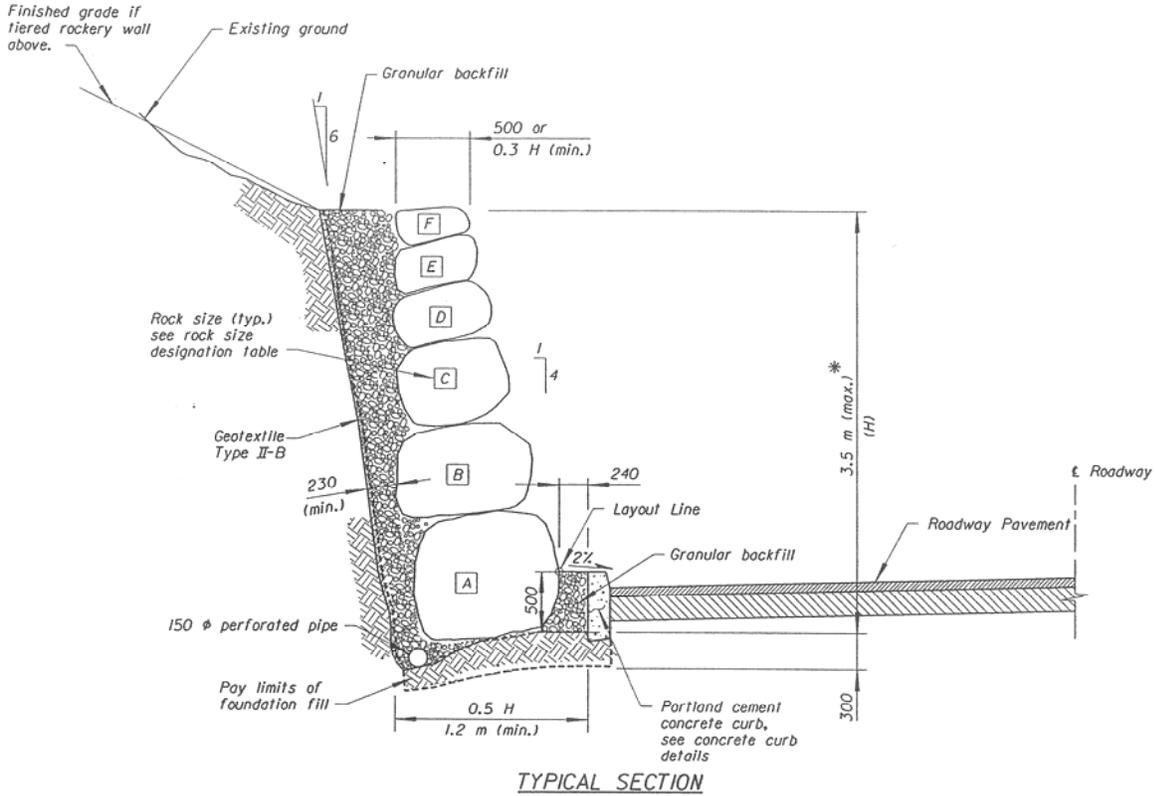


Figure 6. Graphic. Typical rockery section from Guanella Pass bid documents.

**ROCK SIZE DESIGNATION**

Wall Height (H)	A	B	C	D	E	F
≤ 1.0 m	3	2	-	-	-	-
≤ 2.0 m	4	3	2	-	-	-
≤ 3.0 m	4	4	3	2	-	-
≤ 3.5 m	5	4	3	3	2	2

Number in table is equal to rock size.

**ROCK SIZE**

Rock Size □ Person	Rock Weight (kilogram)	Average Dimension (meter)
1	20 to 90	0.300 to 0.450
2	90 to 300	0.450 to 0.710
3	300 to 900	0.710 to 0.900
4	900 to 1,800	0.900 to 1.200
5	1,800 to 2,700	1.200 to 1.350
6	2,700 to 3,600	1.350 to 1.500

Figure 7. Graphic. Typical rockery schedule from Guanella Pass bid documents.



**Figure 8. Photograph. Completed rockery for Guanella Pass Road, Colorado, 2005.**

Rockery design recommendations were also identified in a *Retaining Wall Design Guide* published by United States Department of Agriculture, Forest Service, in 1994 (Report No. FHWA-FLP-94-006). The design guide references reports and figures by Gifford & Kirkland<sup>(5)</sup> and ARC<sup>(1)</sup> as the primary design aids. Rockeries are described as providing limited resistance to lateral earth pressures, and to be used primarily for erosion protection. The maximum recommended height for rockeries is 4.6 m (15 ft). For rockeries retaining fill, it is recommended that the fill consist of Mechanically Stabilized Earth (MSE) for heights over 2.4 m (8 ft).

### **Pacific Northwest**

Rockeries have been constructed in the Pacific Northwest for at least the last four decades, as evidenced by Gifford and Kirkland,<sup>(5)</sup> who describe a robust rockery design practice in 1978. Because of the long history of construction in the area, many of the local municipalities have developed detailed guidelines for construction specific to rockeries. These guidelines generally include prescriptive methods for design and construction of rockeries less than 1.8 m (6 ft) tall. Engineered design is generally required for rockeries that exceed a height of 1.8 m (6 ft), and in

some cases, less. Prescriptive design parameters were obtained for the City of Seattle, Washington; City of Brier, Washington; and Mason County, Washington. The prescriptive design parameters for these municipalities are summarized later in this chapter. Portions of the guidelines have also been adopted by the City of Sparks, Nevada, and the City of Knoxville, Tennessee.

The rockery construction profession also developed guidelines for rockery construction. Based on experiences in the Pacific Northwest, these guidelines, published by the ARC,<sup>(1)</sup> provide guidance regarding bidding practices, rockery layout, drainage, and the role of the geotechnical engineer. The guidelines also state that rockeries are generally used as erosion control structures; where they retain lateral earth pressures, they should be designed by a geotechnical engineer.

One significant innovation to come out of the Pacific Northwest has been the use of a zone of crushed rock behind the rockery. In addition to providing a dedicated drainage layer, the crushed rock provides a zone of material with a high natural friction angle, and, thus, a relatively low active earth pressure coefficient,  $K_A$ . When the crushed rock comprises a significant percentage of the backfilled zone, the high strength of the crushed rocks helps reduce the lateral earth pressure on the rockery and helps stabilize the rockery system.

Because rockeries have been designed and constructed for several decades in the Pacific Northwest, many of the published references located for this study originate from this area. The information obtained from these studies follows.

### ***Gifford and Kirkland***

In their paper *Uses and Abuses of Rockeries*, Gifford and Kirkland<sup>(5)</sup> provide generally prescriptive design standards for rockeries, but with much more detail and for taller rockery heights than the municipal recommendations discussed previously. When Gifford and Kirkland prepared their report in 1978, rockeries were seeing increasing use as retaining structures. However, the paper suggests Gifford and Kirkland believed rockeries were an inferior retaining structure relative to other engineered structures, and that they believed the best use of rockeries was to protect otherwise stable slopes. Specifically, Gifford and Kirkland commented that:

- Rockeries are similar to retaining walls, but that “wherever a rockery is built, a reinforced concrete wall would be a better substitute.” The paper does not explain how or why a reinforced concrete wall would be considered “better.”
- Rockeries have relatively low tolerance for movements because the rocks are not structurally tied together. If settlement occurs, the rocks may lose contact with each other, leading to bulging or toppling failure.
- Rockeries are generally best for aesthetic improvement or erosion control uses. A rockery should not be used unless the slope it faces is stable without support from the rockery.
- Rockeries are generally not designed, but rather “just built” using prescriptive standards. Often, rockeries are built by contractors without plans. Design, where performed, usually involves standards that have evolved over time using “trial-and-error” methods—designs that perform well are reused and those that fail are revised.

Gifford and Kirkland reference an unpublished report prepared by A. J. Hendron, Jr. in 1960. The original report by Hendron could not be located. Per Gifford and Kirkland, Hendron determined the stability of rockeries is controlled by overturning, rather than inter-rock or global sliding, and that the rockery stability is highly dependent on the rockery face inclination. The research included development of a design chart, as shown in Figure 9, which presents “critical” height-to-base-width (H/B) ratios for prescribed soil conditions based on overturning stability. The nature of the term “critical” is not discussed in the Gifford and Kirkland paper, and because the original work by Hendron could not be located, it is not clear if the “critical” value represents a factor of safety of 1.0 or an “allowable” value. For the purposes of this study, it has been assumed “critical” implies failure is imminent, yielding a factor of safety of 1.0.

The chart provides design curves for both well-constructed rockeries (WCR) and poorly constructed rockeries (PCR). This distinction was made to account for assumptions in the analyses regarding the point about which rotation is assumed to occur. The point of rotation is assumed to be at the midpoint of the base of the rockery for PCRs, whereas it is near the face of

the rockery for WCRs. Gifford and Kirkland recommended that because this subjective measure of workmanship was difficult to evaluate, the curves for PCRs should be used for design.

The chart is used by determining the wall height ( $H$ ), internal soil friction angle ( $\phi$ ), and slope inclination ( $i$ ). Because the chart was constructed assuming a value of  $17.6 \text{ kN/m}^3$  (110 pcf) for the soil unit weight ( $\gamma$ ), the chart should be regenerated, using the equations shown on Figure 9, for larger values of  $\gamma$  as a larger lateral earth pressure will be imposed on the rockery. The values of  $i$  and  $\phi$  are then used, along with the PCR curve, to determine the critical  $H/B$  ratio. This ratio is in turn used to determine the minimum average base rock width ( $B$ ).

For example, given a rockery height of 3 m (10 ft), a slope inclination of 80 degrees, and a soil friction angle of 30 degrees, the critical  $H/B$  ratio read from the Hendron chart is about 4. Therefore, the critical base width required to satisfy minimum overturning stability requirements is 0.75 m (2.5 ft). If the friction angle is increased to 35 degrees,  $H/B$  increases to about 5.5 and  $B$  decreases to about 0.5 m (1.6 ft). In practice, larger values would likely be used for design because these are “critical” values and likely represent a factor of safety of 1.0. Alternatively, if  $B$  is known, the chart can be used to determine the critical height, above which the factor of safety is less than 1.0. For example, given a base width of 1.2 m (4 ft), a slope inclination of 85 degrees, and a friction angle of 30 degrees, the  $H/B$  ratio is 2 and the maximum (critical) rockery height is about 2.4m (8 ft).

The chart that has been provided (as originally published in Gifford and Kirkland) is valid for the values of  $\phi$  between 30 and 35 degrees, and a value of  $\gamma$  of about  $17 \text{ kN/m}^3$  (110 pcf). For other values, the user should generate new curves using the equations shown in Figure 9.

CRITICAL H/B RATIOS

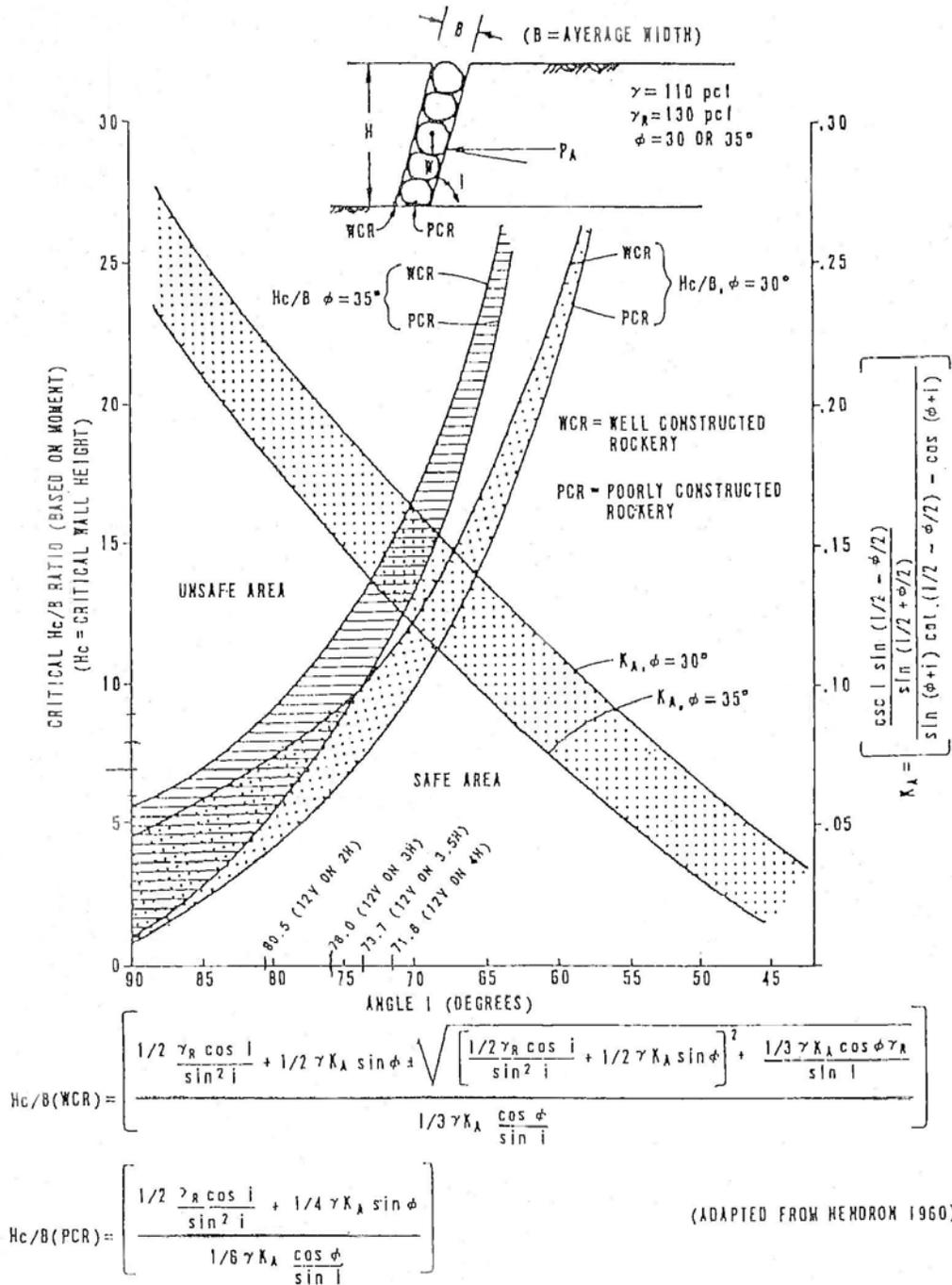


Figure 9. Chart. Critical height-to-base-width (H/B) ratios for poorly constructed rockeries (PCRs) and well-constructed rockeries (WCRs), after Hendron.

Gifford and Kirkland indicated that the maximum rockery base thickness (B) they have observed is about 1.2 m (4 ft), with an average rockery thickness of about 0.9 m (3 ft). In addition, most rockeries they observed had face batters between 12V:1H to 12V:4H (“i” values between 85 and 72 degrees, respectively). For a reasonable range of retained soil friction angle ( $\phi$ ) and using the Hendron chart (PCR curves, Figure 9), Gifford and Kirkland concluded the theoretical maximum rockery height is between 3.7 and 11 m (12 and 36 ft).

However, Gifford and Kirkland also noted that most failures observed through their practice occurred to rockeries constructed to heights exceeding 4.6 m (15 ft), and, therefore, 4.6 m (15 ft) is a reasonable maximum single-tier design height. Furthermore, where rockery failures have been observed, they indicated that it generally required more rocks to repair a failed rockery than were originally required to construct it, suggesting that failures are highly dependent on the original density (packing) of the rocks and that a denser packing results in a more stable rockery.

An interesting statement in the Gifford and Kirkland report is that they contend most owners understand there is a “lower standard of engineering” for rockeries than for other engineered retaining structures, such as cast-in-place or concrete masonry unit walls, and that rockery failure is a possibility. However, this potential for failure is outweighed by construction cost savings. Where failures have been observed, they listed the six most common causes (additional details were not provided by Gifford and Kirkland):

- Little or no drainage was provided.
- The backfill was of poor quality or poorly placed and compacted.
- Rockery face was constructed too steep or too high.
- The rockery was constructed over a poor foundation.
- The rockery was constructed of unsound rock.
- The overall workmanship of the constructed product was poor.

They recommended periodic maintenance and inspection of rockeries so that remedial measures could be taken if the potential for failure is observed.

***Hemphill Consulting Engineers***

In 1990, Hemphill Consulting Engineers (Hemphill) of Bellevue, Washington, presented analytical methods for rockery design in a report titled *The Engineering Method for Rockery Design*.<sup>(unpublished, 1990)</sup> Hemphill noted that rockeries can function adequately as retaining walls if they are properly designed. However, many contractors believe they must “design” rockeries in the field through proper placement and field fitting because the plans and specifications provided are “so ambiguous as to be nearly useless.” Hemphill cites the widespread use of the man rock designation as one problematic feature because the dimensions of the rocks are poorly defined and often vary between municipalities. However, at the time of his report, the use of man rock designations appears to have been standard of practice in the area.

For the Hemphill procedure, the active earth pressure on the back of the rockery is computed in 0.3-m (1-ft) increments, beginning at the top of the rockery and ending at the base. For each increment, a rock width is computed that satisfies factor of safety requirements for both inter-rock sliding and overturning. The computations account for the weight of the trial rock at the increment being checked, as well as the weights of all overlying rocks. The overturning moment due to the lateral earth pressure is then computed about the toe of the rock at each level. The width of the rock is varied until the desired factor of safety with respect to both sliding and overturning is computed. Below finish grade, the minimum rock width is computed by checking for adequate factor of safety to resist sliding using both friction on the bottom of the base rock and passive pressure against the face of the rockery. A sample calculation, excerpted from the Hemphill report, is shown in Figure 10.

A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P
ROCKERY HEIGHT ABOVE ROCK INTERSECTION THAT COULD SLIDE OR OVERTURN	ACTIVE SOIL FORCE ABOVE ROCK INTERSECTION CAUSING SLIDE OR OVERTURN	REQUIRED FRICTION FORCE AT ROCK INTERSECTION RESISTING SLIDING BETWEEN ROCKS	THEORETICAL TOTAL WEIGHT ABOVE ROCK INTERSECTION TO CREATE SLIDING TO RESIST	ACTUAL TOTAL WEIGHT OF ROCK ABOVE INTERSECTION INCLUDING MIN ALLOWABLE THICKNESS AT TOP OF ROCKERY	PREVIOUS WEIGHT OF ROCKERY REQUIRED FOR PREVIOUS SECTION	ACTUAL WEIGHT OF ROCKERY TO ACHIEVE FRICTION OR MIN THICKNESS	REQUIRED THICKNESS OF ROCKERY TO ACHIEVE INCREMENTAL WEIGHT OR MIN THICKNESS	MOMENT OF OVERTURN CAUSED BY LATERAL SOIL PRESSURE	LEVER DISTANCE FROM FACE OF ROCK TO CENTROID OF UPPER ROCKERY	RESISTING MOMENT TO OVERTURN FROM WEIGHT OF ROCKERY AT TOE	SAFETY FACTOR AGAINST OVERTURNING ASSUMING TOE AT FACE OF ROCKERY	REQUIRED THICKNESS OF BOTTOM ROCK TO RESIST SLIDING ON SOIL	RESISTING FRICTION FORCE ON SOIL AT BOTTOM OF ROCKERY TO RESIST SLIDING	PASSIVE SOIL PRESSURE RESISTING SLIDING AT BOTTOM OF ROCKERY	SAFETY FACTOR AGAINST SLIDING OF BOTTOM ROCK ON SOIL
H (ft)	$F_{a} = S \cdot \rho \cdot H^2$ (lb)	$F_{r} = F_{a} \times SF$ (lb)	$W_{t} = \rho \cdot H \times D \times D$ (lb)	$W_{t} = W_{t} + W_{p}$ (lb)	$W_{p}$ (lb)	$W_{f} = W_{t} - W_{p}$ (lb)	$T_{r} = W_{f} / D \times SF$ (ft)	$M = F_{a} \times H / 3$ (ft-lb)	LA (ft)	$M = W_{f} \times LA$ (ft-lb)	$SF = M / M_{o}$	$T_{ra}$ (ft)	$F_{r} = F_{a} \times f_{s}$ (lb)	$P_{p} = 3 \times P_{p}$ (lb)	$F_{s} = F_{a} / F_{s}$ (ft)
1	15	26	46	198	0	198	1.5	5	0.8	149	29.7				
2	60	102	185	396	198	198	1.5	40	0.8	297	7.4				
3	135	230	417	594	396	198	1.5	135	0.8	446	3.3				
4	240	408	742	792	594	198	1.5	320	0.8	594	1.9				
5	375	638	1,159	1,159	792	367	2.8	625	1.8	2,119	3.4				
6	540	918	1,669	1,669	1,159	510	3.9	1,000	2.6	4,339	4.0				
7	735	1,250	2,272	2,272	1,669	603	4.6	1,775	3.0	6,908	4.0				
8	960	1,632	2,967	2,967	2,272	695	5.3	2,560	3.5	10,336	4.0				
9	1,215	2,066	3,755	3,755	2,967	788	6.0	3,645	3.9	16,773	4.1				
10	1,500	2,550	4,656	4,656	3,755	881	6.7	5,000	4.4	20,351	4.1				
ADDITIONAL ROCKERY IS FOR FROST PROTECTION & RESISTANCE TO SLIDING FAILURE															
11	1,815	3,066	5,610	5,610	4,656	974	7.4	7.4				23.3	2,244	100	1.3
12	2,160	3,672	6,676	6,676	5,610	1,066	8.1	8.1				27.0	2,671	400	1.4
13	2,535	4,310	7,835	7,835	6,676	1,159	8.8	8.8				31.0	3,134	900	1.6
14	2,940	4,998	9,087	9,087	7,835	1,252	9.5	9.5				35.3	3,685	1,600	1.8

Figure 10. Graphic. Sample calculation from the Hemphill report demonstrating application of the Hemphill design method.

Hemphill suggests the factors of safety against overturning and sliding used in the analyses be selected based on the degree of control and level of observation the design engineer will have during construction:

- $FS_{OT} = FS_{SL} = 1.5$  for full-time inspection and well-defined properties.
- $FS_{OT} = FS_{SL} = 2.0$  for periodic inspections.
- $FS_{OT} = FS_{SL} = 2.5$  for poor inspection or poorly defined properties.

### *Seismic Performance During the Nisqually Earthquake*

One topic noticeably absent from both the Gifford and Kirkland and Hemphill papers is seismic design. However, the Pacific Northwest is a seismically active area, and it is expected that rockeries constructed in the region will likely be subjected to seismically-induced ground shaking during their useful life. The Nisqually earthquake, which occurred on February 28, 2001 near Olympia, Washington, provided a case history to evaluate rockery performance during a seismic event.

The earthquake had a moment magnitude ( $M_w$ ) of 6.8 and was centered approximately 17.7 km (11 mi) northeast of Olympia, Washington. Recorded peak ground accelerations (PGAs) ranged from 0.1 to 0.2 times gravity ( $g$ ).<sup>(Pacific Northwest Seismograph Network, unpublished)</sup> On March 8, 2001, a reconnaissance of five sites with completed rockeries was performed to document their properties and conditions. This reconnaissance was performed by Steven H. Sanders of Sanders & Associates Geostructural Engineering (SAGE) and Gordie McCarty of Parsons Brothers Rockery Retaining Walls. A summary of their observations is presented as Table 2. Photographs from each of the five sites visited are presented as Figures 11 through 15.

No rockery failures were observed during the reconnaissance, and the overall conclusion was that the rockeries performed well under the low to moderate PGAs experienced during the earthquake. The rockeries observed provided a fairly wide spectrum of rockery heights, base widths, and quality of construction.

**Table 2. Summary of rockery observations following the 2001 Nisqually earthquake.**

Observed Properties	Site 1	Site 2	Site 3	Site 4	Site 5
Distance from Epicenter, km	32.2	32.2	17.7	14.5	25.8
Estimated PGA, g	0.15	0.1	0.15	0.12	0.1
Estimated Construction Completion, year	2001 (ongoing)	1980s to 1995 (3 walls)	N/A	N/A	N/A
Rockery Height, m	3	2.7 to 7.3 (tiered portions)	1.2	2.1 – 2.4	7.6
Rockery Length, m	45.7	N/A	N/A	N/A	N/A
Base Rock Width, m	1.2 – 1.5	1.2 – 1.5	0.6 – 0.9	0.6 – 0.9	1.5 – 2
Base Width, % of H	0.4H – 0.5H	0.4H – 0.7H	0.6H – 0.75H	0.25H – 0.4H	0.25H
Cap Rock Width, m	0.3 – 0.9	0.3 – 0.6	N/A	0.6 – 0.9	1.2 – 1.5
Backfill	Native silty sand with 50- to 200-mm crushed rock	Crushed rock	Soil	Crushed rock	Crushed rock (50 to 100 mm)
Foundation	Silty sand	N/A	N/A	N/A	Silty sand
Apparent Construction Quality	Fair	Poor to fair	Fair	Good	Fair; lots of voids
Nearby Damage (unrelated to rockery)	None	None	Cracks to Capital Building dome	None	None
Observed Rockery Damage	None	(a)	(b)	(c)	None

Notes: <sup>(a)</sup>Some bulging noted from foundation movement, does not appear related to the earthquake.

<sup>(b)</sup>Some static movement (shifting) appears to have occurred in the past.

<sup>(c)</sup>Horizontal separation between rocks at end of rockery; may be caused by construction or earthquake.

N/A = Data not recorded, not applicable, or unknown.



**Figure 11. Photograph. Single-tier, 3-m (10-ft) rockery, Military Road at Enchanted Parkway, Federal Way, Washington (Site 1).**



**Figure 12. Photograph. Two-tier, 7.3-m (24-ft) rockery, 15th Avenue at 12th Street, Puyallup, Washington (Site 2), with guy wire anchored at base of rockery.**



**Figure 13. Photograph. Single-tier, 1.2-m (4-ft) rockery near state capital building, Olympia, Washington (Site 3).**



**Figure 14. Photograph. Single-tier, 2.4-m (8-ft) rockery, Nisqually National Wildlife Refuge, Washington (Site 4).**



**Figure 15. Photograph. Single-tier, 7.6-m (25-ft) rockery in Tacoma, Washington (Site 5).**

### **Gray & Sotir**

One objective of this study was to identify scholarly references, juried papers, or textbooks regarding rockery design and construction. Unfortunately, very few of these types of documents were uncovered. With the exception of Gifford and Kirkland<sup>(5)</sup> and the Forest Service design guide, most of the references previously cited have been either unpublished or contained within city or county municipal codes.

One exception is *Biotechnical and Soil Bioengineering Slope Stabilization*, a textbook written by Gray & Sotir.<sup>(4)</sup> Rockeries are discussed in Chapter 5 of the textbook, although they are referred to as “rock breast walls.” Gray & Sotir describe the typical rockery as 3 m (10 ft) high or less, constructed of courses of rocks that are a single rock-width wide, and within which upper rocks should rest on two lower rocks with at least three-point bearing. Angular rocks are the preferred construction material. They also note rockeries should be primarily used to “defend the toe of the slope and to prevent slope damage by erosion...”

Gray & Sotir conclude rockery stability is governed by the rockery face batter angle and H/B ratio, which is consistent with the previous references discussed. They believe H/B should

typically not exceed 3. Furthermore, they indicate that typical base rock widths are about 0.6 m (2 ft), and, therefore, typically rockery heights are about 1.8 m (6 ft).

The Gray & Sotir text provides a closed form solution that can be used to evaluate rockery stability under the proper conditions. Because, like previous researchers, they concluded stability is typically governed by overturning (in lieu of inter-rock shear and bulging), their equations were developed to solve for overturning stability. Figures 16 and 17 present the Gray & Sotir equations that can be used to compute H/B ratios as a function of the properties of the retained soil, facing rocks, and desired overturning factor of safety. It should be noted that there is a minor error in the equations presented in the textbook that has been corrected in the equations presented throughout this report. The corrected equations have been confirmed as correct by Professor Gray (personal communication).

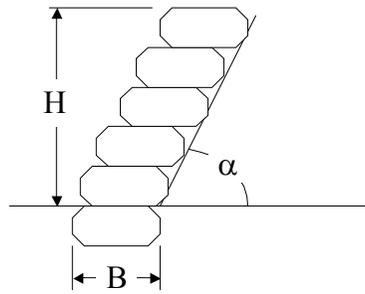
$$\left(\frac{H}{B}\right) = \frac{0.5b \pm \sqrt{\frac{b^2}{4} + \frac{0.33(FS_{OT})\gamma_s\gamma_R K_A \cos\phi}{\sin\alpha}}}{\frac{0.33(FS_{OT})\gamma_s K_A \cos\phi}{\sin\alpha}}$$

**Figure 16. Equation. Height-to-base-width (H/B) as a function of factor of safety, rockery inclination, backslope inclination, and soil and rock properties, from Gray & Sotir.**

$$b = \left[ \frac{\gamma_R \cos\alpha}{\sin^2\alpha} + (FS_{OT})K_A \gamma_s \sin\phi \right]$$

**Figure 17. Equation. Definition of the term “b” in Figure 16.**

The equations presented in Figures 16 and 17 were developed assuming angular rocks; modifications are required for rounded rocks. These equations also neglect backfill cohesion, and, therefore, may be conservative for some circumstances. The equations indirectly address backslope inclination through the use of the active earth pressure coefficient,  $K_A$ .



**Figure 18. Graphic. Assumed geometric relationships to be used for equations shown in Figures 16 and 17.**

In addition to the equations provided above, Gray & Sotir also provide a schematic rockery diagram with several rules of thumb, including:

- Embedding the base rock a minimum of 300 mm (12 in) below the ground surface.
- Utilizing a rockery face batter less than 3V:1H.
- Using a cap rock with a minimum width of 400 mm (16 in).
- Designing for a maximum backslope inclination of 1V:2H.
- Placing a layer of free-draining gravel behind the rockery that is tied into a drainage pipe at the base. The gravel layer should be a minimum 200 mm (8 in) wide with a gradation between 50 and 100 mm (2 and 4 in).
- Designing for a maximum height of 3 m (10 ft).

Many of these recommendations are similar to those previously discussed, including the minimum embedment, rockery drainage, and maximum backslope inclination.

Of additional interest is that much of the early work for the Gray & Sotir textbook was originally published without the equations presented in Figures 16 and 17 in the United States Department of Agriculture, Natural Resources Conservation Service *Engineering Field Handbook* (Chapter 18) in October 1992. Although the *Engineering Field Handbook* implementation of the rockery

design methods proposed by Gray & Sotir appear much less rigorous than the later textbook, it appears the *Handbook* is still in use by some Federal agencies.

### Northern California Region

Rockeries have been designed and constructed in northern California for about the last 10 years. Single tier rockeries are generally capable of supporting cut slopes up to about 4.6 m (15 ft) in height, whereas the maximum height of rockeries used to retain unreinforced fill is typically limited to about 3.7 m (12 ft). For fills between 3.7 and 4.6 m (12 and 15 ft), it is often necessary to reinforce the fill with geogrids to create a mechanically stabilized earth (MSE) fill and use the rockery as a facing element. MSE fills may also be required for heights less than 3.7 m (12 ft) depending on the properties of the retained soil or global stability concerns. Under these conditions, the rockeries serve as a protecting rockery because the MSE is designed to be globally stable without the facing. Protecting rockeries can also be used in conjunction with other types of earth retention systems, such as soil nails, in cut conditions. Examples of single-tier rockeries in both cut and fill conditions are presented as Figures 19 through 21.



**Figure 19. Photograph. Single-tier rockery constructed in cut condition, El Dorado Hills, California (2001).**



**Figure 20. Photograph. “Protecting” rockery used as a facing for a 9.1-m-high (30-ft-high) MSE wall with a 5.5-m (18-ft) maximum tier height in Henderson, Nevada (2001).**



**Figure 21. Photograph. “Protecting” rockery used as a facing material for a two-tier, 7.6-m-high (25-ft-high) soil-nail wall in Rocklin, California (1999).**

As Figures 20 and 21 also show, rockery heights can be significantly increased through the use of tiers. The primary considerations for rockeries in tiered conditions are the surcharge posed on lower tiers by upper tiers and global stability of the overall system. Figure 22 shows another example of a three-tier, 9.1-m-high (30-ft-high) protecting rockery constructed to face an MSE fill.



**Figure 22. Photograph. Three-tier, 9.1-m-high (30-ft-high) MSE slope with protecting rockery facing, in Folsom, California (2003).**

The authors (SAGE) have developed in-house analysis programs for rockery design that are based on the concepts common to conventional gravity wall design. These concepts generally follow the rational approach presented in the American Association of State Highway Transportation Officials (AASHTO) for gravity wall design. Driving forces are based on lateral earth pressures caused by the retained soil (Coulomb method) as well as any additional surcharges, such as sloped backfill or traffic. Resisting forces are based on soil-rock friction and soil bearing capacity, generally using parameters provided by the project geotechnical engineer. As recommended by the AASHTO, passive resistance at the toe of the rockery is usually neglected. The analysis method consists of assuming a base rock size and checking to determine if the overall factors of safety against bearing, overturning, and sliding are acceptable. The following static factors of safety are used:

- $FS_{OT} = 2.0$
- $FS_{SL} = 1.5$
- $FS_{BC} = 3.0$  (or less, if allowed by the geotechnical engineer)

Unlike some of the procedures developed in the Pacific Northwest, rockeries are specified by base width rather than through a man rock designation. The minimum base width is nominally specified as  $0.5H$ . By specifying a base width, closed-form equations can be solved for the

factors of safety listed above. Designation of a specified base width also removes some of the construction uncertainty and variability previously associated with “man rock” designations. Sizes of overlying rocks are then determined by specifying a vertical back of rockery, a 4V:1H batter for the front face of the rockery, and a minimum cap rock size.

Because the seismic activity is relatively high throughout California, the rockeries are designed for use in Uniform Building Code (UBC) Seismic Zones 3 and 4.<sup>(8)</sup> Seismic earth pressures are determined using the Mononobe-Okabe method,<sup>(9)</sup> and the potential for rockery movement due to the seismic surcharge pressure is checked using an allowable displacement method proposed by Richards and Elms<sup>(10)</sup> for gravity walls. This method computes a value of the seismic coefficient,  $k_h$ , by assuming an allowable permanent rockery displacement in a manner analogous to the Newmark sliding block analysis procedure originally developed for evaluation of seismic slope stability. The computed value is typically less than the presumptive value of one-half the peak ground acceleration required by other design methodologies. The procedure consists of the following steps:

1. Determining a tolerable permanent displacement of the rockery.
2. Developing a design seismic coefficient ( $k_h$ ).
3. Determining the seismic force on the rockery using the Mononobe-Okabe approach.
4. Determining the required rockery weight to resist rockery failure.
5. Checking that an adequate factor of safety has been achieved to yield a high degree of confidence that actual displacement will be less than or equal to the allowable displacement.

Based on review and analysis of the Richards and Elms methods, Whitman<sup>(11)</sup> concluded a factor of safety of 1.1 with respect to the required wall weight resulted in a 90% probability that actual wall movement will not exceed the tolerable displacement. The tolerable displacement is generally estimated as about 5% of the base width. Considering most base rock widths vary between 0.9 and 2.4 m (3 and 8 ft), tolerable displacements are typically about 50 to 120 mm

(1.8 to 4.8 in). The significance of this method is that smaller base rocks are required for seismic design that would be required if no movement were allowed.

A key assumption in this analysis method is that the rockery can be treated as a single unit, and that gravity retaining wall design and analysis procedures can be applied to address the stability of the overall rockery system. Unlike the method proposed by Hemphill, this method assumes that proper construction of the rockery will resolve any concerns related to inter-rock stability. Specifically, inter-rock stability is addressed by specifying that the contact point between the upper and lower rocks is always within 150 mm (6 in) of the average rockery face; that is, the contact point is at, or very close to, the front of the rockery. This results in an acceptable factor of safety for inter-rock overturning, and, because inter-rock overturning has been shown to be more critical than inter-rock sliding, inter-rock sliding is also resolved. Similar to other methods described previously, several prescriptive requirements are specified for each rockery, including:

- The long rock dimension should be placed perpendicular to the slope.
- The rocks should slope down towards (into) the slope at an inclination of at least 5%.
- At least 300 mm (12 in) of granular drainage material consisting of 100- to 150-mm (4- to 6-in), screened crushed rock should be placed behind the rockery.
- The drainage material should be separated from the retained soil using geotextile filter fabric and drained to a suitable outlet using a minimum 100-mm-diameter (4-in-diameter) PVC drain pipe.
- Cap rocks should weigh at least 90 kg (200 lb) and should not be moveable by hand.
- Finishing rocks less than 90 kg (200 lb) should be grouted in place.
- Voids greater than 150 mm (6 in) should be chinked, and chinking rocks should not be moveable by hand.
- The outermost point of contact between an upper and a lower rock should be within 150 mm (6 in) of the face of the rockery.

- Facing rocks should conform to materials and placement specifications described in Section 72–Slope Protection of the State of California, Department of Transportation (CalTrans) Standard Specifications, <sup>(12)</sup> and should be angular, indurate, and non-fractured.

To verify that the rockeries are constructed according to these specifications, intermittent observation by an experienced engineer is required during construction of all rockeries taller than 1.8 m (6 ft). Deficient construction materials or procedures can then be identified and corrected.

### **SUMMARY OF EXISTING DESIGN METHODS**

As evidenced by the preceding sections, the literature search performed for this study suggests that the analytical methods used for the design and evaluation of rockeries is limited and varies considerably. However, although the design methods vary, many of the general rules-of-thumb used to guide rockery design are relatively consistent. The various design procedures and rules-of-thumb are summarized (by source) in Table 3 (Prescriptive Methods) and Table 4 (Analytical Methods).

Although not shown in Table 3, none of the prescriptive methods specify minimum values for the base rock width; chinking; rock shape or angularity; or the minimum cap rock width or weight.

**Table 3. Comparison of prescriptive design methods in different cities.**

Design Parameter	City of Seattle, WA <sup>(2)</sup>	City of Brier, WA <sup>(3)</sup>	City of Knoxville, TN <sup>(13)</sup>	City of Sparks, NV (unpublished)
Maximum height, m (ft)	1.8 (6) <sup>(a)</sup>	1.8 (6)	1.2 (4) <sup>(a)</sup>	1.8 (6)
Minimum embedment, mm (in)	300 (12)	N/A	600 to 900 (24 to 36)	300 (12)
Maximum batter	4V:1H	N/A	12V:1H	4V:1H
Maximum backslope	3V:8H	1V:3H	N/A	1V:3H
Rock size designation	Man rocks	N/A	Width 20 to 60 cm	Man rocks
Rockerries allowed in fill conditions?	No <sup>(b)</sup>	Yes	N/A	Yes <sup>(b)</sup>
Backdrain required?	Yes	Yes	Yes	Yes
Minimum backdrain thickness, mm (in)	300 (12)	N/A	N/A	460 (18)

Notes: <sup>(a)</sup>Maximum height is for prescriptive design; taller rockeries are allowed with engineered design.

<sup>(b)</sup>Not considered a retaining structure; no surcharge is allowed.

**Table 4. Comparison of analytical design methods.**

Design Parameter	Gray & Sotir <sup>(4)</sup>	Gifford & Kirkland <sup>(5)</sup>	Hemphill (1990; unpublished)	SAGE (unpublished)
Design methodology	Solve equation for height-to-base-width (H/B) ratio based on soil friction angle ( $\phi$ ), overturning factor of safety ( $FS_{OT}$ ), and facing geometry	Solve for critical H/B ratio based on $\phi$ and slope inclination (i) values; use poorly constructed rockery (PCR) curves	Design rockery in 0.3-m (1-ft) increments from top down; solve for $FS_{OT}$ above finish grade at base of rockery, and sliding factor of safety ( $FS_{SL}$ ) for embedded portion of rockery	Design as a gravity retaining structure; satisfy internal stability through rock placement specification; include seismic forces
Minimum factors of safety:				
Overturning ( $FS_{OT}$ )	No minimum specified	1.0 <sup>(a)</sup>	1.5 to 2.5 <sup>(b)</sup>	2.0
Sliding ( $FS_{SL}$ )	N/A	N/A	1.5 to 2.5 <sup>(b)</sup>	1.5
Bearing ( $FS_{BC}$ )	N/A	N/A	N/A	3.0
Maximum height, m (ft)	3.0 (10)	4.6 (15)	N/A	4.6 (15)
Minimum base width	N/A	H/3	N/A	H/2
Embedment determination	Prescriptive; 300 mm (12 in) minimum	Prescriptive; 300 mm (12 in) minimum <sup>(c)</sup>	Calculated; based on $FS_{SL}$	Prescriptive; 300 mm (12 in) minimum
Maximum face batter	3V:1H	4V:1H	N/A	4V:1H
Maximum backslope inclination	1V:2H	Flat	N/A	1V:1.5H
Rock size designation	Width	Man rocks	Width	Width
Used for fill conditions	No	Possible	Yes	Yes <sup>(d)</sup>
Backdrain required?	Yes	Yes	Yes	Yes
Minimum backdrain thickness, mm (in)	200 (8)	Varies; based on back cut	N/A	300 (12)
Minimum cap rock weight, kg (lb)	N/A	N/A	N/A	90 (200)
Minimum cap rock width, mm (in)	400 (16)	N/A	460 to 610 (16 to 24)	400 (16)
Rock shape	Angular	Angular	N/A	Angular
Chinking required?	N/A	Yes	Yes; voids >150 mm (6 in)	Yes; voids >150 mm (6 in)

Notes: <sup>(a)</sup> Assumes “critical” H/B ratio is based on factor of safety of 1.0.

<sup>(b)</sup> Factor of safety varies depending on level of observation by engineer.

<sup>(c)</sup> Minimum embedment to base of rock; requires an additional 150 mm (6 in) gravel foundation layer.

<sup>(d)</sup> For rockeries in fill more than 3.7 m (12 ft) tall, used as a facing for mechanically stabilized earth (MSE).



**CHAPTER 3 – COMPARISON OF EXISTING DESIGN METHODS**

Several different design methodologies were identified in Chapter 2 that have been used to design rockeries. Prescriptive design methods based on assumed rock dimensions are appropriate for rockeries less than 1.8 m (8 ft) high in cut conditions.<sup>(2,3)</sup> Analytical or closed-form solution methods, such as those suggested by Gray & Sotir,<sup>(4)</sup> Hendron,<sup>(5)</sup> Hemphill<sup>(unpublished)</sup>, and SAGE<sup>(unpublished)</sup>, have been used for rockeries that exceed 1.8 m (8 ft) or rockeries constructed in fill conditions.

Because each method makes various assumptions regarding parameters such as soil strength, backfill geometry, and rockery face inclination, it generates different rockery design geometries for the same input data. To evaluate the differences between each method and determine the methods most appropriate for use as a design standard for retaining rockeries, four example design cases were developed and considered. The cases represent a typical range of soil conditions, backslope geometry, and back cut geometry that might be encountered for a typical retaining situation, including:

- Case #1 – Level backslope, 8V:1H back cut, loose granular backfill ( $\phi = 30^\circ$ ).
- Case #2 – Level backslope, 8V:1H back cut, medium dense to dense granular backfill ( $\phi = 36^\circ$ ).
- Case #3 – 1V:2H backslope, 8V:1H back cut, medium dense granular backfill ( $\phi = 34^\circ$ ).
- Case #4 – Level backslope, 2V:1H back cut, loose granular backfill ( $\phi = 30^\circ$ ).

For each case, the rockeries were analyzed for heights of 1.2, 2.1, and 3.0 m (4, 7, and 10 ft). Surcharges were not considered because most of the design procedures being reviewed are unable to incorporate surcharge pressures. The soil and rockery design properties used for each case are summarized in Table 5 and represented graphically on Figures 23 and 24. For the

purposes of comparing and contrasting the results from the various analysis procedures, impacts due to other factors, including frost heave or scour, were neglected.

**Table 5. Soil and rockery properties used for benchmarking of analysis procedures.**

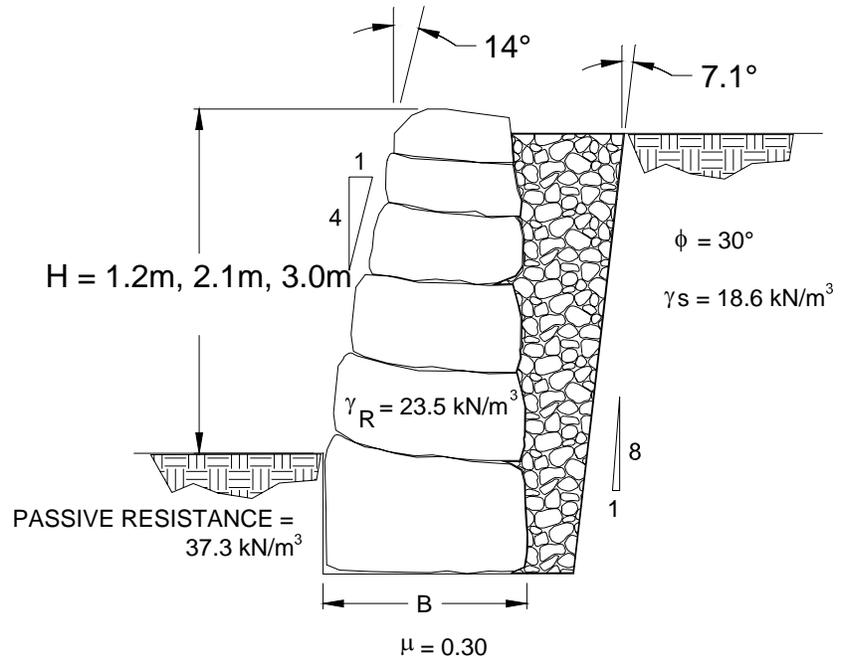
Design Property	Case #1	Case #2	Case #3	Case #4
Soil Friction Angle, $\phi$ ( <i>assumed</i> )	30	36	34	30 / 40 <sup>(a)</sup>
Retained Ground Surface Inclination ("Backslope"), $\beta$ ( <i>assumed</i> )	Level	Level	1V:2H	Level
Allowable Sliding Friction Factor, $\mu$ <sup>(b)</sup> FS=1.5	0.30	0.35	0.35	0.30
Allowable Passive Resistance at Toe (EFP), kN/m <sup>3</sup> (pcf) FS=1.5 <sup>(c)</sup>	37.3 (230)	49.0 (300)	44.1 (275)	37.3 (230)
Face Batter (V:H) ( <i>assumed</i> )	4:1 (14°)	4:1 (14°)	4:1 (14°)	4:1 (14°)
Back Cut Inclination (V:H) ( <i>assumed</i> )	8:1 (7.1°)	8:1 (7.1°)	8:1 (7.1°)	2:1 (26.5)
Retained Soil Density $\gamma_s$ , kN/m <sup>3</sup> (pcf) ( <i>assumed</i> )	18.6 (120)	18.6 (120)	18.6 (120)	18.6 (120)
Rockery Facing Density $\gamma_R$ , kN/m <sup>3</sup> (pcf) ( <i>assumed</i> )	23.5 (150)	23.5 (150)	23.5 (150)	23.5 (150)

Notes: EFP = Equivalent Fluid Pressure

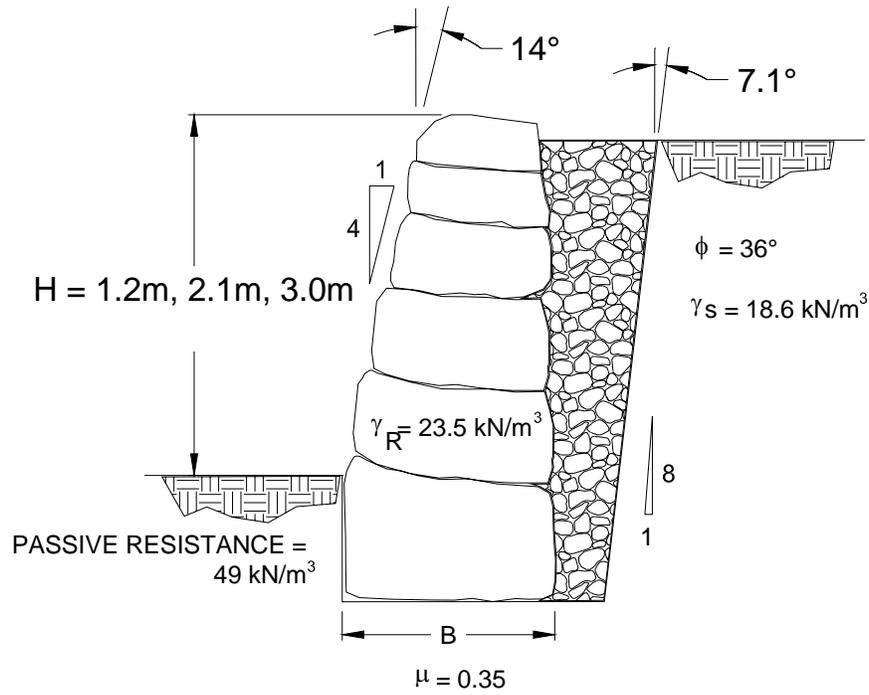
<sup>(a)</sup> For Case #4, a relatively conservative value of 40° was selected for the friction angle ( $\phi$ ) of the crushed rock backfill, even though the actual value would likely be higher.

<sup>(b)</sup> Passive resistance at the toe was computed using a Rankine pressure coefficient ( $K_p$ ) equal to  $\tan^2(45+\phi/2)$ . Reduced from ultimate to allowable value using a factor of safety of 1.5.

<sup>(c)</sup> Frictional resistance ( $\mu$ ) was computed using an average of  $\tan\phi$  and  $\tan(2/3\phi)$ . Reduced from ultimate to allowable value using a factor of safety of 1.5.



CASE #1



CASE #2

Figure 23. Graphic. Design geometry for example design Cases #1 and #2.

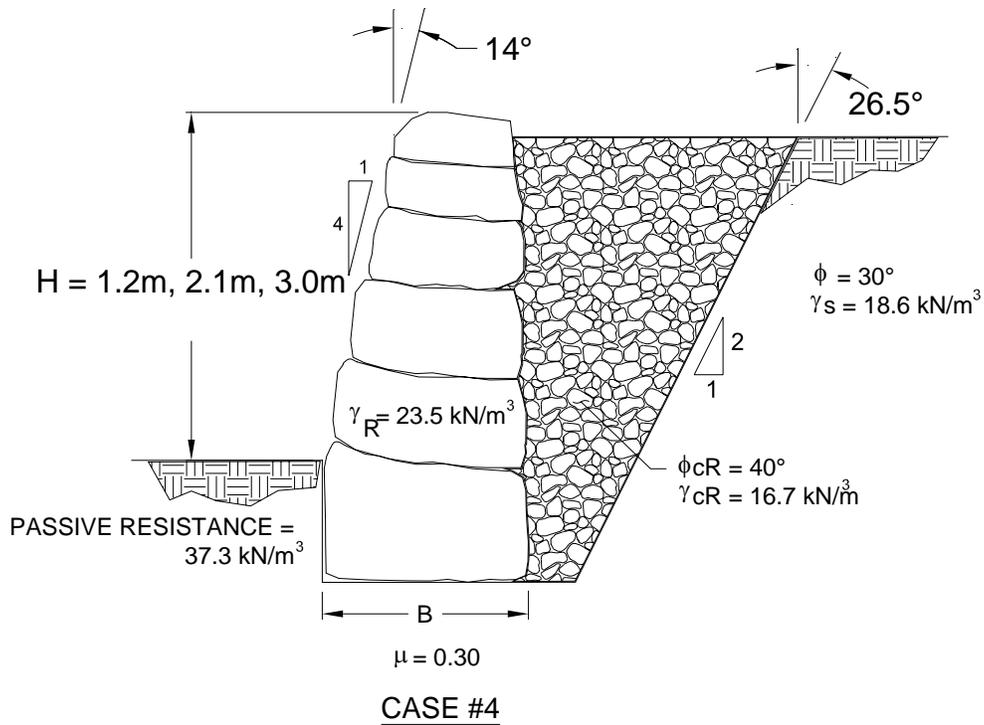
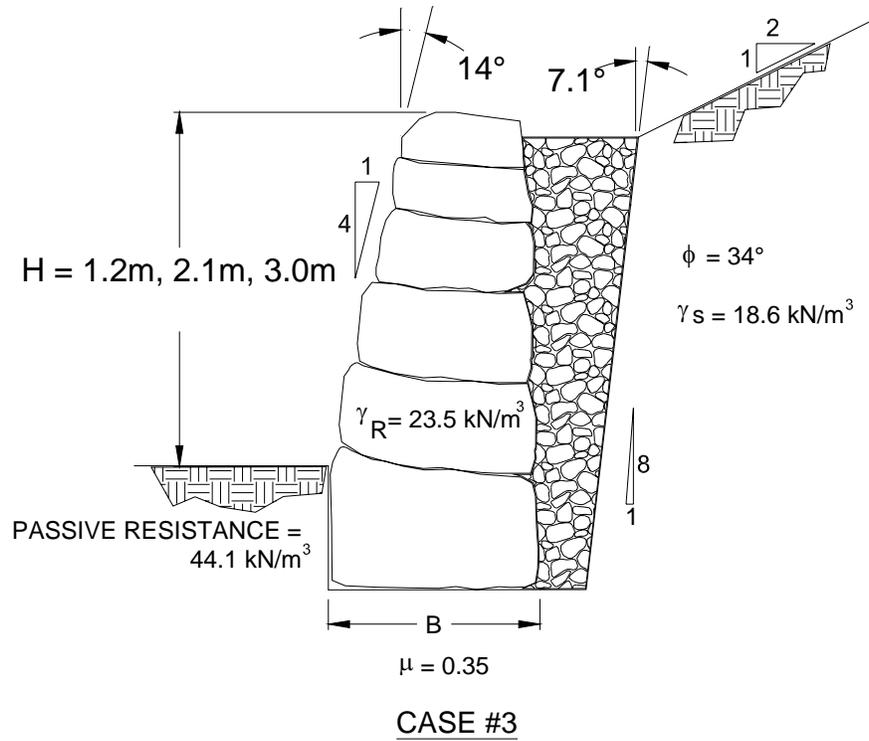


Figure 24. Graphic. Design geometry for example design Cases #3 and #4.

## COMPARISON OF METHODS

The required minimum rockery base rock width was determined for each of the typical cases listed above using the methodologies by Gray & Sotir<sup>(4)</sup>, Hendron<sup>(5)</sup>, Hemphill, and SAGE. Case #3 could not be checked using the Hendron equations because the equations were developed for level backfill conditions only.

In general, all four methods are based on the assumption that the retained soil exerts a lateral earth pressure on the back of the rockery and that the rockery must resist this pressure through rock interaction, rockery weight, and rockery size. In that respect, the methods are similar and, as a result, the computed base widths are similar.

With the exception of the Hendron charts, the methods reviewed all incorporated the concept of a factor of safety into the analysis. The analysis method currently used by SAGE typically uses a minimum overturning factor of safety ( $FS_{OT}$ ) of 2.0 for checking external and internal (inter-rock) overturning. However, the analysis is often controlled by external sliding, for which a minimum factor of safety ( $FS_{SL}$ ) of 1.5 is used. This is in contrast to statements made by other authors that overturning controls rockery design and sizing.

For the Hemphill method, a factor of safety of 2.0 was also used for both  $FS_{OT}$  and  $FS_{SL}$ . This value is based on guidance in the Hemphill report regarding rockeries constructed with intermittent observation by the designer. Because rockery construction is typically not observed on a full-time basis, we believe this is an appropriate assumption for design consistent with this method.

A design factor of safety is not proposed by Gray & Sotir<sup>(4)</sup> for use with their procedure. Because this method is based solely on overturning stability, a value of  $FS_{OT}$  of 2.0 was selected for the evaluation, which is consistent with the other methods evaluated.

Unlike the other three methods, the Hendron equation was developed to determine critical H/B ratios. It is implied in the Gifford and Kirkland paper<sup>(5)</sup> that the equations were derived for a factor of safety of 1.0, although it is never explicitly stated. As a result, although the computed base rock widths appear similar to other methods, wider base rocks would actually be required to achieve equivalent factors of safety.

For the Gray & Sotir, Hendron, and SAGE methods, the rockery thickness is assumed to be either uniform or to taper from bottom to top with a relatively shallow inclination (4V:1H to 6V:1H). Therefore, the computed mass used to resist overturning is similar for each procedure, and these methods result in similar base rock widths when computing  $FS_{OT}$ .

However, the Hemphill method checks for overturning in a step-wise fashion from the top down. As a result, upper rocks tend to be much smaller than lower rocks, which results in a less uniform, and more stepwise, rockery thickness. Furthermore, because the size and weight of the uppermost facing rocks is minimized early in the design procedure, there is less normal force (weight) with which to generate frictional resistance, and, as a result, the design procedure tends to result in wider base rocks than the other methods. In addition, the base rocks computed using the Hemphill method tend to be more deeply embedded than the other methods in which foundation depth is generally prescribed.

The computed base rock widths for each method and design case are presented in Table 6. The calculations generally yielded base widths that are less than the minimum prescriptive values recommended in each reference. Therefore, if these methods were used for design, the prescriptive base rock widths would control the design.

While performing the analyses for each of the different methods, insights were gained regarding specific methods and results. These insights are described in the following subsections.

### **Gray & Sotir Method**

As discussed in Chapter 2, the equations presented in the Gray & Sotir<sup>(4)</sup> textbook for design of “rock breast walls” appear to contain an error. The corrected equations, which have been confirmed with the authors, are summarized below in Figures 25, 26, 27, and 28. For the computation of the coefficient of active earth pressure ( $K_A$ ), the value of  $\delta$  should be assumed to be equal to  $\phi$ .

Table 6. Comparison of computed base rock widths using four design methods.

Computed Base Width “B” (meters) <sup>(a)</sup>						
Design Cases		Methodology				
		Prescriptive B=0.3H - 0.5H	SAGE FS <sub>OT</sub> ≥ 2 FS <sub>SL</sub> ≥ 1.5	Hemphill FS <sub>OT</sub> ≥ 2 FS <sub>SL</sub> ≥ 2	Gray & Sotir FS <sub>OT</sub> ≥ 2	Hendron <sup>(b)</sup> FS <sub>OT</sub> = 1
Case #1	1.2 m	0.4-0.6	0.3	1.4	0.3	0.4
	2.1 m	0.6-1.1	1.0	2.3	0.5	0.7
	3.0 m	0.9-1.5	1.4	3.2	0.7	1.0
Case #2	1.2 m	0.4-0.6	0.5	0.8	0.2	0.3
	2.1 m	0.6-1.1	0.8	1.6	0.4	0.5
	3.0 m	0.9-1.5	1.0	2.1	0.5	0.7
Case #3	1.2 m	0.4-0.6	0.6	1.6	0.3	N/A <sup>(c)</sup>
	2.1 m	0.6-1.1	1.1	3.0	0.5	N/A <sup>(c)</sup>
	3.0 m	0.9-1.5	1.5	4.1	0.8	N/A <sup>(c)</sup>
Case #4	1.2 m	0.4-0.6	(0.4) <sup>(d)</sup>	0.5 (1.2) <sup>(d)</sup>	0.1 (0.2) <sup>(d)</sup>	0.1 (1.0) <sup>(d)</sup>
	2.1 m	0.6-1.1	(0.8) <sup>(d)</sup>	1.3 (2.0) <sup>(d)</sup>	0.2 (0.35) <sup>(d)</sup>	0.1 (1.7) <sup>(d)</sup>
	3.0 m	0.9-1.5	(1.1) <sup>(d)</sup>	1.8 (2.8) <sup>(d)</sup>	0.3 (0.5) <sup>(d)</sup>	0.2 (2.4) <sup>(d)</sup>

Notes: <sup>(a)</sup> Only computed values are shown. In most cases, a prescriptive, minimum value such as 0.5H or 0.3H would be used for design.

<sup>(b)</sup> The Hendron equations are for the critical H/B ratio with a presumed factor of safety of 1.0. The method does not provide for inclusion of a specified factor of safety.

<sup>(c)</sup> Method derived for level backfill conditions only; not applicable to sloping backfill.

<sup>(d)</sup> Values in (parentheses) recalculated assuming the soil failure plane lies entirely within the crushed rock backfill zone. Based on  $\gamma_{\text{crushed rock}} = 16.7 \text{ kN/m}^3$ ,  $\phi=40^\circ$ , and a vertical face to the back of the facing rocks.

The revised equations were used to check a chart solution presented in the Gray & Sotir text. The results closely match the sample chart provided in Gray & Sotir for level backfill conditions. However, the charted values are about 5% to 10% lower than the original Gray & Sotir chart for backslope inclinations ( $\beta$ ) between 15° and 20° degrees. We believe this difference is related to the method in which the  $K_A$  value is computed for sloping backfill conditions. The method used to calculate  $K_A$  was not discussed in the Gray & Sotir text.

Of note is the fact that the equation for well-constructed rockeries (WCRs) developed by Hendron, which is shown in Figure 9, is identical to the Gray & Sotir equations as revised by the authors. Some factors have been presented differently in the two equations; for example, Gray &

Sotir have substituted the term “b” (see Figure 27) into the equation, whereas Hendron wrote out all the terms.

$$K_A = \frac{\cos^2(\psi + \phi)}{\cos^2(\psi) \cdot \cos(\delta - \psi) \cdot \left[ 1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \beta)}{\cos(\delta - \psi) \cdot \cos(-\psi - \beta)}} \right]^2}$$

Figure 25. Equation. Computation of active earth pressure coefficient ( $K_A$ ) by the Coulomb method.

$$\left(\frac{H}{B}\right) = \frac{0.5b \pm \sqrt{\frac{b^2}{4} + \frac{0.33(FSOT)\gamma_S\gamma_R K_A \cos\phi}{\sin\alpha}}}{\frac{0.33(FSOT)\gamma_S K_A \cos\phi}{\sin\alpha}}$$

Figure 26. Equation. Corrected equation for height-to-base-width ( $H/B$ ) ratio for use with Gray & Sotir analysis method.

$$b = \left[ \frac{\gamma_R \cos\alpha}{\sin^2\alpha} + (FSOT)K_A \gamma_S \sin\phi \right]$$

Figure 27. Equation. Corrected definition of the term “b” in Figure 18.

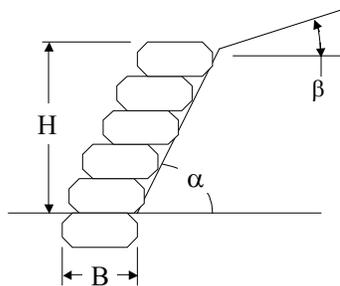


Figure 28. Graphic. Assumed geometric relationships to be used for equations shown in Figures 25, 26, and 27.

Table 6 also indicates that base rock widths computed using the Gray & Sotir method ( $FS_{OT} = 2.0$ ) are actually smaller (less conservative) than those computed using the Hendron method ( $FS_{OT} = 1.0$ ). Normally, one would assume the smaller rock widths would be computed using the method with the lower factor of safety. However, Gifford and Kirkland<sup>(5)</sup> recommended the Hendron PCR design curves be used for design in lieu of the WCR curves (see Chapter 2 and Figure 9), and as a result, the Hendron method is more conservative than the Gray & Sotir method.

### **Hemphill Sliding and Overturning Stability Analyses**

As discussed previously, the target factor of safety for both  $FS_{SL}$  and  $FS_{OT}$  in the Hemphill analyses is recommended to be between 1.5 and 2.5. Hemphill recommends lower factors of safety be used as the amount of control and observation the design engineer has during construction increases. A factor of safety of 2.0 was used in the analyses performed for this study, which is consistent with Hemphill's recommendations for intermittent construction observation. This factor of safety is applied to both internal (inter-rock) sliding and overturning above the ground line and external sliding below the ground line.

As discussed previously, the Hemphill method tends to minimize the size and weight of the rocks at the top of the rockery, which reduces the overall structure weight over the exposed face of the rockery. The net impact is that there is less weight to provide a normal force to resist sliding along the bottom of the base rock. Furthermore, the value of  $FS_{SL}$  used is higher than other analysis methods, and more resisting force is required to resist sliding while obtaining the desired factor of safety. As Table 6 demonstrates, the combined impact of these factors results in base rock sizes that are larger than the other three methods. While the use of a higher factor of safety ( $FS_{SL}$ ) with this method leads to somewhat of an "apples-to-oranges" comparison to the other methods, the value of  $FS_{SL}$  was selected based on the criteria provided in the Hemphill report. This type of comparison has value because it is important to understand the differences in design that can be expected based on the method-specific recommendations.

In addition, the required embedment computed for 1.2- to 3.0-m-tall (4- to 10-ft-tall) rockeries is 1.0 to 1.7 m (3.3 to 6.0 ft), which is about 60 percent to 80 percent of the exposed rockery height.

For the other three methods used, the embedment is generally a prescribed depth based on the conditions at the toe of the rockery and varies between 0.3 and 0.6 m (1 and 2 ft).

#### **Shallow Back Cuts (Design Case #4)**

The equations developed by Hendron and Gray & Sotir account for the inclination of the back cut slope by assuming the rockery is constructed with facing rocks directly against the stable soil slope. In this respect, the methods assume the rockery is constructed as a protecting rockery rather than a retaining rockery. Consequently, the minimum values of B developed for Case #4 are relatively small. Because the Hemphill method also accounts for this through the selection of  $K_A$ , which decreases as the back cut angle ( $\psi$ ) increases, smaller base rock widths are computed for Case #4 than for the other three cases. The SAGE equations have been derived in such a way that they are not applicable without extensive modification to the evaluation of a sloping soil condition with  $\alpha$  less than  $70^\circ$ . As a result, base rock widths are only provided for the crushed rock backfill condition, and were not computed for the shallow slope condition.

Many modern rockeries are constructed by filling the space between the facing rocks and native soil with crushed rock, as shown in Figures 23 and 24. As Figure 24 demonstrates, the constructed rockery system may not match the geometric assumptions used by Hendron or Gray & Sotir for an inclined back cut because although the soil slope is relatively flat, the rockery face is still relatively steep. Therefore, for the Hemphill, Hendron, and Gray & Sotir methods, the minimum base widths required were checked to determine if the shallow retained soil or retained crushed rock was the controlling backfill condition.

For this evaluation, a conservative  $\phi$  value of  $40^\circ$  was used for the crushed rock backfill.

Although the actual value of  $\phi$  is likely to be significantly higher for the crushed rock, we selected a conservative value that would provide a “worst case” value if poor quality rock were used.

Review of Table 6 indicates that for the Gray & Sotir method, the values of B increase to values that are similar to Cases #1 and #2, which have a similar geometries but lower  $\phi$  values. Similar results were found using the Hemphill method.

Evaluating the crushed rock backfill using the Hendron method, however, resulted in computed base widths that were larger than previously computed for Case #1. Review of the equation for this method indicates the base widths increase relatively quickly as the assumed back cut (in this case, the back of the rockery) nears vertical. The full derivation of the Hendron equations is not presented in Gifford and Kirkland, so it is difficult to ascertain why this occurs. However, it appears that because the equations were originally derived for protecting rockeries, assumptions were made that lead to conservative rockery designs for retaining applications.

### **SUMMARY**

Relatively speaking, the Gray & Sotir and Hendron methods produced the narrowest overall base widths as compared to the other methods, except for Case #4. For Case #4, the base widths computed for the gravel backfill by Hendron's method were larger than those for the SAGE and Gray & Sotir methods, and comparable to the Hemphill method, as discussed previously. When only the shallow soil slope was considered for Case #4, the Hendron and Gray & Sotir procedures produced very narrow base rock widths that are more consistent with a protecting rockery.

The Hemphill method produced the largest base widths because it tends to optimize (minimize) the rock sizes at the top of the rockery at the expense of the base. It is likely the results could be made to more closely match the other methods if the upper rocks are sized artificially large for the analysis method; that is, if overly large factors of safety were used at the top of the rockery. However, this is impractical for design because the base dimension would need to be known in advance. The SAGE method produced values that were near the average for the other methods.

The calculated base widths (B) presented in Table 6 represent the nominal values computed from each method. However, the minimum prescriptive values for B are between  $0.3H$  and  $0.5H$ , and would likely be used for design even if the analysis indicated a smaller base rock could be used. Furthermore, for the Hendron method, the ratios presented in Table 6 include a  $FS_{OT}$  of only 1.0. Because this represents incipient failure, larger base widths would be required for design regardless.

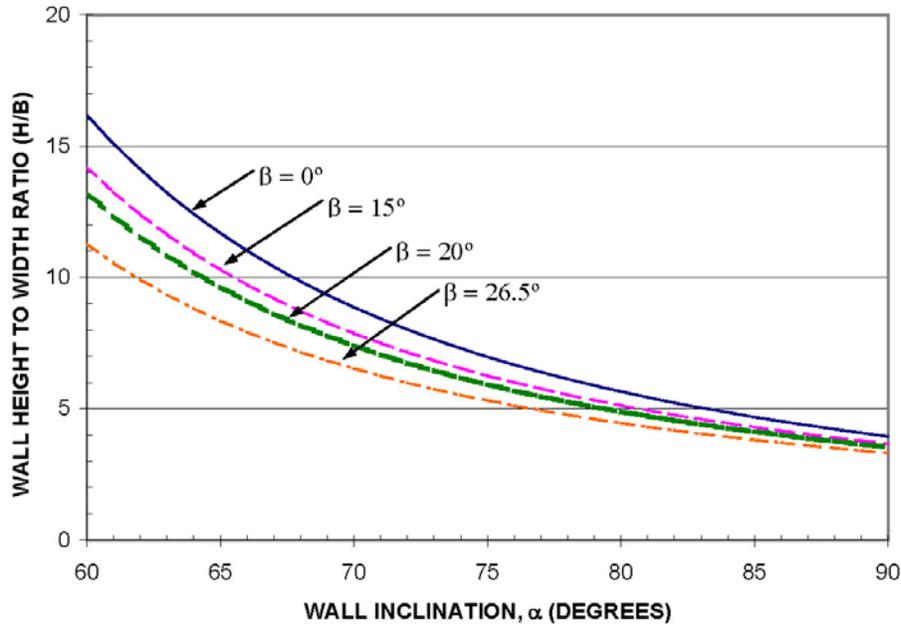
## CONCLUSIONS

### Retaining Rockeries

Based on the evaluation performed for this report, it can be concluded that each method provides valuable tools for rockery evaluation or design. The methods proposed by Gray & Sotir, Hendron, and Hemphill, although difficult to adapt to a rigorous design methodology, can all be easily adapted for evaluation of the stability of existing rockeries. Although the Hemphill method seems to result in the most conservative rockery designs, it has the advantage that it can be adapted to seismic design because the loads are computed within the analysis. The Hendron and Gray & Sotir methods address static loads only.

At a minimum, these methods provide valid screening methods that can be used to determine if further analysis is warranted. Charts developed from the Gray & Sotir and Hendron analyses, in particular, can be used for rapid evaluation of a particular combination of soil conditions and slope geometry. An example chart developed using the Gray & Sotir method is presented as Figure 29.

For the recommended design methodology to be useful, however, it must be adaptive to multiple design variables, including rockery size and height, backslope conditions, soil conditions, and applied loading. Ultimately, a closed-form analytical analysis method, such as the method used by SAGE, appears to be the most versatile for rockery design. Because this series of equations is based on gravity wall design methodology, the equations can be easily adapted to various site and soil conditions, vehicle surcharges, guardrail loads, and seismic loading conditions. This method can also be adapted to existing FHWA requirements for segmental gravity wall design. Moreover, these closed-form equations typically result in rockery base widths,  $B$ , on the order of  $0.3H$  to  $0.5H$ , which is consistent with the “rules of thumb” for most of the methods reviewed during this study.



**Figure 29. Chart. Typical chart plotting slope inclination ( $\alpha$ ) vs. height-to-base-width (H/B) ratio using the Gray & Sotir method. Chart developed for  $\phi = 32^\circ$ ,  $\gamma = 18.8 \text{ kN/m}^3$  (120 pcf), and FS = 2.**

An additional advantage of a closed-form method is that it can be adapted to unique situations, which gives the Design Engineer or Geotechnical Engineer flexibility. Because the method evaluates any driving or resisting forces acting on the rockery structure, the equations can be easily adapted to include forces other than lateral earth forces. Thus, the engineer is free to use engineering judgment when applying loads to the rockery system.

### Protecting Rockeries

Protecting rockeries, which are generally used for erosion protection at the toe of slope or channel, are not the primary focus of this report. However, where slopes are inherently stable enough for the rockeries to be constructed as protecting rockeries, the use of smaller base rocks between  $0.25H$  and  $0.5H$  is acceptable. In this case, the rockery acts more like “organized riprap” than a retaining structure. In general, the rockeries can be considered protecting rockeries when the slope inclination is shallower than the potential failure envelope for a given friction angle, that is,  $\alpha < (45^\circ + \phi/2)$  or  $\psi > (45^\circ - \phi/2)$ . However, the Geotechnical Engineer

should perform a global stability analysis using appropriate slope stability software to confirm the global stability of the soil slope.

The Hendron and Gray & Sotir methods appear to be most applicable for designing protecting rockeries. Because these methods are based on the evaluation of only one method of potential instability (overturning), they do not account for other failure mechanisms, such as sliding or seismic loading. Furthermore, these methods are based on a limited set of assumptions and are difficult to extrapolate to other design cases, such as seismic loading or traffic surcharges.

**CHAPTER 4 – RECOMMENDED ROCKERY DESIGN GUIDELINES**

Based on the thousands of commercial rockeries in existence that are performing well, rockeries can be a dependable and effective retaining solution provided they are properly designed, detailed, and constructed. In this respect, rockeries are no different from conventional reinforced concrete retaining walls or mechanically stabilized earth or embankment (MSE) walls. While there is no question that a well-designed rockery that is constructed poorly will perform poorly, the evidence indicates that well-constructed, modern rockeries still perform well even though the design procedures used may vary considerably.

A primary objective of this study is to develop rational design procedures that can be used by Federal Lands Highway (FLH) geotechnical engineers for the design of future rockeries and evaluation of existing rockeries. Rockeries are composed of large blocks of stacked rock, heavy enough and dimensionally adequate to form a structure that resists overturning and sliding forces. In this respect, rockeries can be treated as gravity walls, and can be analyzed rationally using modified forms of conventional gravity retaining wall design methodologies.

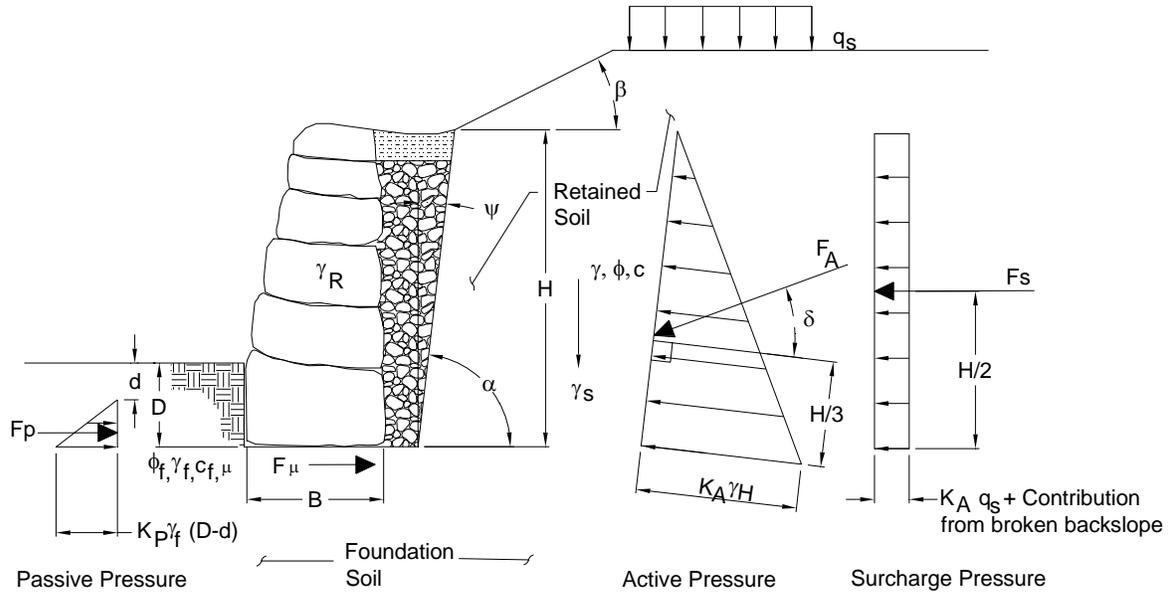
**STATIC DESIGN**

Design of any retaining structure involves the determination of two categories of forces—driving forces and resisting forces. For rockeries, driving forces may include lateral earth pressures behind the rockery, surcharge pressures (both vertical and horizontal), hydrostatic fluid pressures, and seismic pressures. Resisting forces can include the total weight of the rockery and individual rocks, inter-rock friction, base rock–foundation friction, and, in some cases, passive pressure at the toe of the rockery. Where Coulomb earth pressures are used, the vertical component of the active earth pressure can also aid in stabilizing the rockery.

**Design Parameters**

A typical rockery section is shown in Figure 30, along with the design parameters and dimensions that must be determined prior to rockery design. Driving forces include those from

the active soil pressure ( $F_A$ ) and surcharge loads ( $F_S$ ). Resisting forces include base friction ( $F_\mu$ ) and passive resistance ( $F_P$ ), if used.



**Figure 30. Graphic. Schematic rockery section showing critical dimensions and parameters to be determined for design.**

For design, the project Geotechnical Engineer should provide the following geotechnical parameters:

- Friction angle ( $\phi$ ) and true cohesion ( $c$ ) for both retained and foundation soils. In most cases, the effective stress values ( $\phi'$  and  $c'$ ) will be used for design. For the purposes of this report, these values are simply labeled as “ $\phi$ ” and “ $c$ ” and it is assumed the user will use the appropriate parameters.
  - Total stress parameters may occasionally be required, such as in the case of rapid water drawdown for a rockery used in a shoreline application or when checking the stability of a temporary cut slope. The need for total stress parameters should be determined on a case-by-case basis.
  - Because effective (drained) friction angles are generally used for design, it is common to conservatively assume that soil cohesion is zero. This results in a

higher active pressure on the back of the rockery and removes uncertainty regarding non-uniform or problematic soils, such as residual soils. Modifications to the equations presented herein would be required if a non-zero value of “c” is used.

- Interface friction angle ( $\delta$ ). Per Coulomb’s earth pressure methods, the interface friction angle is often assumed to be equal to some fraction of the soil friction angle, typically on the order  $2/3\phi$  to  $\phi$ .
- Allowable back cut angle ( $\psi$ ).
- Soil unit weight ( $\gamma_S$ ); typically between 17.2 and 20.4 kN/m<sup>3</sup> (110 and 130 pcf).
- Unit weight for rock ( $\gamma_R$ ); typically assumed to be 23.5 kN/m<sup>3</sup> (150 pcf), including void space.
- Minimum required embedment depth (D). Because passive resistance at the toe is commonly neglected for rockery design, the value of D is typically constrained by the potential for disturbance of the soil at the toe. As a minimum, D should be 300 mm (1 ft) for level toe conditions, and should extend below the depth of frost penetration or anticipated scour. For sloping toe conditions, D should be sufficient to achieve 1800 mm (6 ft) of lateral soil cover in front of the base rock.
- An upper thickness of soil (d) is often neglected at the ground surface when computing the passive resistance. The value of d is commonly assumed to be 300 mm (12 in) for level toe conditions.
- Allowable bearing pressure and estimated settlement due to the weight of the rockery. In the case where thick fills will form the foundation for the rockery, the estimated total and differential settlement due to compression of the underlying fill should also be calculated.
- Applicable surcharge pressures, including traffic surcharge, construction surcharge, or guardrails. As shown in Figure 30, surcharges will most often apply to level backslope

conditions. It is rare for surcharge loads to be placed on an “infinite” backslope behind a rockery. However, where broken backslope conditions exist, the combined influence of both the broken backslope and any upslope surcharges should be considered.

### **Lateral Earth Pressures**

In most cases, rockeries are assumed free to rotate about the base, and the earth pressure is typically computed by multiplying the lateral earth pressure coefficient for active soil conditions ( $K_A$ ) by the unit weight of the soil ( $\gamma_s$ ). Because the resulting value has the units of density (Force/Length<sup>3</sup>), it is often expressed in terms of an equivalent fluid pressure (EFP) of retained soil. A more detailed discussion of lateral earth pressures may be found in most foundation engineering textbooks, such as those by Das<sup>(14)</sup> or Bowles.<sup>(15)</sup>

For most rockeries, there are two potential sources of lateral earth pressure acting on the back of the rockery—that exerted by the retained soil, and that exerted by the crushed rock backdrain. Generally, the pressure exerted by the crushed rock backdrain is less than that exerted by the retained soil, for three reasons:

- The friction angle of the crushed rock ( $\phi_{CR}$ ) is typically on the order of 40° to 45°, and is generally much higher than the soil ( $\phi=30^\circ$  to 36°). This results in a smaller value of  $K_A$ .
- The crushed rock typically has a lower unit weight than the retained soil due to the increased void space. Typically,  $\gamma_{CR} = 16.7 \text{ kN/m}^3$  (105 pcf), and  $\gamma_s = 18.6 \text{ kN/m}^3$  (120 pcf).
- The crushed rock layer is generally relatively narrow, on the order of 300 mm (12 in) thick. As a result, the active failure wedge typically extends through the crushed rock and into the retained soil behind the crushed rock.

Consequently, for most rockery design cases, the theoretical failure plane crosses through two soil types (crushed rock and retained soil) and a compound failure wedge is developed. While it is feasible to develop closed-form equations for this condition, acceptable results can be obtained by making the simplifying assumption that the crushed rock is part of the rockery system and the

lateral earth pressure is developed solely by the retained soil. Therefore, the lateral earth pressure acts on the back of the crushed rock layer at the crushed rock/slope interface rather than the back of the rockery facing elements. Because the friction angle of the crushed rock is almost always greater than that of the retained soil, this simplifying assumption is usually conservative.

However, as demonstrated previously by example Case #4 (Figure 24), this procedure may not be conservative under all conditions. Where the soil slope is laid back at a relatively shallow inclination (greater than 2V:1H), the value of  $K_A$  computed for the soil slope would likely be lower than for a wide zone of crushed rock backfill. Under these conditions, the lateral earth pressures for both the crushed rock backfill and the soil slope should be evaluated, and the larger value should be used in the design.

The Coulomb method, which accounts for frictional interaction between the retained soil and the retaining structure, is the recommended method for determination of  $K_A$ . In this case, the “back of the retaining structure” is actually the back of the crushed rock backdrain. Because of the roughness of the crushed rock layer, the interface friction angle,  $\delta$ , is assumed equal to the friction angle of the retained soil,  $\phi$ . Where a filter or separation fabric is used between the retained soil and the crushed rock backfill, a value of  $2/3\phi$  may be more appropriate. Figure 31 presents an equation for the computation of  $K_A$  using the Coulomb method that accounts for soil strength, back cut inclination ( $\psi$ ), and ground surface inclination ( $\beta$ ).

$$K_A = \frac{\cos^2(\psi + \phi)}{\cos^2(\psi) \cdot \cos(\delta - \psi) \cdot \left[ 1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \beta)}{\cos(\delta - \psi) \cdot \cos(-\psi - \beta)}} \right]^2}$$

**Figure 31. Equation. Determination of lateral earth pressure coefficient,  $K_A$ , using the Coulomb method.**

The back cut angle ( $\psi$ ) used in Figure 31 is measured back from the slope face from the vertical. However, the slope inclination is often described by as angle measured up from horizontal, designated  $\alpha$ . The value of  $\alpha$  is equal to  $90^\circ - \psi$ .

For values of  $\psi$  greater than about  $20^\circ$  ( $\alpha$  less than about  $70^\circ$ ), the value of  $K_A$  begins to decrease rapidly and a “protecting rockery” condition may exist. Under these conditions, the lateral pressure applied by the crushed rock backfill should be checked to determine if it is more critical than the retained soil. In addition, a global slope stability analysis may be required to determine if the overall stability of the slope is acceptable.

The preceding discussion assumes that non-expansive materials are exposed in the back cut. If highly plastic native clay or expansive bedrock is present in the back cut, surcharge pressures could be imposed on the back of the rockery due to swelling caused by seasonal moisture change. Special design considerations are required under these conditions and should be evaluated by the Geotechnical Engineer.

### **Sliding Resistance**

Once the earth pressure coefficient has been determined, the horizontal forces acting on the back of the rockery due to both the retained soil ( $F_{A,H}$ ) and any surcharge loads ( $F_S$ ) can be determined. For a uniform vertical surcharge,  $q_s$ , applied to the ground surface behind the rockery, the horizontal active earth pressure imposed can usually be estimated as the quantity  $K_A \cdot q_s$ .

The standard American Association of State Highway Transportation Officials (AASHTO) traffic surcharge<sup>(16)</sup> is assumed to be equivalent to 0.6 m (2 ft) of additional soil stockpiled above the rockery. For this condition, the value of  $q_s$  is computed as  $\gamma_s(0.6 \text{ m})$ . Therefore, assuming a soil unit weight of  $20 \text{ kN/m}^3$  (125 pcf) the total traffic surcharge force per unit width ( $F_S$ ) of rockery ( $F_S$ ) can be computed as  $12K_A H \text{ kN/m}$  ( $250K_A H \text{ lb/ft}$ ).

According to Coulomb theory, the lateral earth pressure force acts at an angle  $\delta$  from the perpendicular relative to the back of the retaining surface, where  $\delta$  is defined as the interface friction angle. However, the back of the crushed rock backdrain, which is inclined at an angle  $\psi$  back from vertical, is conservatively used as the back of the retaining surface. Therefore, the inclination of the lateral earth pressure force must be adjusted to determine the correct magnitude of the horizontal component. The equation for computing the horizontal force acting on the back of the rockery appears in Figure 32.

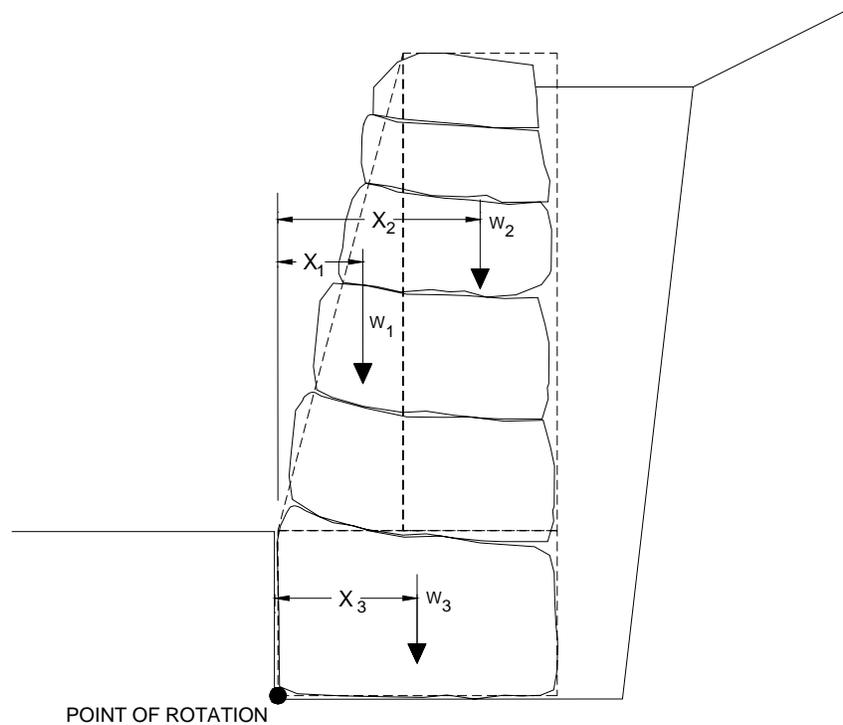
$$F_H = F_{A,H} + F_S = \frac{1}{2} \gamma_s K_A H^2 \cos(\delta - \psi) + q_s K_A H$$

**Figure 32. Equation. Horizontal force on back of rockery, equal to the sum of the lateral earth pressure and any surcharge loads.**

Rockeries generally resist sliding primarily through friction along the bottom of the base rock, which is a function of the normal force acting on the base of the rockery and the coefficient of sliding between the base rock and foundation soil. The normal force consists of the vertical component of the Coulomb active earth pressure ( $F_{A,V}$ , acting downward) and the weight of the rockery.

The weight of the rockery can be estimated by assuming certain minimum dimensions for the rockery, breaking the rockery into a few easy to define geometric shapes, assuming a unit weight for the rockery mass, and computing the total weight as the sum of each component. The unit weight of the individual, sound, intact rocks is about 25.9 kN/m<sup>3</sup> (165 pcf), which corresponds to a specific gravity of about 2.65. However, once the voids in the rockery are considered, a reasonable unit weight for a well-constructed rockery is about 23.6 kN/m<sup>3</sup> (150 pcf). The Geotechnical Engineer should evaluate the supply of locally available rocks and determine if these parameters are valid at the start of the project.

Figure 33 shows an example of a rockery that has been divided into three sections for the computation of the rockery weight. Although the lateral earth pressures are assumed to act on the back of the crushed rock backdrain behind the rockery, the weight of the crushed rock is typically not included as a resisting force. Because the crushed rock is not physically connected to the back of the rockery and the facing rocks and crushed rock interact only through frictional contact, it is not clear that the weight of the crushed rock would provided a significant resistance to movement, particularly overturning. Therefore, the weight of the crushed rock is conservatively neglected. After the design is complete, the final rockery dimensions should be checked to verify the assumed geometry and weight are correct.



**Figure 33. Graphic. Estimation of rockery weight and centroidal distances.**

The friction along the bottom of the base rock is computed by multiplying the friction factor for sliding between the rock and the foundation soil ( $\mu$ ) by the sum of the vertical forces acting on the base of the rockery. The value of  $\mu$ , which is related to the roughness of the base rock, the internal friction angle of the foundation soil ( $\phi$ ), and the degree of “nesting” or interaction between the two, should be determined by the project Geotechnical Engineer for each anticipated subgrade material. Typical values of  $\mu$  for some common materials are listed in Table 7.

The values of  $\mu$  presented in Table 7 are ultimate values; that is, they are unfactored. In the western United States, it is common practice to reduce the magnitude of the friction coefficient when passive and friction are considered concurrently because differing amounts of rockery movement are required to develop the peak values of each. The implementation of this reduction varies regionally. For the design procedures presented in this report, a factor of safety of 1.5 is recommended where passive resistance and friction are used concurrently. Where passive pressure is neglected completely, reduction of the frictional component is not required.

**Table 7. Typical friction factors for determination of FS<sub>SL</sub>.**

Base Rock Texture	Foundation Material	Estimated Ultimate Friction Factor, $\mu$
Rough	Dense, medium-grained sand $\phi=36^\circ$	0.7
Smooth, angular rocks with flat faces	Stiff silt or clay $\phi=30^\circ$	0.4
Rough	Moderately weathered bedrock $\phi=36^\circ$	0.6
Rough	300 mm thick layer of crushed rock $\phi=40^\circ$	0.8
Smooth, angular rocks with flat faces	300 mm thick layer of “foundation fill” with 100% passing 50 mm sieve, 6% maximum passing 75 $\mu\text{m}$ sieve $\phi=35^\circ$	0.7

Figure 34 presents the equation for the computation of the friction force on the base of the rockery.

$$F_{\mu} = \mu \cdot (W + F_{A,V}) = \mu \cdot \left( \sum_i W_i + \frac{1}{2} \gamma_s K_A H^2 \sin(\delta - \psi) \right)$$

**Figure 34. Equation. Computation of frictional resistance along the base of the rockery.**

The equation presented in Figure 34 is only valid for values of  $\psi \leq \delta$ . Where  $\psi$  exceeds  $\delta$ , there is a reversal of the computed direction of the vertical component of the lateral earth pressure force; that is, it acts upward instead of downward. However, under these conditions, the back-cut inclination is likely shallow enough that retention of the crushed rock backfill will control the design.

Passive resistance at the toe of the rockery can, in some circumstances, also resist horizontal sliding. However, passive resistance should be used with care, as construction practices can have a significant impact on the magnitude of the passive resistance. Unlike concrete footings,

which are typically cast against a vertical soil cut, the foundation for a rockery is generally constructed by excavating to the foundation depth and placing the base rock in the foundation excavation. The base rock may or may not be in solid contact with the face of the excavation, and significant lateral movement may be required to develop any appreciable lateral resistance. As a result, passive resistance should only be included if the project specifications include recompaction of the soil against the toe of the rockery. Because it is often difficult to enforce and/or verify uniform compaction at the toe of the rockery without full-time construction observation and testing, it is recommended passive resistance be neglected completely. In addition, passive resistance should not be used if there is a potential for soil to be removed from the toe after construction, such as in the case of scour.

If it is desired to include passive resistance, a Rankine passive pressure coefficient yields a more conservative resisting value than a Coulomb coefficient and is recommended for use. The use of a Rankine coefficient for this application also appears to be in line with current geotechnical standard of practice. Because Rankine pressures neglect the influence of friction between the soil and the rockery, the pressure acts horizontally.

Figure 35 presents the recommended equations for computation of the passive pressure at the toe of the rockery;  $\phi_F$  is the friction angle of the soil in front of the rockery, which may be different than that for the retained soil. The value  $D$  is the depth of embedment of the base rock, that is, the distance from the ground surface at the toe to the bottom of the rockery. The value  $d$  is the depth of soil to neglect when performing the passive resistance computation. Within the geotechnical community, it is common practice to neglect the upper 300 mm (12 in) of soil when calculating passive resistance, unless the soil at the toe of the rockery is protected by a slab or pavement. For rockeries founded 300 to 450 mm (12 to 18 in) below grade,  $D \approx d$ , which is yet another reason why passive resistance should often be neglected completely.

$$F_p = \frac{1}{2} \gamma_s K_p (D - d)^2, \text{ where}$$

$$K_p = \frac{\tan^2 \left( 45^\circ + \frac{\phi_F}{2} \right)}{FS}$$

**Figure 35. Equation. Evaluation of passive resistance at the rockery toe.**

The factor of safety used in the definition of  $K_P$  is included to limit the amount of rotation and deflection required to develop the maximum value of passive resistance. A factor of safety of 1.5 is typically assumed for this application. However, considering the difficulty in uniformly compacting soil against the uneven face of a rockery, a larger factor of safety may be justified to limit the amount of resistance that is relied upon. The actual factor of safety used should be selected by the Geotechnical Engineer based on the tolerable amount of horizontal movement by the rockery. Additional discussions regarding wall rotation and passive resistance are presented in Das,<sup>(14)</sup> Bowles,<sup>(15)</sup> and NAVFAC.<sup>(17)</sup>

Once the forces described above have been determined, the factor of safety against sliding can then be defined as the ratio of the resisting forces ( $F_\mu, F_P$ ) to the driving forces ( $F_H$ ), as shown in Figure 36. A minimum factor of safety of 1.5 should be used to check against sliding.

$$FS_{SL} = \frac{F_\mu + F_P}{F_H}$$

**Figure 36. Equation. Expression for factor of safety against sliding ( $FS_{SL}$ ).**

### **Overturning**

In addition to the tendency to cause translational movement, the forces acting horizontally behind the rockery will also tend to cause it to tip forward about its toe. These forces include the horizontal component of the lateral earth pressure ( $F_{A,H}$ ) and the additional horizontal pressure due to a vertical surface surcharge ( $F_S$ ). The overturning moments caused by these forces are counterbalanced by resisting moments due to the weight of the rockery ( $W$ ), the vertical component of the lateral earth pressure ( $F_{A,V}$ ), and the passive resistance at the toe of the rockery ( $F_P$ ). The overturning and resisting moments are computed by summing moments about the toe of the rockery as illustrated in Figures 37 and 38. The total resisting moment due to the weight of the rockery is computed for each component of the rockery weight ( $W_i$ ) multiplied by the horizontal distance from the centroid of each rockery segment to the toe of the rockery ( $x_i$ ). A graphical example showing the locations of the centroidal distances was previously presented as Figure 33. The factor of safety is computed as shown in Figure 39.

$$M_o = \frac{1}{2} \gamma_s K_A H^2 \cos(\delta - \psi) \left( \frac{H}{3} \right) + q_s K_A H \left( \frac{H}{2} \right)$$

**Figure 37. Equation. Determination of overturning moments about the toe of the rockery.**

$$M_r = \sum_i W_i X_i + \frac{1}{2} \gamma_s K_A H^2 \sin(\delta - \psi) \left( \frac{H}{3} \cdot \tan(\psi) + B \right) + \frac{1}{2} \gamma_s K_P (D - d)^2 \left( \frac{D - d}{3} \right)$$

**Figure 38. Equation. Determination of resisting moments about the toe of the rockery**

$$FS_{OT} = \frac{M_r}{M_o}$$

**Figure 39. Equation. Determination of factor of safety against overturning.**

A minimum factor of safety of 2.0 should be used for  $FS_{OT}$ . Although the resisting force applied by passive resistance at the toe is shown in Figure 38, it is recommended that passive pressure be neglected for the reasons discussed previously.

The factor of safety against internal (inter-rock) overturning can normally be addressed through proper specification and construction practices, because the coefficient of sliding and section properties within the rockery itself typically produce internal stability results that exceed those for external rockery stability. As a practical note, the internal stability requirements are generally met if the first point of contact between upper and lower rocks is no more than 150 mm (6 in) from the face of the rockery.

However, if large surcharge loads are applied above the rockery, inter-rock overturning or sliding should be checked, particularly near the top of the rockery. Internal overturning is evaluated in the same manner as external overturning, except moments are summed about the base of the section of rockery under consideration, rather than about the base of the entire rockery cross-section. Internal overturning stability at any point in the rockery can be computed using the equations presented in Figures 40 and 41. For these equations, it is assumed:

- Overturning of upper rocks over lower rocks is checked at a distance H' measured up from the base of the rockery.
- The base width of the upper rock (about which overturning is computed) is B'.
- The distance from the face of the rockery to the outermost bearing point is equal to x'. This value is typically assumed to be about 150 mm (6 in), but may be larger, particularly if an existing rockery is being analyzed.

The geometry is defined graphically in Figure 42. As with external overturning, a minimum factor of safety ( $FS_{OT\_int}$ ) of 2.0 should be used to check the internal overturning analysis.

$$M_{o\_int} = \frac{1}{2} \gamma_s K_A (H - H')^2 \cos(\delta - \psi) \left( \frac{H - H'}{3} \right) + q_s K_A (H - H') \left( \frac{H - H'}{2} \right)$$

**Figure 40. Equation. Calculation of internal overturning moment at a distance H' from the base of the rockery.**

$$M_{r\_int} = \sum_i W_{i\_top} (x_i - x') + \frac{1}{2} \gamma_s K_A (H - H')^2 \sin(\delta - \psi) \left( \frac{H - H'}{3} \cdot \tan(\psi) + B' \right)$$

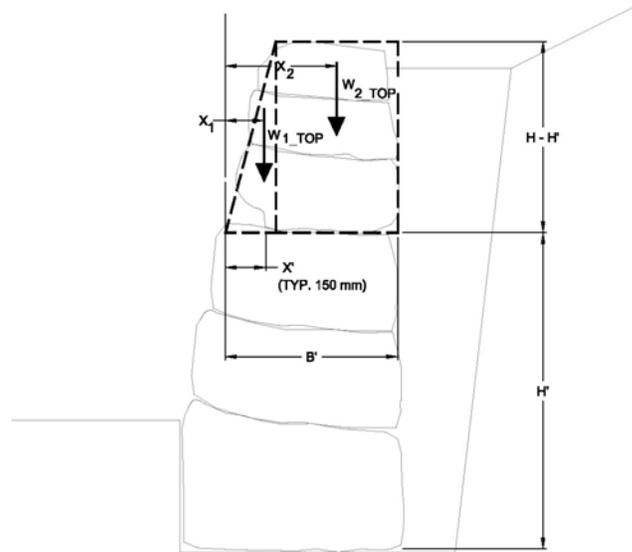
**Figure 41. Equation. Calculation of internal resisting moment at a distance H' from the base of the rockery, with outermost bearing distance x' from the face of rockery.**

Inter-rock sliding is similar to external sliding, except that the total weight is only computed for the rocks above the place of sliding. For rock-to-rock friction, a nominal value for  $\mu$  of 0.55 is recommended.<sup>(Hemphill, 1990)</sup> Higher values can be used where additional data, such as high rock roughness or laboratory testing, indicates a higher value is warranted.

### Bearing Capacity

The final aspect of static design to be checked is the bearing capacity of the foundation soils. Explanation of bearing capacity theory and determination of the maximum allowable bearing pressure for a given foundation soil is beyond the scope of this report. For highway projects, the allowable bearing capacity should be determined by the Geotechnical Engineer in accordance

with Section 4.4 of the AASHTO *Standard Specifications for Highway Bridges, 17<sup>th</sup> Edition – 2002*.<sup>(16)</sup> Additional information on soil bearing capacity can also be referenced in Das,<sup>(14)</sup> Bowles,<sup>(15)</sup> or NAVFAC.<sup>(17)</sup>



**Figure 42. Graphic. Geometric relationships for determination of internal stability.**

For a typical rockery, the average (uniform) bearing pressure exerted on the foundation soils ( $q$ ) can nominally be computed as the sum of vertical forces ( $W + F_{A,V}$ ) divided by the base width,  $B$ . In reality, the application of a moment to the rockery by the retained soil results in a non-uniform pressure distribution in which the bearing pressure at the toe of the rockery is higher than at the heel. It has been widely reported for gravity retaining walls<sup>(14,15,17)</sup> that the resultant of all vertical forces ( $W + F_{A,V}$ ) acts at a particular distance from the toe of the wall, resulting in load eccentricity about the base of the footing. This concept can be easily extrapolated to rockeries. The distance from the point of action of the vertical resultant force to the toe of the rockery can be determined by dividing the sum of the net moments taken about the toe by the resultant force. The distance from the point of action of the resultant force to the center of the base rock is then defined as the eccentricity,  $e$ . The equation for determination of the eccentricity is presented in Figure 43.

$$e = \frac{B}{2} - \frac{M_x - M_o}{W + \frac{1}{2} \gamma_s K_A H^2 \sin(\delta - \psi)}$$

**Figure 43. Equation. Determination of eccentricity, e, about the center of a base rock of width B.**

If the magnitude of the eccentricity places the resultant within the middle third of the base rock, the minimum pressure at the heel of the rockery (the back of the base rock) will be such that all portions of the rock are in contact with the underlying soil. If the eccentricity falls outside the middle third of the base rock, negative bearing pressures can develop. Because the soil-rock interface cannot generally support tension, the rock could lift off or lose contact with the subgrade. To verify the resultant is within the middle third of the footing, the absolute eccentricity value (e) should be less than B/6. Provided this criterion is met, the maximum pressure imposed by the rockery can be determined using the equation presented in Figure 44.

$$q_{\max} = \frac{W + \frac{1}{2} \gamma_s K_A H^2 \sin(\delta - \psi)}{B} \cdot \left( 1 + \frac{6e}{B} \right)$$

**Figure 44. Equation. Determination of maximum bearing pressure (q<sub>max</sub>) applied at the toe of the base rock.**

The factor of safety against bearing capacity failure is determined by dividing the allowable bearing capacity by q<sub>max</sub>. Various references<sup>(15,16,17)</sup> suggest factors of safety for bearing capacity (FS<sub>BC</sub>) in the range of 2 to 3. A factor of safety of 2.5 is commonly used for MSE walls with flexible facings, and a value as low as 2.0 can be used if settlements are checked and are tolerable. Depending on soil conditions and tolerance for settlement, it is recommended a factor of safety between 2 and 3 be used based on the judgment of the Geotechnical Engineer.

Typically, vertical surcharges at the ground surface are located behind the back face of the rockery, and, therefore, q<sub>s</sub> is not included in the calculation of q<sub>max</sub>. However, if an anticipated surcharge, such as a construction stockpile, will act on the top of the rockery, the surcharge load should be added to the computed value of q<sub>max</sub> when determining the factor of safety against

bearing capacity failure. In the case of a construction surcharge, the use of a reduced  $FS_{BC}$  for short-term (temporary) loading conditions may be appropriate.

### **Global Stability**

In some cases, the overall rockery design may be controlled by global stability considerations. This is especially true for cuts in previously placed fills or for walls with a sloping toe condition. The purpose of a global stability analysis is to check that the rockery or retained improvements will not be damaged by a slope stability failure through or below the wall facing.

Global stability analyses can be performed using most commercially available limit equilibrium slope stability programs. Limit equilibrium analyses are performed by assuming the geometry for a potential failure plane (either a sector of a circle or a wedge-like block) and computing the ratio of the net resisting force (soil shear strength) relative to the net driving forces (soil mass, surcharge, seepage pressures, and/or seismic accelerations). This ratio is defined as the “factor of safety.” For rockery analysis, the potential failure plane may pass through the face of the wall or below and behind the facing rocks. More often than not, the critical failure plane will pass beneath the toe of the rockery.

Commercial slope stability programs use algorithms to check multiple failure plane geometries and the lowest factor of safety computed for a given combination of slope geometry and strength parameters yields the critical slip surface. For a global failure to occur, the resisting forces are less than or equal to the driving forces; that is, the factor of safety is around 1.0. For static slope stability analyses, a minimum factor of safety of 1.5 is typically considered. For highway projects, it may be feasible to lower this factor of safety to 1.3; this determination can be made on a case-by-case basis by the Geotechnical Engineer.

As part of the project geotechnical investigation, the Geotechnical Engineer should evaluate soil and rock strength parameters to be used in the global stability analysis. The Geotechnical Engineer should carefully consider the properties used for the facing rocks in the analysis. Because the facing rocks consist of discrete blocks, they are typically very strong across the width of an individual block, but relatively weak at the frictional interface between the blocks. Use of an anisotropic strength envelop, in which the rocks are modeled using frictional

parameters for near-horizontal failure planes and unconfined compressive strengths for oblique failure planes, should be considered by the Geotechnical Engineer.

If the factor of safety is less than the minimum required for static slope stability analyses, it will be necessary to lower the height of the rockery, move potential surcharges further back from the face of the wall, or reinforce the soil behind the rockery to create a MSE fill. Design considerations for MSE rockeries are discussed later in this report.

## SEISMIC DESIGN

### Governing Regulations

Many regions of the United States can be expected to experience periodic seismic ground shaking, which can impart additional seismic loads on rockeries. For roads and highway structures, the governing code regarding seismic design requirements is the *AASHTO Standard Specifications for Highway Bridges, 17<sup>th</sup> Edition – 2002*.<sup>(16)</sup> Per the AASHTO standard specifications, seismic design is required for all areas with a seismic coefficient, *A*, greater than 0.09. For those familiar with the Uniform Building Code (UBC), this is roughly equivalent to areas classified as Seismic Zones 2, 3 or 4.<sup>(8)</sup> However, the AASHTO and UBC criteria are not directly analogous, so the AASHTO criteria should be checked for each project.

The value of *A* should be interpolated from maps in Section 3.2, Division I-A of the AASHTO manual. The values read from the maps should be divided by 100, which results in a decimal percentage less than 1.0. These percentages represent probabilistic peak ground accelerations (PGAs) with a 10% probability of exceedance in 50 years, which corresponds to a return period of about 475 years. The AASHTO regulations note that if any of the following conditions exist, a more detailed seismicity evaluation may be required:

- (a) The site is located within 5 km (3 mi) of an active fault.
- (b) Long duration earthquakes are expected in the region.
- (c) The importance of the structure requires a longer recurrence interval.

For items (a) and (b), the Geotechnical Engineer should be consulted to evaluate site seismicity and determine if supplemental seismic recommendations are warranted.

The AASHTO design criteria further rank sites into a seismic performance category (SPC) of A through D. While bridge design criteria vary depending on the SPC, for the purposes of this study, the requirements that apply to rockeries are constant for categories B, C, and D. Seismic design is not required for category A.

### **Seismic Analysis**

For the most part, design of rockeries for seismic conditions is similar to the design for static conditions. In general, the driving and resisting forces acting on the rockeries are determined and the rockery is checked for acceptable factors of safety against sliding, overturning, and bearing capacity failure. However, a seismic surcharge pressure acts on the rockery during earthquake shaking and should be considered as an additional driving force. The magnitude of this pressure is a function of the PGA, the rockery and slope geometry, and the quality of the retained soil.

The impact of the ground surface acceleration on the mass of retained soil is incorporated through the use of a pseudostatic horizontal seismic coefficient,  $k_h$ . The vertical seismic coefficient,  $k_v$ , is typically assumed to be zero. The value of  $k_h$  is typically less than the site PGA, and represents the effective acceleration applied to the mass of retained soil and which causes the soil mass to move towards the rockery. This concept is analogous to the seismic coefficient used for pseudostatic slope stability analyses.

Published values of  $k_h$  range between 0.05 and 0.2, although some researchers have suggested ranges between 0.3 to 0.5 times the site PGA.<sup>(9,18)</sup> For SPC categories B, C, and D, AASHTO recommends  $k_h$  be taken as 0.5A for gravity retaining structures. This value is expected to result in lateral rockery movement of about 250A mm (10A in) for “gravity” rockeries. The value of  $k_h$  is different for rockery-faced MSE, as discussed later in this chapter.

In accordance with the procedure developed by Mononobe-Okabe, the total thrust acting on the back of the rockery during an earthquake consists of the static rockery load ( $F_A$ ) plus the seismic pressure increment ( $\Delta F_{AE}$ ).  $\Delta F_{AE}$  is difficult to compute directly, and textbooks<sup>(14,15)</sup> recommend

$\Delta F_{AE}$  be computed by subtracting  $F_A$  from the total force ( $F_{AE}$ ) acting on the rockery ( $\Delta F_{AE} = F_{AE} - F_A$ ). The equations required to compute the total thrust on the rockery ( $F_{AE}$ ) are presented in Figures 45, 46, and 47. The value of  $\Delta F_{AE}$  can also be estimated with reasonable accuracy as  $0.5(0.75k_h)\gamma_s H^2$ .<sup>(19)</sup>

$$\theta = \tan^{-1} \left( \frac{k_h}{1 - k_v} \right)$$

**Figure 45. Equation. Determination of the term  $\theta$  for computation of  $K_{AE}$  by the Mononobe-Okabe procedure.**

$$K_{AE} = \frac{\cos^2(\phi - \theta + \psi)}{\cos(\theta) \cdot \cos^2(-\psi) \cdot \cos(\delta - \psi + \theta) \cdot \left[ 1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \theta - \beta)}{\cos(\delta - \psi + \theta) \cdot \cos(\beta + \psi)}} \right]^2}$$

**Figure 46. Equation. Determination of term  $K_{AE}$  in accordance with the Mononobe-Okabe procedure.**

$$F_{AE} = \frac{1}{2} (1 - K_v) K_{AE} \gamma_s H^2$$

**Figure 47. Equation. Determination of total thrust (seismic plus static) on rockery in accordance with the Mononobe-Okabe procedure.**

In accordance with the Mononobe-Okabe procedure,  $\Delta F_{AE}$  is represented by an inverted triangular pressure distribution with the maximum value near the top of the rockery, and a resultant acting at a point  $0.6H$  above the base of the rockery. As discussed previously, the resultant for a static equivalent fluid weight soil pressure distribution acts at a point  $H/3$  above the base of the rockery. It follows, then, that the overall resultant for  $F_{AE}$  acts at a point somewhere between the resultants for the static and seismic pressure distributions. An equation for the determination of the exact line of action of  $F_{AE}$  is presented in Figure 48.

$$z = \frac{0.6H \cdot \Delta F_{AE} + F_A \left(\frac{H}{3}\right)}{F_{AE}}$$

**Figure 48. Equation. Vertical distance (z) from the base of the rockery to the point of application of  $F_{AE}$  along the back of the rockery.**

Once the seismic forces have been determined, it is necessary to check that external overturning, sliding, and bearing capacity are satisfied for seismic conditions. The equations are similar to those described previously except that the seismic pressure increment ( $\Delta F_{AE}$ ) and the inertial force on the rockery itself ( $k_h W$ ) must be included. Figures 49 through 54 present the equations for determining the seismic driving and resisting forces for overturning, sliding, and bearing capacity. For the overturning equation, the inertial force acts horizontally, and therefore, the vertical moment arm of the mass centroid ( $\bar{y}$ ) must be determined to properly estimate the moment about the base of the rockery. Alternatively, the rockery can be broken into components and individual moments for each section can be computed, as is shown previously in Figure 33.

$$M_{O,S} = M_O + \Delta F_{AE} \cos(\delta - \psi) \cdot 0.6H + k_h W \bar{y}$$

**Figure 49. Equation. Determining overturning moment for seismic conditions.**

$$M_{r,s} = M_r + \Delta F_{AE} \sin(\delta - \psi) [(0.6H) \tan \psi + B]$$

**Figure 50. Equation. Determining resisting moment for seismic conditions.**

$$F_{H,S} = F_{A,H} + F_s + \Delta F_{AE} \cos(\delta - \psi) + k_h W$$

**Figure 51. Equation. Determining horizontal driving force to check sliding for seismic conditions.**

$$F_{\mu,S} = \mu (W + F_{A,V} + \Delta F_{AE} \sin(\delta - \psi))$$

**Figure 52. Equation. Determining horizontal resisting force to check sliding for seismic conditions.**

$$e_s = \frac{B}{2} - \frac{M_{r,s} - M_{o,s}}{W + F_{AE} \sin(\delta - \psi)}$$

**Figure 53. Equation. Determining eccentricity under seismic conditions.**

$$q_{max\delta} = \frac{W + F_{AE} \sin(\delta - \psi)}{B} \cdot \left( 1 + \frac{6e_s}{B} \right)$$

**Figure 54. Equation. Determining maximum applied bearing pressure under seismic conditions.**

For seismic conditions, factors of safety of at least 1.5 and 1.125 should be used to check seismic overturning and sliding potential, respectively. For bearing capacity, the factor of safety may be taken as 75% of that used to determine the static bearing capacity.

The foregoing presents a general seismic design procedure for rockeries in accordance with AASHTO guidelines and standard design methodology. However, these analyses are likely to result in large, potentially uneconomical base widths for sites with high seismic activity and ground accelerations. One way to reduce base rock sizes in zones of high seismic activity is to use seismic analyses that account for the effects of seismic deformations in the determination of  $k_h$ , such as that developed by Richard and Elms.<sup>(10)</sup> This approach may be more appropriate for evaluating the seismic stability of gravity retaining structures such as rockeries. At the current time, however, this procedure is not reflected in current codes. An overview of the Richards and Elms approach is presented in Appendix A.

### **Seismic Slope Stability**

Wherever global stability is checked for static conditions it should also be checked for seismic conditions. A minimum factor of safety of 1.1 should be used for seismic conditions.

Depending on the results of the seismic slope stability analysis, a deformation analysis may also be required to check that estimated upslope movements are acceptable where upslope improvements exist or are planned.

For the seismic analysis, a traditional pseudostatic analysis using a horizontal seismic coefficient ( $k_h$ ) is usually appropriate. Typically,  $k_h$  is taken to be about one-third to one-half of the site PGA. Additional guidance regarding selection of  $k_h$  in regions of high seismicity is presented in California Department of Conservation, Division of Mines and Geology Special Publication 117.<sup>(18)</sup>

### **ROCKERY-FACED MSE, RSS, AND SOIL NAILS**

Up to this point, the design discussion has focused on gravity rockeries retaining soil or rock. The practical limit on gravity rockeries in fill conditions is about 3.7 m (12 ft). In cut conditions, it is recommended the maximum single-tier rockery height be limited to about 4.6 m (15 ft). Rockeries exceeding these heights often require a backfill reinforced with geosynthetics to create an MSE fill or that the back cut be stabilized using a permanent retaining system, such as soil nails. Alternatively, rockeries taller than 4.6 m (15 ft) can be separated into two or more tiered rockeries.

The MSE fills may be near vertical (an MSE wall) or laid back (a reinforced soil slope, RSS). In the case of an MSE wall, the rockery would serve primarily as a protecting rockery, but would look similar to a retaining rockery. For the RSS condition, the rockery would clearly serve as a protecting rockery.

A design and construction guidelines manual for MSE and RSS, titled *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines*, has already been published by the FHWA.<sup>(20)</sup> Therefore, recommendations regarding design and construction of these reinforced soil structures will not be repeated in this study and the reader is urged to review the MSE manual for detailed design guidelines. However, it should be noted that for seismic design of a rockery-faced MSE, the computation of  $k_h$  follows design guidelines for MSE walls and differs from that for gravity walls. Specifically, the value of  $k_h$  is amplified to act at the center of mass of the facing/reinforcement and will differ from the value discussed for gravity walls above.

Because MSE structures are designed for external stability, generally without regard for the facing being used, the rockery facing provides an attractive aesthetic treatment and mainly serves as an erosion control measure. Therefore, detailed analysis and design is generally not required

for the facing rocks. Typically, a prescribed base width of  $0.3H$  to  $0.5H$  is used, with the remainder of the rockery geometry based on the prescribed face batter and top rock width.

Although RSS structures are generally constructed in an oversteepened condition, the combination of the back slope and soil reinforcement results in relatively low lateral earth pressures being applied to the back of the rockery facing. Thus, smaller rocks are generally required for this application. The methods proposed by Gray & Sotir<sup>(4)</sup> are appropriate for design of the rockery facing under these conditions.

When MSE or RSS rockeries are constructed, global stability is an important consideration. Often, the length of the reinforcing geosynthetic is governed by global stability. Therefore, the slope stability programs used to evaluate global stability of MSE rockeries should have the capability to model geosynthetic reinforcement. The reinforcement strength used in the program should be the allowable tensile strength ( $T_a$ ) rather than the Long Term Design Strength (LTDS). The LTDS is computed by dividing the ultimate tensile strength of the geosynthetic reinforcement by reduction factors to account for long-term creep, installation damage, and durability/aging. To compute  $T_a$ , the LTDS should be divided by an appropriate factor of safety, typically assumed to be 1.5.

The above discussions apply for any case in which rockery is used as a facing element rather than a structural element. A similar example is that of a soil nail wall with rockery facing, as shown in Figure 21.

Another example would be the case of a hybrid system, such as a Shored Mechanically Stabilized Earth (SMSE) wall. For this system, the slope is generally retained using a permanent shoring system, such as soil nails. Then, a narrow fill is placed in front of the shoring system and facing with a segmental facing element, such as rockery. This type of system is advantageous when adding lanes to the outboard face of an existing slope.

The narrow wedge of fill placed in front of the soil nail wall applies some lateral earth pressure to the back of the facing element, which in this case would be a rockery. Thus, the required rockery design would be somewhat smaller than for the retaining rockery but larger than for a protecting rockery. In some cases, short geosynthetic reinforcement is also required within the

narrow wedge of fill. Although further discussion is outside the purview of this report, additional recommendations for SMSE design can be obtained from the FHWA *Shored Mechanically Stabilized Earth (SMSE) Wall Systems Design Guidelines* manual.<sup>(21)</sup>

### **TIERED ROCKERIES**

In some cases, it is necessary to tier rockeries for design or aesthetic reasons, or to reduce single-tier heights for rockeries that would otherwise exceed 4.6 m (15 ft).<sup>(5; SAGE, unpublished)</sup> While rockeries taller than 4.6 m (15 ft) have been constructed the Pacific Northwest, it is more common to tier rockeries exceeding this height. In many cases, the use of tiers is also more visually appealing and less imposing than a single, tall rockery.

Design concepts for tiered rockeries are similar to those previously discussed for gravity rockeries or rockery-faced MSE. For the lower tier, the design must include any surcharge loads imposed by the upper-tier rockery. The surcharge load can be estimated using chart solutions, elastic half-space estimates, or commercially available computer programs. For fill conditions, the lower tier often consists of an MSE rockery unless relatively short heights are constructed.

Assuming the upper rockery is set back at least 1.6 m (6 ft) from the back of the lower tier rockery, it can generally be designed as a single-tier rockery; that is, the lower tier does not materially impact the design of the upper tier rockery. However, estimated settlements should be calculated, particularly if thick fills are constructed beneath the upper tier. In addition, the global stability analyses performed for the tiered system should consider the geometric relationship between the rockery tiers; namely, they should be analyzed as a group. It may be necessary to reinforce the upper tier with geogrid in order to satisfy global stability requirements.

### **ROCKERY LAYOUT**

One important consideration for the construction of a rockery the Design Engineer can easily overlook is the space required to construct the rockery. Because the base rocks can be on the order of 0.6 to 2.5 m (2 to 8 ft) wide, rockeries are not well suited for narrow spaces or roadway shoulders that are tightly constrained in the lateral direction. The grading and layout plans should show, to scale, the anticipated width of the rockery facing and the relationship to the overall site geometry.

Another important consideration is the potential to form “ski slopes” or “launch ramps” from the constructed rockery. This condition occurs when a rockery is constructed with a sloping toe and improvements are proposed at the toe of the slope. If a cap rock were to become dislodged, either by acts of nature or acts of man, the rock could roll down the slope and damage downslope improvements. Where this condition exists, the Design Engineer should consider an alternate site layout that reduces the potential for dislodged rocks to roll downslope. One example would be to construct a wide, flat bench in front of the rockery and prevent rocks from rolling downhill. If there is insufficient space for a flat bench, then it may be necessary to specify that all cap rocks be grouted in place to prevent them from becoming dislodged.

**SUMMARY**

The design of a rockery that resists static and seismic earth pressures and lateral pressure surcharges is analogous to the design of a concrete gravity retaining wall. Gravity rockeries can generally be constructed up to 3.7 m (12 ft) tall in fill conditions without geogrid reinforcement, and up to 4.6 m (15 ft) as a facing for an MSE fill. Regardless, a maximum single-tier height of 4.6 m (15 ft) should be used in cut or fill conditions. The lateral pressures acting on the back of the rockery should be determined, and the rockery checked for an adequate factor of safety against sliding, overturning, and bearing capacity failure. The factors of safety presented in Table 8 are recommended when performing rockery analyses.

**Table 8. Recommended factors of safety for static and seismic rockery design.**

<b>Mode of Failure</b>	<b>Static Factors of Safety</b>	<b>Seismic Factor of Safety</b>
Sliding	1.5	1.1
Overturning	2.0	1.5
Bearing	2.0 to 3.0	1.5 to 2.0
Global stability	1.3 (Highways) 1.5 (Other sites)	1.1 <sup>(a)</sup>

Notes: <sup>(a)</sup> A deformation analysis may also be required.



**CHAPTER 5 – ROCKERY CONSTRUCTION GUIDELINES**

For most civil works, the performance of a structure is directly related to the quality of construction. For a rockery, this concept is magnified several times by the fact that rockeries are constructed from irregularly shaped, naturally occurring materials. Unlike timber, steel or concrete retaining structures, for which the construction material properties are relatively well-defined and well-controlled, rockeries are constructed of variably shaped rocks, no two of which are identical. Therefore, the skill of the contractor constructing the rockery has a large impact on the overall performance, including:

- The capability of the rockery to act as a single mass.
- Individual rock overturning and sliding resistance.
- Overall rockery appearance.

As a result, careful observation during construction and quality control procedures are required to verify that the rockery is constructed per the plans and specifications, and that the actual ground conditions encountered are consistent with the design assumptions.

**EXCAVATION**

Where rockeries are constructed, cuts generally consist of excavation of the base rock subgrade (foundation) and the back cut. In native cut situations, the back cut will be made in native soil or bedrock. For fill situations, the back cut will consist of engineered fill. To ensure that the back cut is within adequately compacted engineered fill, the fill should generally be overbuilt several feet prior to rockery construction and then trimmed back to the desired back cut location.

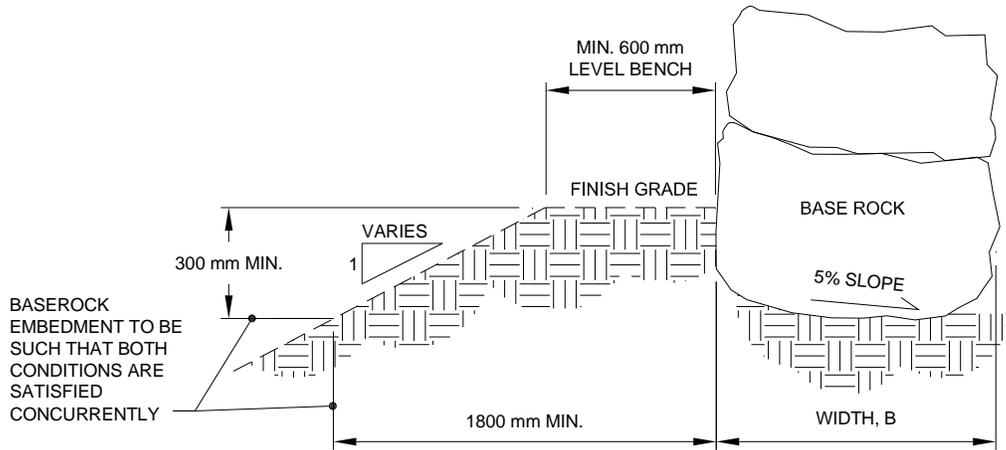
In some instances, the site conditions are such that it is more practical to place the retained fill concurrently with the facing rocks. One such example would be a site with a slope below the toe of the rockery. When the fill and rockery are constructed concurrently, the width of the gravel drainage layer should be increased to 600 mm (24 in) because of the difficulty in obtaining adequate compaction at the outer face of the fill.

### **Foundation Excavation**

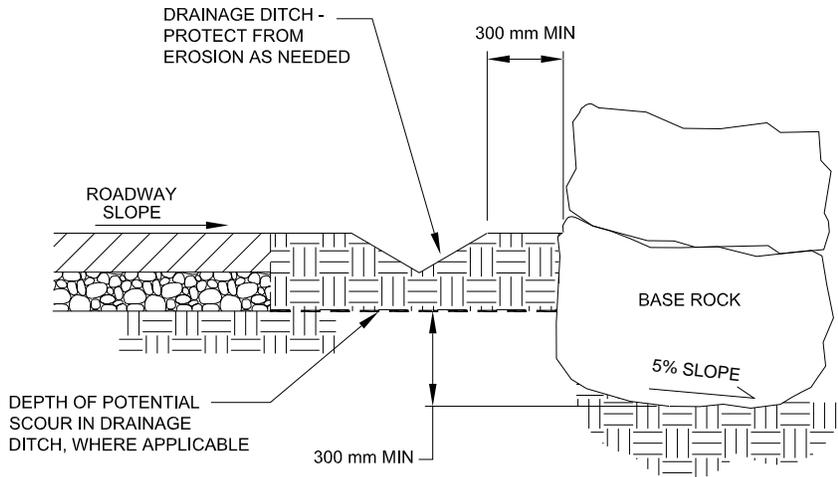
The foundation excavation should be sufficiently wide to permit placement of the specified leveling course. The leveling course can consist of at least 300 mm (12 in) of crushed, screened, 100- to 150-mm (4- to 6-in) backdrain rock. Alternatively, the leveling pad can consist of *Foundation Fill* meeting the requirements of the *Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects (FP-03), Metric Units, U.S. Department of Transportation, Federal Highway Administration, Federal Lands Highway, Section 704.01, Foundation Fill*. The base of the excavation should be level or inclined back slightly (on the order of 5%) into the slope, as shown in Figure 55. The foundation excavation should never slope forward towards the face of the rockery, as this could lead to rockery instability.

The depth of the foundation excavation will vary depending on the conditions at the toe of the rockery. For level toe conditions, nominal embedment of 300 mm (12 in) is generally sufficient unless frost considerations apply, in which case the rockery should be founded below the zone of frost heave. For sloping toe conditions, the embedment should be such that at least 1800 mm (6 ft) of soil is present horizontally in front of the rockery. This requirement provides protection against shallow erosion or other disturbances that could remove the soil at the toe of the rockery or possibly undermine the rockery. An example of the necessary cover at the toe of the rockery is shown in Figure 55. The Geotechnical Engineer should perform a slope stability analysis, particularly where toe conditions are steeper than 1V:2H, to check that the slope below the rockery, as well as the over overall rockery system, has an acceptable factor of safety.

The minimum embedments discussed previously are prescriptive values, and larger values may be required to satisfy other engineering requirements, such as bearing capacity, global stability, scour, or frost heave. For example, where scour is a concern, the rockery should be founded at least 0.6 m (2 ft) below the estimated scour depth. In addition, the base rocks for such a rockery should be sized for a total height equal to the planned exposed height plus the estimated scour depth.



**Figure 55. Graphic. Minimum embedment required for a sloping toe condition.**



**Figure 56. Graphic. Example of embedment requirements for a rockery adjacent to a roadway with a drainage ditch subject to potential scour.**

Unless otherwise supplied by the Geotechnical Engineer, the allowable bearing pressure for the expected soil or rock conditions at the foundation subgrade should be determined in accordance with AASHTO Section 4.4 and clearly specified on the plans. If weak soils are encountered, the bearing capacity may be less than assumed during design and/or excessive settlement could result. If the soil is in a state in which compaction can be performed, the subgrade soils should be compacted until the desired density is achieved.

If the soil cannot be compacted (for example, the soil is too wet or soft), it will be necessary to overexcavate the foundation subgrade to expose competent soil or bedrock. In general, overexcavation and recompaction should be performed in accordance with FP-03, Sections 208.09(d) and 208.11. The maximum dry density and optimum moisture content of the fill should be determined in accordance with AASHTO T 180 and compacted to at least 90% of the maximum dry density.

Care should be taken when overexcavating for a rockery. Because passive resistance at the toe is normally neglected, the height retained by the rockery will increase due to the overexcavation, which could increase the base rock width. Therefore, the Geotechnical Engineer should be consulted whenever overexcavation is required to determine the appropriate mitigation measures. Mitigation measures could consist of constructing a wider rockery or stabilizing the base of the overexcavation with crushed rock and backfilling with crushed rock or recompacted soil.

Where a rockery is used in a marine environment, such as for a bulkhead, the rockery should be protected from erosion, particularly at the ends. In the Pacific Northwest, rockeries are either constructed such that they abut existing concrete bulkheads, or they are “returned” into the site at a 90° angle to the wall face. Where a return is constructed, the foundation excavation should extend at least three times the base rock width (3B) into the site.<sup>(1)</sup> The non-woven geotextile placed behind the rockery (as discussed later in the chapter) should extend down the entire back cut and across the entire width of the foundation excavation.

Where a rockery return is constructed on a slope, the requirements for foundation preparation depend on the slope inclination. For slopes equal to or less than 1V:2H, a sloping foundation excavation can be used in lieu of a stepped foundation. The base of the excavation should be inclined no steeper than 1V:2H, and the cross slope on the base rocks should be no greater than 1V:10H. A gravel leveling pad comprised of 100- to 150-mm (4- to 6-in) crushed rock or *Foundation Fill* should be constructed along the base of the foundation excavation as shown in Figure 57.

For slopes steeper than 1V:2H, it will be necessary to step the foundation to maintain the 1V:2H maximum inclination, as shown in Figure 58. The subgrade should be flat for at least a base rock width on either side of the step.

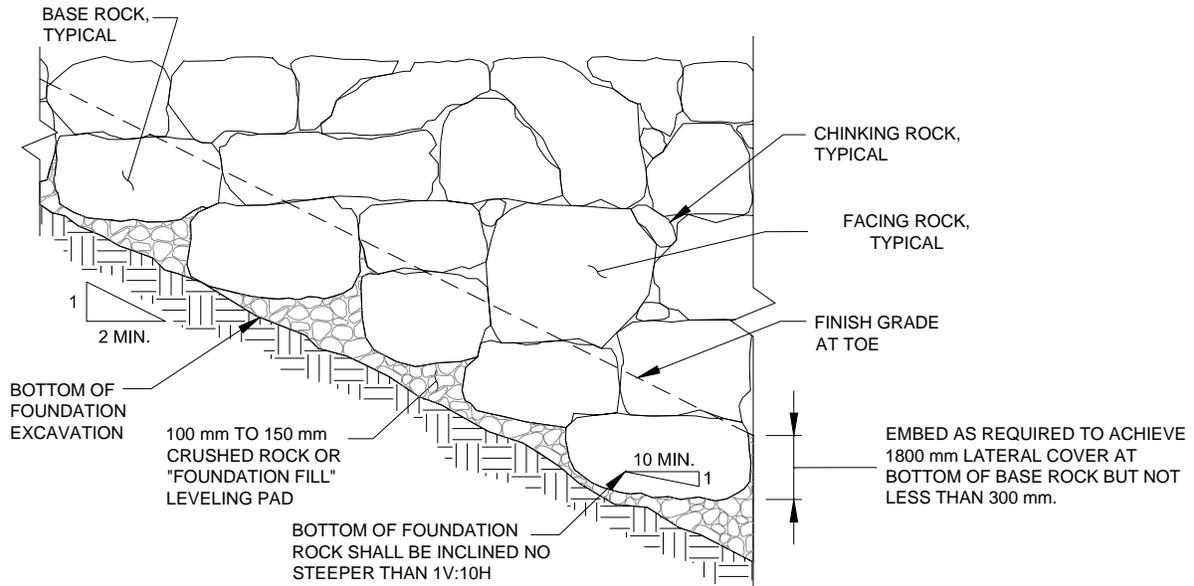


Figure 57. Graphic. Typical gravel leveling pad beneath base rocks (partial elevation).

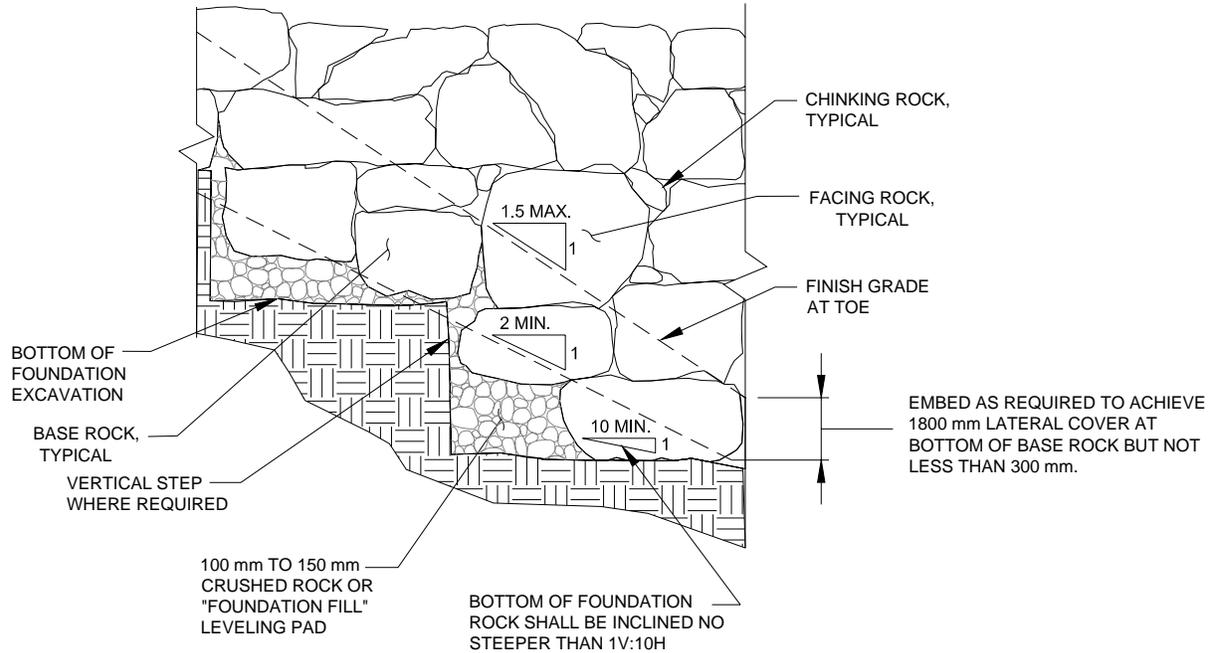


Figure 58. Graphic. Typical step in rockery foundation (partial elevation).

## **Back Cut Excavation in Soil**

### ***Back Cut Inclination***

As with any gravity type retaining structure, back cuts are required for construction of rockeries in cut situations. Back cuts may also be required in fill situations if the fill is overbuilt and cut back before the rockery is constructed. The back cut inclination is a specified input parameter in the design procedure discussed in Chapter 4, and, therefore, the assumed inclination of the back cut should be clearly stated on the plans.

Because the allowable back cut inclination is an input value required by the Design Engineer, the Geotechnical Engineer should provide a value that is consistent with the expected soil and rock conditions as well as recognized safety regulations, such as OSHA. If the back cut must be laid back during construction due to slumping or raveling or to comply with safety regulations, changes in the lateral earth pressure on the rockery could occur, and these changes may be conservative or unconservative.

For example, if the back cut is laid back at a shallower angle than anticipated by the Geotechnical Engineer, a larger volume of crushed rock will be required to fill the space behind the rockery. As discussed in Chapter 3, this generally results in a reduction of the lateral earth pressure imposed on the rockery, which would be conservative. However, if relatively low lateral earth pressures were used during design, such as for a cut in bedrock, the lateral earth pressure imposed by the crushed rock may be higher than assumed during design, which would be an unconservative change. As a result, an earth pressure less than that for the crushed rock should never be used because there will always be a crushed rock drainage layer that will encompass a portion of the Coulomb failure wedge.

### ***Back Cut Stability***

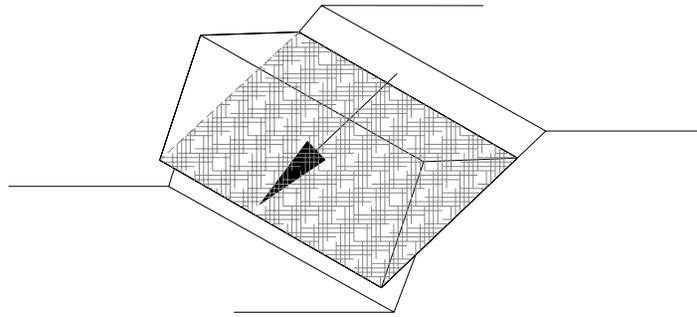
Where cuts are to be made in native soil, fill, or rock, the temporary stability of the back cut during construction should be evaluated by the Geotechnical Engineer. Although some cuts may be initially stable, they may become destabilized over time due to loss of moisture, desiccation cracking, or equipment vibrations.

Where potentially unstable soils are expected in the back cut, it made be necessary for construction to be performed in stages such that the length of time the cut is exposed is minimized. The allowable duration of exposure will vary on a case-by-case basis. In the case of stable soils, it may be possible to leave the cut exposed for weeks or months, whereas cuts in unstable soils may need to be limited to only several days of exposure. Constructing a rockery in phases, however, reduces the contractor's efficiency, and, therefore, can increase the rockery cost. Thus, phased construction should not be proposed unless warranted.

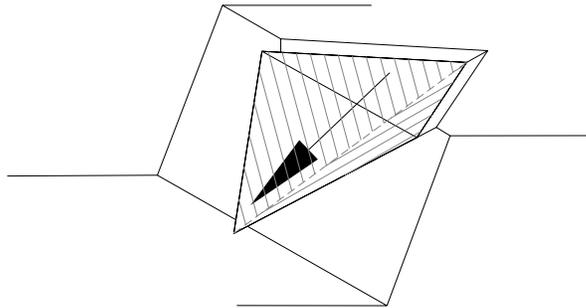
Where back cuts are made in bedrock, care should be taken to recognize potentially adverse or unstable rock conditions that may present a worker safety hazard during construction and/or could result in additional surcharge loading after the rockery has been completed. Structural discontinuities (i.e., bedding, joints, foliation, faults, etc.) present in the bedrock may be oriented such that failure could occur during or after excavation of the back cut. Two common modes of failure, planar and wedge-type failures, are shown in Figures 59 and 60. Where unstable rock conditions are recognized, a Geotechnical Engineer or Engineering Geologist should evaluate the cut and estimate the increased surcharge loads from potential failures. The Geotechnical Engineer should also review and, if necessary, revise the design recommendations to account for the increased loading conditions. Alternatively, the back cut can be laid back to a stable configuration and the resulting excavation backfilled with crushed, screened, 100- to 150-mm (4- to 6-inch) crushed rock, provided the design is checked for this condition as discussed previously.

### **QUALITY OF BASE AND FACING ROCKS**

The rocks used to construct the rockery can be obtained on-site (for example, from blasting or excavation) or imported from a quarry. The most critical physical traits of the rocks must be that they are hard, angular, and durable and will resist physical, climatic, and chemical decomposition. Rock decomposition could lead to shifting, settlement, or loss of contact between rocks. Rocks should be roughly rectangular, tabular, or cubic in shape; rounded rocks and cobbles should not be used. The rocks should consist of intact blocks without open fractures, foliation, or other planes of weakness, particularly in climatic zones prone to significant freeze-thaw conditions.



**Figure 59. Graphic. Typical planar failure on an adversely oriented discontinuity that daylights in the back cut.**



**Figure 60. Graphic. Typical wedge failure on two intersecting discontinuities with a line of intersection that daylights in the back cut.**

Many laboratory tests exist for determining rock soundness. Conveniently, rock quality designations and tests for riprap, which must also be hard, angular, and resistant to weathering, are generally also suitable for rockeries. Therefore, rocks used for constructing rockeries can generally be specified in accordance with FP-03, Section 705.02, Riprap Rock, which includes the following minimum properties:

- Apparent specific gravity  $\geq 2.5$  per AASHTO T 85.
- Absorption  $\leq 4.2\%$  per AASHTO T 85.
- Coarse durability index  $\geq 50$  per AASHTO T 210.

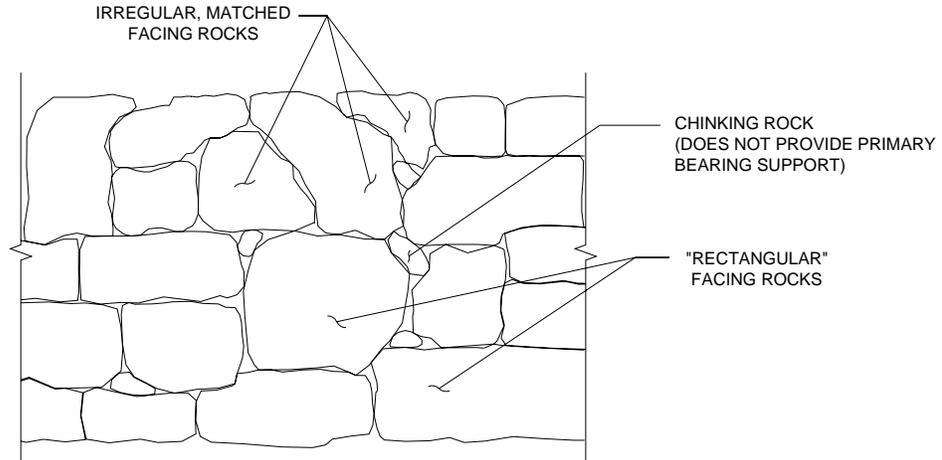
In addition to the requirements of Section 705.02, rocks used for rockeries should also meet the following minimum requirements:

- Less than 40% loss when tested in the Los Angeles machine at 500 revolutions per AASHTO T 96.
- Less than 10% loss when tested with sodium sulfate (5 cycles) per AASHTO T 104 (rocks exposed to salt water).
- Less than 10% loss when tested for freezing and thawing (12 cycles) per AASHTO T 103.

The additional requirements are based on other commonly referenced riprap standards, including the California Department of Transportation (CalTrans) Standard Specification 72–Slope Protection, Section 2.02,<sup>(12)</sup> and FHWA Hydraulic Engineering Circular 11 (HEC 11), Appendix A–Suggested Specifications, Section 7.1.2a.<sup>(22)</sup>

As discussed previously, rocks used to construct a rockery should be generally rectangular, tabular, or cubic in shape. Rocks that are triangular in shape can be difficult to stack in a stable configuration. However, it may be difficult to obtain a significant number of ideal, rectangular rocks. If irregular rocks with steeply pitched edges must be used, they should be matched with other irregular rocks such that generally level bearing surfaces with a gradual backward tilt into the back cut are formed. Examples of rectangular and matched irregular rocks are shown in Figure 61.

In general, rounded rocks, such as cobbles or river rocks, should not be used. The rounded nature of the rocks reduces the potential for interlocking and generally results in a less stable structure. Where protecting rockeries are constructed against relatively flat slopes, it may be feasible to use rounded rocks if that is the only locally available material. However, the use of rounded rocks should be avoided and the acceptability of using rounded rocks should be determined by either the Design Engineer or Geotechnical Engineer on a case-by-case basis.



**Figure 61. Graphic. “Rectangular” and “matched irregular” facing rocks (partial elevation).**

## ROCK PLACEMENT

### Equipment

Proper placement of the rocks comprising the rockery requires skill and experience because of the irregular and non-uniform nature of the materials involved. In many ways, a rockery is like a jigsaw puzzle. Some rocks only fit in some places and not others, and finding the proper match between rocks to form a stable structure can be a trial-and-error process even if the operator is highly experienced.

Because the rocks must be “finessed” into proper interlocking positions, the use of proper equipment for rock placement can be the difference between a successful and unsuccessful project. While the rocks for some early rockeries were moved by hand—hence the term “man rocks”—the use of modern hydraulic equipment greatly eases and speeds construction. Perhaps the most useful piece of modern equipment used for rockery construction is the hydraulic excavator with a rotating clamshell attachment, as shown in Figure 62. The clamshell allows the rock to be grasped uniformly on two sides, and the powered rotation capability allows the operator to quickly make adjustments to the rock orientation and alignment. In addition, a clamshell with rotation capability also allows one rock to be placed and replaced at multiple locations to determine the best fit without the need to move the excavator or regrasp the rock.

An excavator with a rotating clamshell should be specified in the plans, as it improves rockery construction and reduces time of installation.



**Figure 62. Photograph. Hydraulic excavator with a clamshell constructing a rockery.**

If a clamshell is not available, an excavator with a “thumb” can also be used, as shown in Figure 63. However, the capability to carefully align the rocks is much more limited, particularly without numerous small movements of the excavator. In addition, because the rocks should be placed with the long dimension into the slope, the use of a “thumb” requires the rocks to be placed from the side, rather than in front, of the rockery. In general, the use of “thumb” type equipment should not be allowed except on relatively small jobs. It should be noted that due to the nature of the thumb, the rock in Figure 63 is being placed with its largest dimension parallel to the face of the wall, rather than perpendicular, which is not preferred.

### **Base Rock Placement**

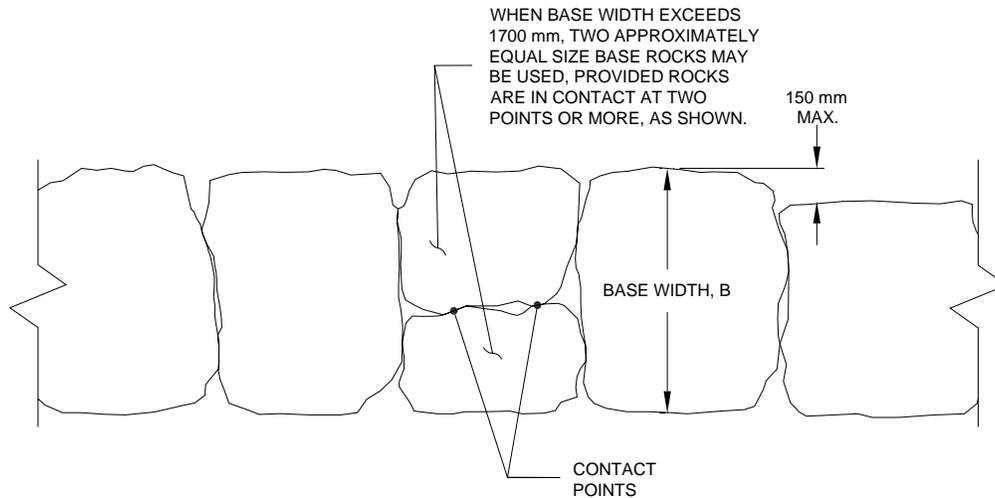
Base rocks should be placed on a properly prepared foundation excavation, as discussed previously. The minimum base rock width,  $B$ , should be specified on the plans and should be based on overall rockery height, retained soil properties, and any surcharge loads. All rocks, including the base rocks, should be placed with the longest rock dimension perpendicular to the face of the rockery. The second largest dimension should be parallel to the layout line of the rockery, and the smallest rock dimension should be its vertical dimension. The base rocks

should be placed such that the tops of the rock are sloped back at least 5% towards the back of the rockery.



**Figure 63. Photograph. Placement of facing rocks using an excavator with a “thumb.”**

The allowable tolerance for base rock widths should be 150 mm (6 in). However, two or more consecutive base rocks should be not be placed with a width less than specified on the plans, and the overall use of base rocks with a width less than B should be minimized. Once required base rock widths exceed 1700 mm (5.5 ft), locating and transporting rocks with the required minimum width becomes difficult. In these circumstances, it is reasonable to use two equally sized rocks with a total width of at least B, provided the rocks are bearing against each other in at least two locations, as shown in Figure 64. Due to limitations in current analysis techniques, facing rocks above the base rock should only consist of single rocks that provide the required rockery width.



**Figure 64. Graphic. Base rock tolerances and use of two base rocks to achieve “B” (partial plan view).**

### Facing Rock Placement

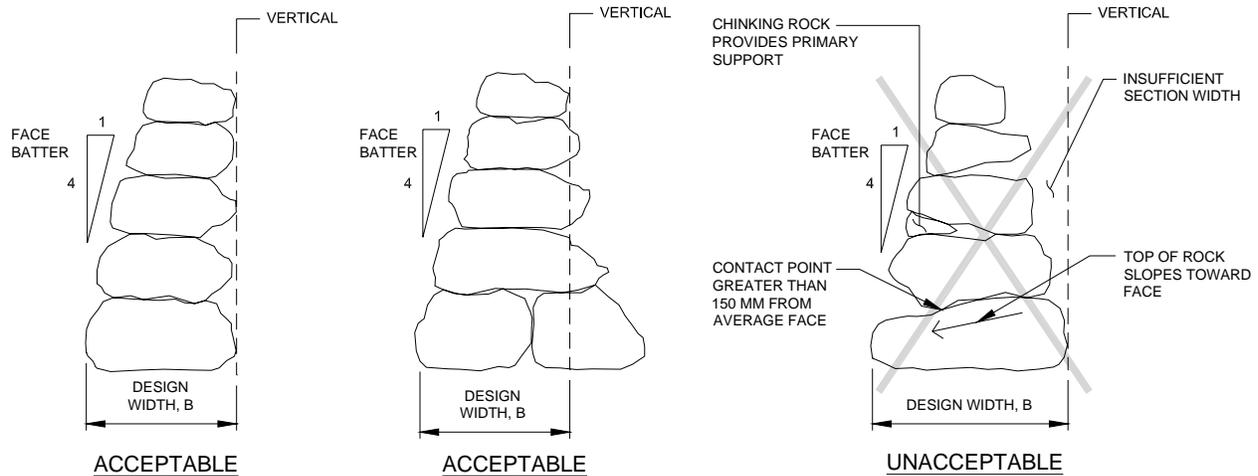
Successive lifts of facing rocks should be placed above the base rocks in accordance with the design schedule. In general, the width of successive rows of facing rocks will be determined based on the design rockery face batter, which will generally vary between 4V:1H and 6V:1H. Each rock should be placed according to the following guidelines:

- Each rock should bear on at least two other rocks.
- Each rock should have at least three bearing points—two in front and one in back.
- The front-most bearing points for each rock should be within 150 mm (6 in) of the average face of the rockery.
- The rear of the rocks should be aligned along an imaginary vertical plane. If rocks larger than the minimum specified B are used, they can extend beyond this imaginary plane provided they do not interfere with rockery drainage.
- The tops of each rock should be sloped back towards the backdrain as previously described for the base rock.

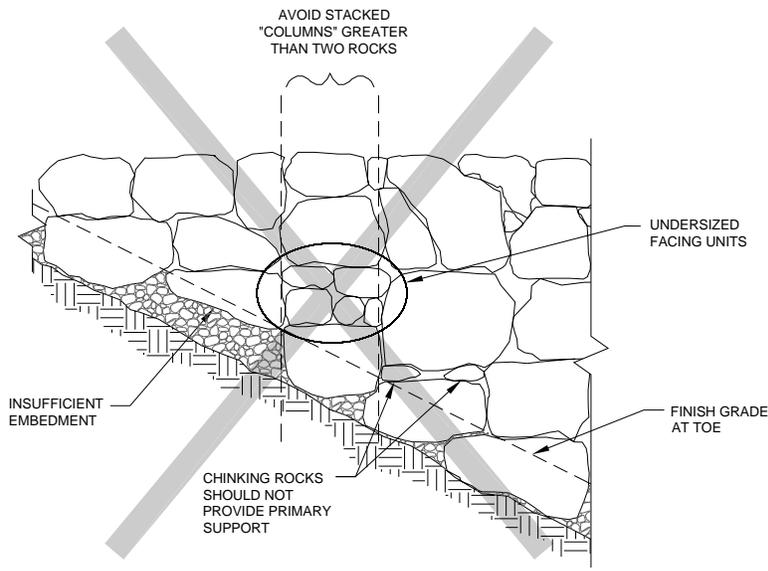
When looking at the face of the rockery, the rocks should be stacked in an approximate “running bond” pattern; that is, there should be no vertical columns of rock or continuous vertical joints running through the rockery. Continuous horizontal joints should also be avoided. The rocks should be selected and stacked such that most of the rocks in a given row are approximately the same size and gaps between rocks are minimized. Rocks with shapes that create voids with a linear dimension greater than 300 mm (12 in) shall be placed elsewhere to obtain better fit. It may be necessary to place rocks at several locations to determine the best fit for a given rock. If gaps larger than 150 mm (6 in) cannot be avoided, they should be chinked (filled) with smaller rocks. However, chinking rocks should not provide primary bearing support for overlying rocks. Chinking rocks are discussed in more detail later in this chapter.

Examples of acceptable and unacceptable alignment relative to the rockery cross section are shown in Figure 65. A typical elevation showing examples of improper rock placement is shown in Figure 66.

An example of a relatively well constructed rockery is shown in Figure 67. Although a few vertical seams can be located, the rocks are generally bearing at the proper locations and stacking in an approximate running bond pattern.



**Figure 65. Graphic. Examples of acceptable and unacceptable rockery alignment.**



**Figure 66. Graphic. Examples of improper rock placement (partial elevation).**



**Figure 67. Photograph. Example of a relatively well constructed rockery.**

Photographs of unacceptable rock bearing and chinking are presented in Figures 68 and 69. Note the wide gaps and main bearing point at the rear of the rockery. In addition, the rocks are not bearing directly on adjacent rocks.



**Figure 68. Photograph. Example of an unacceptable rock bearing.**



**Figure 69. Photograph. Example of improper rock bearing and lack of chinking.**

### **CRUSHED ROCK ZONE**

The crushed rock zone placed behind the rockery facing is an important component of the overall rockery system. In addition to providing drainage, the crushed rock improves overall rockery stability by providing a high strength material behind the facing rocks. This material helps to reduce the overall soil pressure on the rockery system.

The crushed rock should consist of 100- to 150-mm (4 to 6-in), crushed, and screened, angular rock. This material is often called “quarry spalls,” and should meet the gradation requirements presented in Table 9.

**Table 9. Gradation Requirements for crushed rock backdrain.**

<b>Sieve Size</b>	<b>Percent by Mass Passing Designated Sieve (AASHTO T 27 &amp; T 11)</b>
150 mm (6 in.)	100
100 mm (4 in.)	0.0 – 25
19.0 mm (3/4 in.)	0.0 – 15
4.75 mm (No. 4)	0.0 – 5.0
75 µm (No. 200)	0.0 – 2.0

The crushed rock should fill the void between the back cut and the rear of the facing rocks; however, it should be 300 mm (12 in) wide as a minimum. The crushed rock should be capped by at least 300 mm (12 in) of impermeable soil at the ground surface to prevent infiltration of surface water behind the rockery.

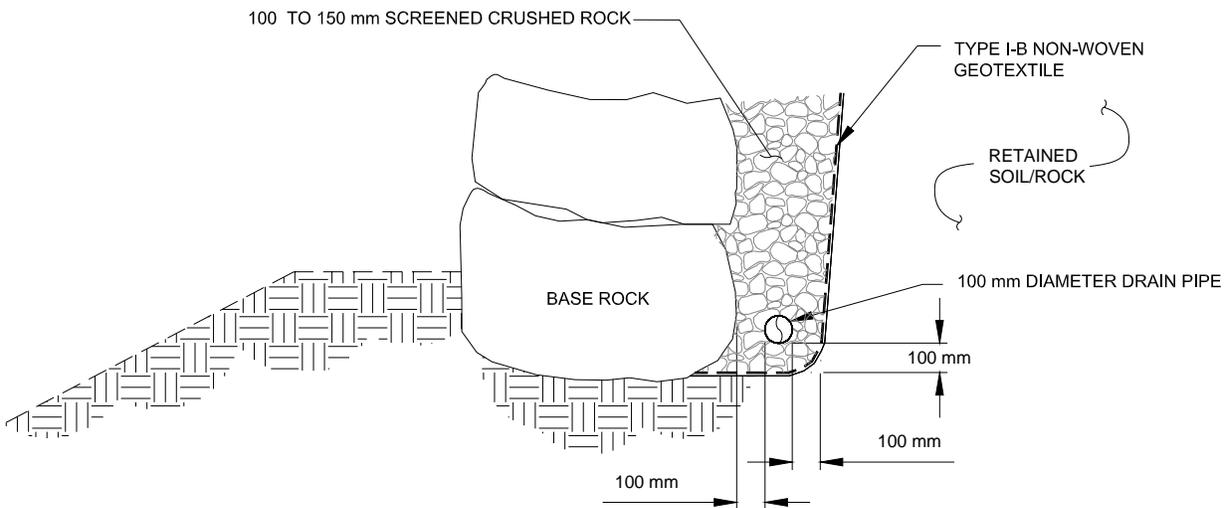
During placement of the crushed rock behind the partially completed rockery, care should be taken that the crushed rock does not spill over the top of the adjacent facing rocks. If the crushed rock is placed between the top of one facing rock and the base of the subsequent facing rock, it could form a place of weakness or prevent the facing rocks from coming into proper contact. Like chinking rocks, the crushed rock should not provide primary bearing between rocks. It is acceptable, however, for the crushed rock to fill a portion of the lateral void between adjacent facing rocks, provided the rocks are in direct contact in at least two points.

**DRAINAGE SYSTEM**

**Backdrain Construction**

As the base and facing rocks are placed, it is generally most convenient to construct the rockery backdrain and crushed rock zone concurrently. The backdrain pipe should generally consist of a 100-mm-diameter (4-in-diameter) perforated drain pipe surrounded on all sides by at least

100 mm (4 in) of screened, 100- to 150-mm (4- to 6-in), angular crushed rock unless unusual conditions exist as determined by the Geotechnical Engineer. The drain pipe should consist of either corrugated high-density polyethylene (HDPE) pipe or smooth polyvinyl chloride (PVC) pipe conforming to FP-03, Section 706.08(d) or 706.08(e). The pipe should be placed with the perforations down. A diagram of the backdrain components is presented in Figure 70 and photographs of installed crushed rock backdrains are presented in Figures 71 and 72.



**Figure 70. Graphic. Backdrain components (partial section).**



**Figure 71. Photograph. Example of crushed rock placed behind rockery.**



**Figure 72. Photograph. Placement of drain blanket and non-woven geotextile behind rockery.**

Where tiered rockeries are constructed, drainage of the upper tier is an important detailing consideration. Typically, it is difficult to outlet the backdrain directly from the upper tier because it is typically located mid-slope. In these circumstances, the perforated drain pipe can be tied into solid discharge pipes and directed downslope. The solid pipes should be sloped to the back of the lower tier, taken down the back of the lower rockery, and outlet at a similar location as the lower tier drainage. The drainage systems for the rockeries should not be interconnected, so separate discharge pipes will be required for both the upper and lower tier rockeries.

The pipes should generally be outlet at all low points of the base rock subgrade, at a spacing not to exceed 30 m (100 ft). The outlet pipes should be sloped to drain to a controlled drainage structure. The drainage structure can consist of a storm drain catch basin, storm drain pipe, or riprap-lined roadside ditch. Because of the increased use of Best Management Practice (BMP) mitigation measures to meet storm water discharge requirements, many storm drain systems are being designed to store storm water during heavy rainfall events. The discharge pipes for the backdrain system should not be connected directly to such systems unless the hydraulic grade line for the storage system is below the elevation of the drain pipe behind the rockery. The

purpose of this recommendation is to prevent storm water from flowing backwards through the system and out the face of the rockery.

For rockeries constructed on the downhill side of a roadway, it may not be feasible to connect the discharge pipe to the storm drain system and a downslope outfall may be needed. If an outfall is used, the outlet should be protected through the use of an energy dissipater and riprap splash pad. In addition, the Geotechnical Engineer should confirm that surficial slope instability and/or erosion is not a concern due to the increased moisture content caused by the outfall.

### **Non-Woven Geotextile**

The soil retained by the rockery must be protected against “piping,” particularly where seepage is observed in the back cut. “Piping” is a process through which fine-grained soil particles are transported from the soil medium into a void by subsurface water flow. Piping can result in loss of ground, ground surface settlement, and ground instability. Because of the large void ratio inherent to screened 100- to 150-mm (4- to 6-in) crushed rock, the potential for subsurface water to pipe fines into the crushed rock is relatively high. Therefore, the crushed rock should be separated from surrounding soil surfaces by a non-woven geotextile. To prevent gaps that could circumvent the filter protection and lead to piping, all seams in the geotextile should be overlapped by at least 300 mm (12 in).

The non-woven geotextile should conform to the requirements of FP-03, Section 714, Type I-B geotextile. All geotextiles used for filtration should consist of non-woven fabrics comprising long-chain synthetic polymers, at least 95% of which are polyolefin or polyester. Woven, slit-film separation fabrics should not be allowed. All geotextile rolls should be delivered to the job site wrapped in a protective plastic sheeting to protect the geotextile from damage during shipment, contamination by soil and mud during storage on-site, and from ultraviolet (UV) degradation due to sunlight.

For most rockery applications, a Type I-B geotextile should provide an acceptable level of performance and survivability. A Type I-B geotextile should have the minimum properties presented in Table 10. All values, with the exception of Apparent Opening Size (AOS), are

presented as minimum average roll values (MARV). Values of AOS are maximum average roll values.

In addition to the properties presented in Table 10, a filtration calculation should be performed to check that the geotextile is compatible with the retained soil conditions. For soils with greater than 50% of the material passing the 0.075 mm (#200) sieve, a Type I-C geotextile is likely more appropriate.

**Table 10. Physical properties requirements for non-woven geotextiles (from FP-03, Section 714, Type I-B geotextiles).**

Property	Elongation ≥ 50%
Grab Tensile Strength	700 N
Sewn Seam Strength	630 N
Tear Strength	250 N
Puncture Strength	250 N
Burst Strength	1,300 N
AOS	0.25 mm max.
Permittivity	0.2 sec <sup>-1</sup>

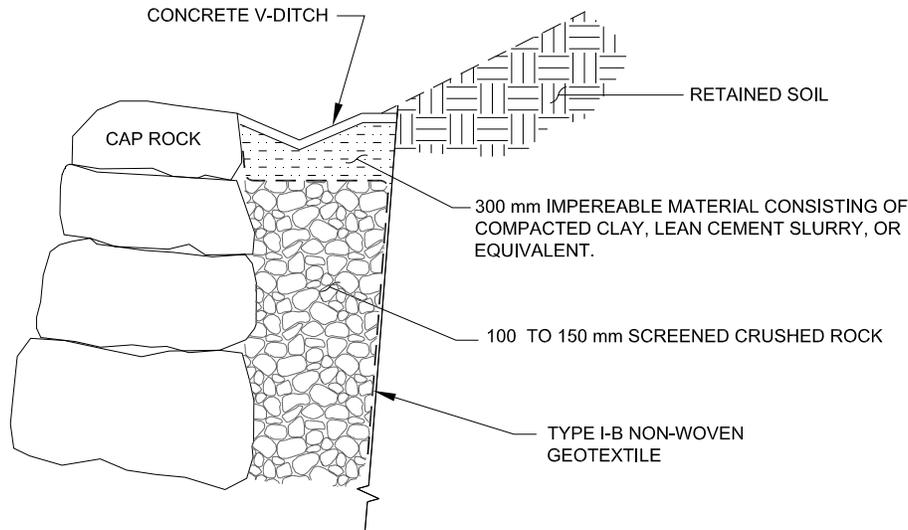
If harsh installation conditions are expected that could damage the non-woven geotextile, the Geotechnical Engineer should consider a Type IV-B non-woven geotextile for additional protection and geotextile strength. For soils with more than 50% of the material passing the 0.075 mm (#200) sieve, a Type IV-C geotextile is likely more appropriate.

**Surface Drainage**

In addition to subsurface water, surface water must also be controlled. To prevent a hydraulic connection between the rockery backdrain and surface water flows, the top of the crushed rock should be capped with at least 300 mm (12 in) of “impermeable” soil over non-woven geotextile. This soil cap can generally consist of on-site soils and should be “impermeable” to the extent that rapid infiltration of surface water cannot occur.

As with any structure that retains soil or rock, surface water should also be directed away from the rockery where possible. Where the rockery is constructed at the toe of a slope or on a slope,

a v-ditch consisting of concrete or impermeable soil should be constructed immediately behind the rockery to direct surface water to a suitable drainage outlet, as shown in Figure 73. In rare circumstances, the surface water drain system can be designed to allow “minor” surface water to enter the backdrain provided the drainage system is sized for the increased flow. Because it is difficult to limit surface water to “minor” amounts, this practice is generally not recommended.



**Figure 73. Graphic. V-Ditch and impermeable cap at top of rockery (partial section).**

### CAP ROCKS

The final rock placed at the top of the rockery is the cap rock. Because the cap rock provides a finished look to the top of the rockery, it is generally smaller and flatter than the facing rocks. To reduce the risk of disturbance to the cap rocks, such as by vandals or rock climbers, cap rocks should weigh at least 90 kg (200 lb). In addition, cap rocks should not be movable by hand. Cap rocks that do not meet these minimum requirements should be grouted or glued in place to prevent accidental dislodging.

Particular care should be taken when placing and sizing cap rocks for rockeries with toe slopes. If improvements are located at the base of the slope, dislodged rocks could roll down the slope and pose a significant hazard. Where this condition occurs, consideration should be given to securely grouting all cap rocks regardless of size.

## CHINKING ROCKS

Because of the irregular nature of the rocks, it is difficult to ensure that every rock conforms to the shape of all adjacent rocks. As a result, gaps will occur between rocks. Where these gaps exceed 150 mm (6 in), they should be filled with chinking rocks consisting of spalls from the parent (facing) rock. The purpose of the chinking rocks is to improve aesthetics and prevent the screened backdrain material from falling out through the face of the rockery. Chinking rocks should not be movable by hand, and can be grouted in place if necessary. In addition, chinking rocks are not to provide primary support for overlying rocks.

## BACKFILL

In fill conditions, the engineered fill is typically placed before rockery construction and trimmed to the desired back cut. However, in the rare cases, backfill may need to be placed behind the rockery concurrent with or after rockery construction.

If soil backfill is placed and compacted behind a rockery, full-size, ride-on compaction equipment should not be used within a distance of 0.75H or 1500 mm (5 ft), whichever is greater, from the back of the rockery because of the potential to surcharge the back of the rockery. Compaction adjacent to the rockery should be performed using hand-operated compaction equipment such as a J-tamper (“jumping jack”) or whacker plate (“turtle”). In addition, it is recommended that the backdrain width be increased to 600 mm (24 in) to provide additional space between the back of the rockery and the compaction zone.

The backfill material, whether placed before or after construction, should conform to the requirements for *Select Topping* per Section 704.08 of FP-03. Specifically, the material should meet the following requirements:

- Be free of trash, organics, or otherwise unsuitable material.
- Be non-expansive with a liquid limit (LL) less than 30.
- Be free of oversize material greater than 75 mm (3 in) in greatest dimension.
- Have less than 15% by weight passing the 75  $\mu$ m (No. 200) sieve.

In addition, the backfill material should have a friction angle ( $\phi$ ) greater than or equal to the value specified by the Geotechnical Engineer. The minimum friction angle should be shown on the plans.

Backfill should be placed and compacted in accordance with FP-03, Sections 208, *Structure Excavation and Backfill for Selected Major Structures*. Specifically, backfill should be placed and compacted in accordance with Sections 208.10 and 208.11. This requires placement of backfill in 150 mm (6 in) loose lifts. The backfill should be compacted to at least 95% of the maximum dry density, as determined by AASHTO T 99, or to at least 90% of the maximum dry density per AASHTO T 180. The Geotechnical Engineer should identify any additional site-specific compaction requirements relative to the soil types present at the project, such as moisture conditioning requirements.

Where MSE fills are constructed, they too should be overbuilt and cut back wherever possible. Geosynthetic selection, placement procedures, and backfill quality and compaction requirements for MSE fills should conform to the recommendations in the FHWA MSE and RSS manual.<sup>(20)</sup>

### **TEST SECTIONS**

A test section generally consists of constructing a limited length of rockery, typically 6 to 9 m (20 to 30 ft), so that the materials, workmanship, and appearance can be checked. Construction of test sections is typically not performed for most rockery projects. However, under certain circumstances, it may be advantageous for the contractor to install a test section prior to the start of production rockery construction. These circumstances could include:

- The contractor is new to the area and/or the contractor's final work product has not been observed previously.
- The quality, shape, or appearance (color) of the rocks to be used is suspect.
- The potential for back cut instability is suspected.

Under these conditions, the Design Engineer should specify in the contract documents that a test section shall be constructed, and that award of the remaining portion of the contract is contingent on acceptance of the completed test section by the Design Engineer.

## CHAPTER 6 – CONSTRUCTION INSPECTION GUIDELINES

Like most engineered structures, careful observation is required during rockery construction to check that the as-built structure conforms to the design intent, plans, and specifications. As for any retaining structure, geotechnical and structural inspections should be performed. The types of items to be checked during each type of inspection include:

### Geotechnical

- The foundation subgrade should be checked to verify a firm bearing layer capable of supporting the rockery is present.
- The back cut should be checked to confirm that the retained soils are consistent with the soil conditions assumed during design.

### Structural

- The items discussed in Chapter 5, including base rock width, maximum height, rock bearing, chinking, etcetera.
- The drainage system (drain pipes and drainage material) should be checked to confirm the proper materials and gradients were used and that all pipes are connected to appropriate drainage outlets.

These items have been summarized in the attached checklists and sample field inspection forms presented at the end of this chapter. Separate forms are provided for projects in which a single rockery is observed at the site and for when multiple rockeries, at different locations, are observed across the same jobsite.

For each rockery over 1800 mm (6 ft) in height, or where significant surcharges are retained, at least four inspections should be performed by a Geotechnical Engineer or qualified Soils Technician/Inspector to observe rockery construction. For rockeries more than 33 m (100 ft)

long, at least one additional inspection should be performed for each additional 33 m segment of rockery face.

Construction inspection should be performed, as a minimum, during the three key stages of construction, defined as follows:

- Initial Inspection: Observe excavation dimensions prior to base rock placement, drainage pipe and filter fabric placement, and base rock width.
- Mid-Construction Inspection: Observe rock placement, size, and batter of rockery face, placement of crushed rock drainage material, and layout of drainage outlets.
- End of Rockery Construction Inspection (contractor preparing to leave site): Observe overall rock placement for completed rockery, including chinking; confirm that exposed drainage outlets will be directed to a suitable outlet; check cap rock placement and size, and determine if grouting is required; and confirm final height is consistent with project plans. At this time, observed defects in rockery construction should be fixed while the contractor is still on-site.

The final inspection should be performed at the completion of all rockery construction and grading above and below the rockeries. The purpose of this visit is to:

- Observe final grading above and below rockery.
- Confirm the final exposed height.
- Confirm drainage outlets have been tied into a suitable collection device or otherwise properly outlet.
- Check that facing rocks or cap rocks have not been disturbed by any grading after the completion of rockery construction.

The actual number of inspections required for each project will depend on the contractor's schedule, the different stages of rockery construction that may be observed in the same visit, and

the number of rockeries being constructed simultaneously. More frequent observation should be performed if difficult or unusual construction conditions are expected.

**ROCKERY INSPECTION CHECKLIST**  
**Initial Inspection (After Excavation)**

Geotechnical

- The foundation subgrade consists of firm, undisturbed soil or rock and is consistent with the subgrade material identified on the project plans. Where weak or disturbed soil was present, it has been recompacted or overexcavated and replaced. (Do not deepen foundation without approval of the Design Engineer or Geotechnical Engineer.)
- Adverse (out-of-slope) bedding, jointing, or foliation was/was not (circle one) observed. Where adverse bedding was observed, the Geotechnical Engineer was notified.

Structural

- The minimum base rock width, B, per the project plans is 0.\_\_\_\_H.
- The excavation width is equal to the design base rock width, B, plus 300 mm (12 in), as a minimum, to accommodate the drainage blanket.
- The excavation is level or sloped toward the back of the rockery.
- The excavation depth provides for a minimum of 300 mm (12 in) of embedment and at least 1800 mm (6 ft) of lateral cover. Both criteria are met concurrently.
- Project plans do/do not (circle one) show additional requirements for scour or frost heave. Where present, these requirements have been met.
- For rockeries constructed perpendicular to slopes, the foundation excavation is sloped for inclinations 1V:2H or less. Foundation steps have been used where the excavation is sloped steeper than 1V:2H. Leveling gravel has been used where necessary.
- The back cut conforms to the minimum batter shown on the project plans.
- Base rocks are hard, angular, and durable, e.g., they ring when struck with a hammer. They are roughly rectangular, tabular, or cubic in shape. They are free of cracks, fissures, foliation, or other planes of weakness, particularly where they are subject to significant freeze-thaw cycles.
- The base rocks generally conform to the minimum width, B, and no rock widths are less than B minus 150 mm (6 in). No two consecutive rocks have a width less than B.
- Where B exceeds 1700 mm, two rocks may comprise the base rock. Where this occurs, the rocks are in contact in at least two (2) locations.
- Base rocks are inclined back into the slope at least 5%.

**ROCKERY INSPECTION CHECKLIST**  
**Mid-Construction Inspection**

- Facing rocks are hard, angular, and durable, e.g., they ring when hit with a hammer. They are roughly rectangular, tabular, or cubic in shape. They are free of cracks, fissures, foliation, or other planes of weakness, particularly where they are subject to significant freeze-thaw cycles.
- Facing rocks are inclined back into the slope at least 5%.
- Each facing rock is in contact with at least two rocks below it. Each rock has at least three (3) bearing points total—two (2) in front, one (1) in back.
- The first contact point between an upper rock and a lower rock is located within 150 mm (6 in) of the face of the rockery.
- The backs of the rocks are aligned along an imaginary vertical plane.
- There are no “columns” of rocks; i.e., no continuous vertical seams.
- There are no continuous horizontal planes in the rockery.
- The maximum face batter is between 4V:1H and 6V:1H or otherwise conforms to the project plans.
- Non-woven, Type I-B geotextile has been placed over the back cut prior to placement of crushed rock backdrain.
- Drain pipes and crushed rock backdrain installation is per the project plans. Suitable outlets have been identified on the project plans. Drain pipes have/have not (circle one) been connected.

**ROCKERY INSPECTION CHECKLIST**  
**End of Rockery Construction Inspection**

Geotechnical

- All rockery backfill has been placed, moisture conditioned, and compacted in accordance with the project plans and specifications and the Geotechnical Engineer's recommendations.

Structural

- All voids greater than 150 mm (6 in) are chinked.
- Chink rocks, where present, cannot be removed by hand.
- There is no rock spalling or soil piping through the voids in the face of the rockery.
- Cap rocks weigh at least 90 kg (200 lb). There are no loose cap rocks or rocks that can otherwise be moved by hand. Small cap rocks have been removed or grouted in place.
- The maximum height (total and exposed) of the rockery conforms to the project plans.
- Surface drainage has been directed away from the rockery or a concrete v-ditch or impermeable soil swale has been constructed above the rockery.

**ROCKERY INSPECTION CHECKLIST**  
**Final Inspection**

- Final grading above and below the rockery conforms to the rockery plans and is consistent with the grading plans.
- Exposed or total height at the completion of grading is consistent with the design height shown on the project plans.
- Drainage pipes are connected to a suitable outlet. If buried, obtain confirmation that pipes were properly connected.
- Neither facing nor cap rocks have been disturbed by grading operations after the completion of rockery construction.

<b>ROCKERY INSPECTION FORM (SINGLE ROCKERY)</b>	PROJECT #:	SHEET: ___ OF ___
	PROJECT NAME:	
	ROCKERY NO./LOCATION:	

LOCATION SKETCH

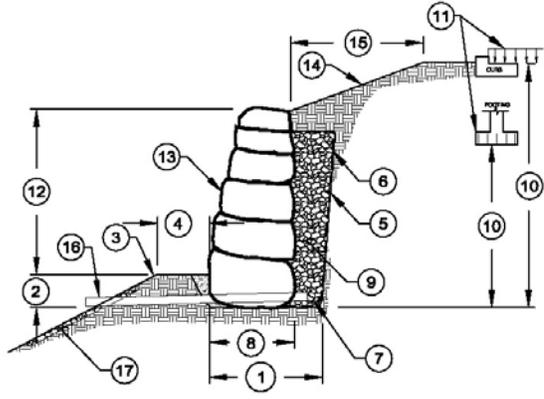
  

ITEM DESCRIPTION	CONDITION	DATE	NOTES	INIT.
1 KEY WIDTH				
2 KEY DEPTH				
3 TOE SLOPE (V:H)				
4 BENCH WIDTH				
5 CUT SLOPE BATTER (V:H)				
6 NON-WOVEN GEOTEXTILE				
7 DRAIN PIPE				
8 BASE WIDTH				
9 CRUSHED ROCK				
10 HEIGHT TO SURCHARGE				
11 SURCHARGE TYPE				
12 FINAL EXPOSED HEIGHT				
13 FACE BATTER (V:H) ROCK PLACEMENT				
14 BACKSLOPE (V:H)				
15 BROKEN BACKSLOPE DIST.				
16 DRAIN OUTLET				
17 EROSION CONTROL				

CONSTRUCTION STATUS	DATE	CONSTRUCTION STATUS	DATE

<b>ROCKERY INSPECTION FORM (MULTIPLE ROCKERIES)</b>	PROJECT #:	SHEET: ___ OF ___	
	PROJECT NAME:		
	FIELD REP:	DATE:	TIME:



**NOTES:**

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LOCATION / ROCKERY #	OBSERVATIONS	OBSERVATIONS	OBSERVATIONS	OBSERVATIONS
1 KEY WIDTH				
2 KEY DEPTH				
3 TOE SLOPE (V:H)				
4 BENCH WIDTH				
5 CUT SLOPE BATTER (V:H)				
6 NON-WOVEN GEOTEXTILE				
7 DRAIN PIPE				
8 BASE WIDTH				
9 CRUSHED ROCK				
10 HEIGHT TO SURCHARGE				
11 SURCHARGE TYPE				
12 FINAL EXPOSED HEIGHT				
13 FACE BATTER (V:H)				
14 BACKSLOPE (V:H)				
15 BROKEN BACKSLOPE DIST.				
16 DRAIN OUTLET				
17 EROSION CONTROL				

**NOTE: THIS INSPECTION FORM DOES NOT CONSTITUTE ACCEPTANCE OF ROCKERY CONSTRUCTION. RATHER, THIS FORM ONLY SHOWS OBSERVATIONS MADE FOR LATER REVIEW. WRITTEN ACCEPTANCE OF ROCKERIES BY THE DESIGN ENGINEER WILL BE PROVIDED AFTER CONSTRUCTION IS COMPLETE.**



## CHAPTER 7 – EVALUATION OF EXISTING ROCKERIES

Rockeries have been used in roadside and highway applications in the United States since the early 1900s. Therefore, it is likely that existing, historic rockeries will be encountered that support existing roadways. Recently, evaluation of existing rockeries was required for El Portal Road in Yosemite National Park (California) and for Trail Ridge Road in Rocky Mountain National Park (Colorado). It is likely the FLH or other transportation agencies will be required to evaluate the suitability of other aging rockeries for continued use in the near future. This chapter presents methods to evaluate the stability of existing historic rockeries.

### PHYSICAL AND GEOMETRIC PROPERTIES

To analyze the stability of existing rockeries, information regarding the rockery geometry and retained soil properties must be determined. The most important properties affecting rockery stability are:

- Rockery height,  $H$
- Base rock width,  $B$
- Rockery batter,  $\beta$
- Friction angle of the retained soil,  $\phi$

The rockery height and batter are relatively easy and straightforward to measure in the field. The rockery base width is more difficult to determine, as discussed in the following subsection. Determination of strength parameters for the retained soil is discussed later in this chapter.

### Rockery Layout

The parameters  $H$ ,  $\beta$ ,  $B$ , and  $\phi$  are most critical with regard to overall rockery acceptability and gross stability. However, to determine if bulging, inter-rock sliding, and inter-rock overturning resistances are acceptable, additional field observations are required. The rockery face should be carefully evaluated to check that the following criteria are satisfied:

- Each rock is in contact with at least two rocks below it.
- The first contact point between an upper rock and a lower rock is located within 150 mm (6 in) of the face of the rockery.
- There are no “columns” of rocks; i.e., no continuous vertical seams exist.
- There are no continuous horizontal planes in the rockery.
- Rocks are inclined back into the slope.
- Rocks are free of obvious signs of distress, including significant weathering, fracturing, or disintegration.
- All voids greater than 150 mm (6 in) are chinked.
- Chink rocks, where present, cannot be removed by hand.
- There are no loose cap rocks or rocks that can otherwise be moved by hand.
- There is no soil spalling or piping through the voids in the face of the rockery.
- Base rocks are larger than upper rocks.

For roadside applications, it is also important to note any surcharges acting on the existing rockeries. Surcharges can include a traffic surcharge pressure, sloping backfill above the rockery, and embedded posts for guardrails or fences.

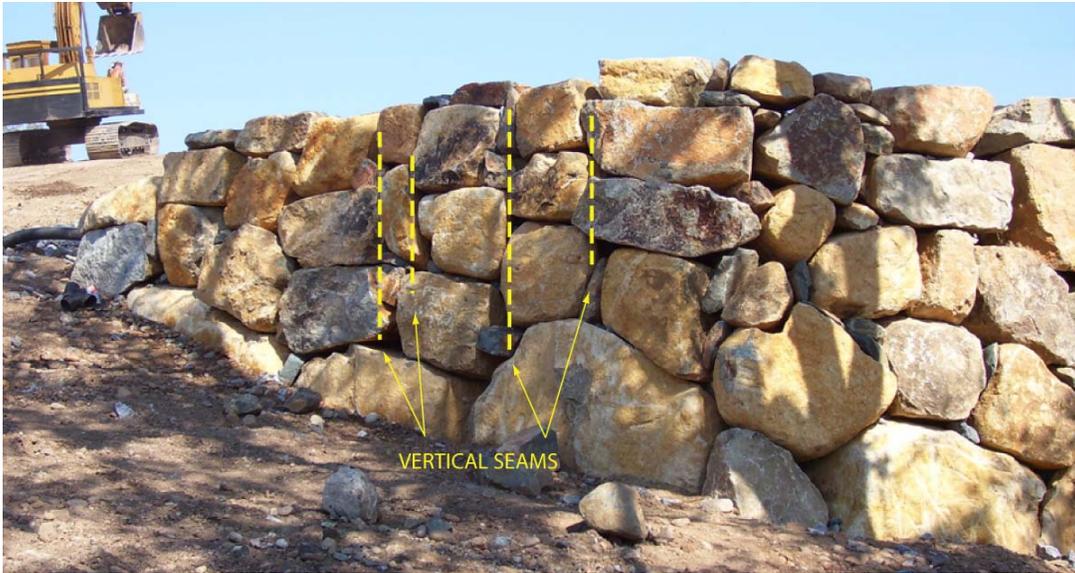
Figure 74 shows an example of a well-constructed rockery. The rocks all bear on at least two lower rocks, the rockery is properly chinked, there are no vertical seams, and the contact points are all at the front of the rockery. Figures 75 and 76 show examples of improperly constructed rockeries with either vertical stacks of rocks and/or rocks tilted out of the face of the rockery.



**Figure 74. Photograph. Example of proper layout, interlock, and chinking of variable shape and size rocks.**



**Figure 75. Photograph. Example of improper rockery layout evidences by vertical seams and rocks inclined out of face.**



**Figure 76. Photograph. Example of improper rockery layout evidenced by several vertical seams.**

### **Drainage**

Because of the presence of voids within the rockery face, it is unlikely that hydrostatic water pressure could build up behind an existing rockery and cause a failure. However, seepage from behind the rockery could wash soil through the face by piping or erosion. Erosion of soil through the rockery could cause settlement behind the rockery due to the resultant ground loss.

To prevent erosion through the rockery, the rockeries should have granular backdrains. The backdrains should discharge to a suitable outlet and be properly graded (or wrapped in a non-woven geotextile) to prevent soil migration from the retained soil into the drainage material. This granular backfill also helps to reduce the lateral earth pressures on the rockery, and, therefore, increases the overall stability. For that reason, the rockery should be inspected to as part of the evaluation to determine if granular material can be observed behind the facing rocks and the presence or absence should be noted in the observation notes. If drain pipes or outlets are observed, they should also be noted.

When performing the field evaluation, temporary stationing should be set along the top of the rockery and observations of the items discussed previously made at regular intervals. The values can be recorded in either tabular or graphical form relative to the temporary stationing. A copy

of the construction observation checklist presented in Chapter 6 can also be used at each cross-section location (station) to check the observed construction relative to current standards.

## **Evaluation of Base Width**

### ***Physical Evaluation***

The rockery base width,  $B$ , may be the most difficult parameter to measure in the field. Base rocks are typically embedded partially below grade, and the tops and backs of base rocks are obscured by overlying facing rocks. One method that can be used to directly measure the base rock width is to deconstruct a portion of the rockery and physically measure the base rocks. While this provides the most accurate measurement of base width, it only provides dimensions at a limited number of points along the rockery; that is, it doesn't necessarily capture variations along the structure. In addition, it is likely that partial deconstruction of an existing rockery is infeasible in most roadside applications without significant disruption to the existing roadway. It may also be unacceptable to deconstruct a historically significant rockery.

Another method that can be used consists of searching for sizable voids between rocks near the base of the rockery and estimating the base rock width with a tape measure. In general, a void at least 50 mm (2 in) in diameter at the face of the rockery is required for this method to be feasible. Furthermore, the rocks must interlock in such a way that the void used to estimate the width extends to the rear of the rockery. Because this process is highly dependent on the rock configuration, layout, and access for each rockery, the feasibility will vary on a case-by-case basis. Nevertheless, while this method will always result in some uncertainty regarding the actual base rock width, any inaccuracies will tend to be conservative as the estimated base rock widths will be equal to or less than the actual width.

### ***Geophysical Evaluation***

If physical measurements cannot be obtained, estimates of base rock width may need to be obtained using nondestructive geophysical methods. Geophysical methods generally consist of using vibrations, seismic waves, or radio waves (radar) to evaluate subsurface conditions. These methods generally rely on reflections caused by different material properties, especially density, to determine where transitions between materials occur. In the case of rockeries, they could be

used to evaluate the difference in material properties between the facing rocks and the retained soil. While geophysical measurement methods are feasible conceptually, the work performed for this study suggests application of these methods has been very limited with respect to rockery evaluation.

Based on review of the FHWA publication *Application of Geophysical Methods to Highway Related Problems*,<sup>(23)</sup> three types of geophysical methods appear feasible for estimation of rockery dimensions in the field—surface waves, ground penetrating radar, and cross-hole logging methods. The methods are discussed in the following sections.

**Surface Waves:**

Surface wave methods include methods such as Sonic Echo (SE), Impulse Response (IR), and the Ultraseismic Horizontal Profiling Method (HP). These methods generally involve generation of a surface wave by imparting a strong shock to the face of the structure being tested and measuring wave reflection using sensitive vibration-monitoring devices, such as geophones or accelerometers. The impact is usually generated using an instrumented sledgehammer or a mallet.

Surface wave techniques such as SE and IR have traditionally been used for vertical applications and may not be applicable to horizontal measurements. However, the HP method shows more promise in this regard as it has specifically been developed for horizontal applications such as bridge abutment evaluation. Unfortunately, a case history regarding the use of HP to estimate the thickness of a rockery could not be identified during this study. Surface wave methods are also limited by the sensitivity of the geophones and accelerometers to background vibrations, and, therefore, the methods would be difficult to implement where background vibrations are high, such as along high-traffic roads, especially highways.

The refractive seismic method, another surface wave method, was used for rockery evaluation with some success on the Apache Trail (SR 88) in Arizona in 2001. <sup>(Dennis Duffy, 2001)</sup> For this application, geophones were attached to the face of an existing rockery and rock accelerations induced by passing traffic were measured. While this data did not provide information relative

to base rock widths, it was used to evaluate overall rockery stability and sensitivity to traffic loads passing above the rockery.

**Ground Penetrating Radar:**

Ground Penetrating Radar (GPR) uses electromagnetic waves to evaluate subsurface conditions and is often used to determine the thickness of concrete, masonry, or timber structures. It can also be used to identify density contrasts beneath the soil, such as underground utilities or voids. The primary application for the evaluation of rockeries would be to estimate the thickness of the rockery by identifying the rear of the facing rocks. To accomplish this goal, it would be necessary to perform a GPR survey line immediately behind the top of the rockery and look for a density contrast between the facing rocks and retained soil, or from reflections off the back of the rocks. Horizontal GPR may also provide valuable information regarding rockery thickness.

**Cross-Hole Logging Methods:**

Cross-hole logging consists of drilling a vertical bore hole within 1.5 m (5 ft) of the back of the rockery and installing hydrophones or three-component geophones in the bore hole. In the case of the Parallel Seismic (PS) method, the waves are generated on the face of the structure and detected within the bore hole by the hydrophones. For the Borehole Radar method, the waves consist of electromagnetic (radar) waves. Information regarding the use of either of these methods for rockery evaluation could not be located as part of this study.

**Summary:**

This study found that geophysical methods have only been used sparingly for the evaluation of rockeries. Many of these methods were developed for, and work best on, continuous masses, such as concrete structures or concrete and steel foundations. Because rockery facing rocks have joints and seams and may only contact each other at discrete points, they are relatively discontinuous. Therefore, they may not transmit or reflect the seismic or electromagnetic waves in an effective manner and the results may be difficult to interpret.

Another potential drawback to geophysical methods is that they are most effective when the adjacent, dissimilar materials being studied have a relatively high density contrast, which is

usually represented by differences in shear wave velocity. Two example materials with much different shear wave velocities are soil and concrete. However, where rockeries are constructed with gravel or crushed rock backdrains, the shear wave velocity contrast between the facing rocks and crushed rock may not be sufficient to differentiate the two. In this case, it is possible the geophysical methods could yield a base rock width greater than what is actually present. Depending on the other assumptions made, this could result in an unconservative stability evaluation. The lack of contrast could also be a concern where rockeries retain very dense soil or bedrock.

Because of the uncertainties involved and the apparent scarcity of case histories, the application of geophysical methods to evaluate existing rockeries appears to be an area where future study should be focused.

### **GEOTECHNICAL/GEOLOGIC INVESTIGATION**

Just like new construction, the strength parameters for the soil or rock retained behind an existing rockery has a significant impact on the performance and acceptability of the rockery. Wherever existing rockeries are to be evaluated, a geotechnical and/or geologic investigation should be performed to evaluate the retained materials. The scope of the investigation should be appropriate for the height and length of the rockery being evaluated and the potential consequences should future instability occur. A typical investigation could involve excavation of test pits, drilling of soil borings, and/or geologic mapping. As a minimum, a properly performed investigation should identify:

- The expected soil or rock type to be retained by the rockery.
- The design parameters for the retained materials, including values for  $\phi$ ,  $c$ , and  $\gamma$ . They can either be determined through laboratory testing or by estimating values based on the soil/rock type and published correlations.
- Whether expansive or compressible soils are present at the site, and whether they can adversely impact rockery design or performance. If expansive soils are present, an addition horizontal surcharge (swelling) pressure may act on the back of the rockery.

- Areas where upslope drainage is directed towards the rockery.
- Groundwater or seepage conditions that could destabilize the rockery.
- Open tension cracks or scarps on the slope above the rockery, if present.
- Any other geotechnical or geologic hazards that could affect rockery design or construction.

## ROCKERY EVALUATION

Once the soil, surcharge, and rockery parameters have been collected, this data should then be used to evaluate the existing rockery. In the case of rockeries that are less than 1800 mm (6 ft) tall, the use of prescriptive standards, such as those published by the City of Seattle,<sup>(2)</sup> may be a sufficient screening tool. Caution is urged in using prescriptive methods for rockeries subject to surcharge loads, especially traffic, as most of the prescriptive methods specifically exclude the application of surcharge loads.

Another relatively simple screening tool is to compare the rockery geometry to the chart developed by Hendron,<sup>(5)</sup> as shown in Figure 9, for poorly constructed rockeries (PCRs). This chart provides a check that the actual height-to-base-width (H/B) ratio is below the critical ratio for the measured values of H, B, and  $\alpha$ . However, while this check provides a method to determine if the H/B ratio is above the critical value, it doesn't provide an actual factor of safety to aid in the evaluation. For instance, an H/B ratio that is twice the critical value may not translate into a  $FS_{OT}$  of 2. It should also be noted that the Hendron chart was developed assuming specific soil and rock properties, and, therefore, may not be applicable for all soil conditions encountered. As with the prescriptive design methods, this chart does not include an allowance for surcharge pressures.

As an alternative, equations developed by Gray & Sotir, as shown in Figures 16 and 17, can be used to generate site-specific charts utilizing the physical and geometric properties of the rockery and retained soil. The type of screening performed using these equations depends on the factor of safety (FS) selected. Using an overturning factor of safety ( $FS_{OT}$ ) of 1.0 would generate a chart that would allow for screening of rockeries that are inadequate and/or require “immediate”

replacement; that is, their H/B ratios are less than the critical value. The results from this analysis should be very similar to those discussed previously for the Hendron chart. Use of a higher FS, however, might allow for identification of rockery segments that are acceptable without further evaluation. Caution should still be used with this chart, as surcharge pressures are not included.

The screening methods described above are adequate to determine if existing rockeries are inherently stable or if further evaluation is required. Where additional evaluation is required, or if heavy surcharge pressures act on the rockeries, the analysis procedures discussed in Chapter 4 of this report should be used to check that the rockeries have adequate factor of safety.

### **Numerical Analysis**

As part of this study, a technical paper was reviewed that discussed the use of numerical modeling techniques to the evaluation of rockeries. The proposed method includes the use of Particle Flow Code (PFC) and the Discrete Element Method (DEM) to dynamically model the interactions between individual rocks.<sup>(Rock, 2006, unpublished)</sup> Rock interactions are modeled using a spring-dashpot constitutive model for normal contact and a Coulomb friction model for shear forces. The individual rocks are modeled as rigid bodies with discrete points of contact. If the factors of safety are low enough, the model uses the equations of motion to estimate movement/failure of the rockery and the final rockery configuration, if applicable.

Review of numerical modeling techniques was not a primary goal of this study, and therefore, the approach described above was not investigated in detail. In general, numerical analysis methods show promise for the evaluation of specific rockery conditions where rockery performance is critical and the soil strength parameters, rockery geometry (including individual rock sizes), and loading conditions are all clearly defined. However, the irregular nature of rockeries and heterogeneous nature of retained soil is such that determination of this level of detail is expected to be difficult under most circumstances. As a result, numerical evaluation of many cross sections for one wall is not likely to be justified, as the determination of the individual rock sizes is expected to be difficult and time consuming. The feasibility of numerical modeling techniques for rockery evaluation is recommended for future study.

CHAPTER 8 – GUIDE SPECIFICATIONS

The following is a guide specification to be used for rockery projects and should be modified, as necessary, to meet the specifics of each individual project. The Section and Subsection numbers shown below refer to the FLH’s *Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects, FP-03*. A standard rockery plan is also attached as Appendix C.

**Section 252. – ROCKERY**

**Description**

**252.01** This work consists of constructing rockery structures at the locations and to the dimensions shown on the plans. Rockeries are formed of interlocking, dry-stacked rocks without reinforcing steel, mortar, or concrete. Rockeries may be constructed as either single structures or in tiers.

**Material**

**252.02** Conform to the following Subsections:

Concrete v-ditch	601
Granular rock backdrain	703.03(c)
Foundation fill	704.01
Base, facing, and cap rocks	705.07
Plastic pipe	706.08(d) and (e)
Geotextile	714.01

**Construction Requirements**

**252.03 Qualifications.** Prior to the start of rockery construction, submit the following:

- (a) References citing satisfactory completion of at least three (3) rockeries of similar height and face area.
- (b) A summary of the experience of the primary equipment operator responsible for placement of base, facing, and cap rocks.

**252.04 General.** Survey according to Section 152.03(i), Construction Surveying and Staking, and verify the limits of the rockery installation.

The following definitions apply to rockery construction:

- (a) **Base Rock:** The base rock is the lowermost rock in the rockery, and bears directly on the soil/rock subgrade.
- (b) **Facing Rock:** The facing rocks comprise the bulk of the rockery and are stacked above the base rock.

- (c) **Cap Rock:** The cap rock is the uppermost rock in the rockery section and “caps” the rockery.

**252.05 Rockery Construction.**

- (a) **Rockery Foundation Excavation.** Perform the work under Section 209, Structure Excavation and Backfill. Excavate a foundation trench at least 300 millimeters (12 inches) below the bottom of the wall, running the full length of the proposed rockery. Deeper embedment may be required where a toe slope is present or where a leveling pad is specified. Excavate the foundation to a minimum width equal to the specified base rock width (‘B’) plus 300 millimeters (12 inches) to include the granular rock backdrain behind the rockery. Conform to the following:

- (1) Excavate the foundation in sections such that the rockery can be constructed in one shift or one day’s work, unless shoring is provided for the purpose to support the excavation.
- (2) If the CO determines the back cut is stable as excavated, the requirement of Subsection 252.05(a)(1) does not apply.
- (3) Exercise care during excavation of the back cut. Stability of temporary cut slopes is the responsibility of the Contractor.

- (b) **Rock Placement.** Place the first course of rock (base rock) on firm, unyielding soil or bedrock with full contact between the rock and the subgrade. Excavate any loose, soft or other wise unsuitable material present at foundation grade and replace with foundation fill as shown in the plans. Compact the foundation fill according to Subsection 204.11, Compaction.. As the rockery is constructed, place the rocks so that there are no continuous joints in either the vertical or lateral direction.

Stockpile a sufficient number of rocks to provide a good selection for placement. To obtain a better fit, place rocks which do not match the spaces offered by the previous course in a different location.

Avoid placing rocks which have shapes that create voids with a linear dimension greater than 300 millimeters (12 inches).

Except in isolated cases, place each rock so that it bears on at least two rocks below it. Locate at least one bearing point a distance no greater than 150 millimeters (6 inches) from the average face of the rockery.

The allowable tolerance for base rock widths is 150 millimeters (6 inches); however, do not place two or more consecutive base rocks with a width less than specified on the plans.

Slope the top surface of each rock towards the back of the rockery at an inclination of at least five (5) percent.

The minimum rockery thickness is based on minimum base rock width, as specified on the plans, and allowable face batter.

Securely place facing rocks so that the rocks are unable to be moved with a pry bar after the rockery is complete.

**(c) Voids.** Where voids with a minimum dimension of 150 millimeters (6 inches) or greater exist in the face of the rockery, chink the voids with smaller rock.

- (1) If there is no rock contact within the rockery thickness, chink the void with a smaller piece of rock.
- (2) Chinking rocks do not provide primary structural support for the overlying rock.
- (3) Chinking rocks can not be moved or removed by hand after rockery is complete. Reset loose chinking rocks until securely placed or grouted in place. Do not allow grout to be readily visible from the face of rockery.

**(d) Rockery Drainage.** Install the granular rock backdrain between the rockery and the back cut face being supported. The granular rock backdrain layer is at least 300 millimeters (12 inches) thick, measured horizontally from the back of the base rock to the face of the back cut. Place granular rock backdrain concurrent with rockery so that at no time is either more than 600 millimeters (24 inches) higher than the other.

- (1) Separate the crushed rock from the back cut by Type 1-B non-woven geotextile. Overlap the non-woven geotextile at least 300 millimeters (12 inches) at all seams.
- (2) Place a 100 millimeter (4 inch) diameter perforated drain pipe as shown in the plans. Surround the pipe on all sides by at least 100 millimeters (4 inches) of permeable backfill according to Subsection 703.04, Permeable Backfill.
- (3) Connect the perforated drain pipe to a non-perforated collector pipe at a spacing not to exceed 30 meters (100 feet) center-to-center. Connect the collector pipe to a controlled drain outlet, such as a storm drain, or outlet to a slope using a riprap apron according to Section 251, Riprap.
- (4) Do not connect collector pipes to systems designed for storm water retention in accordance with Best Management Practices design unless approved by the CO.

- (5) Cap the granular rock backdrain with at least 300 millimeters (12 inches) of native, relatively impermeable soil. Place non-woven geotextile between the soil cover and the granular rock backdrain.
- (6) Construct a concrete v-ditch along the top of the rockery as shown in the plans and according to Section 601, Minor Concrete Structures.

**252.06 Acceptance.** Rock for rockery will be evaluated under Subsection 106.02, Visual Inspection.

Survey work will be evaluated under Section 152, Construction Surveying and Staking.

Structure excavation will be evaluated under Section 209, Structure Excavation and Backfill.

Granular rock backdrain will be evaluated under Subsection 703.03(c), Granular Backfill. Foundation fill will be evaluated under Subsection 704.01, Foundation Fill.

Material for plastic pipe and geotextiles listed under Subsections 706.08, Plastic Pipe, and 714.01, Geotextile, will be evaluated under Subsections 106.02, Visual Inspection, and 106.03, Certification.

Construction of rockeries will be evaluated under Subsections 106.02, Visual Inspection, and 106.04, Measured or Tested Conformance.

### Measurement

**252.07** Measure rockeries by the square meter of rockery front face. Measure the rockery front face vertically from the bottom of the base rock elevation to the top of the cap rock. Compute the area using rockery heights measured at a maximum horizontal spacing of 10 meters (30 feet).

Granular rock backdrain, 100 millimeter (4 inch) drainage pipes (perforated and non-perforated), geotextile, and non-woven geotextile will not be measured for payment and are considered incidental to the rockeries.

Measure foundation fill under Section 208, Structure Excavation and Backfill for Selected Major Structures.

### Payment

**252.08** The accepted quantities, measured as provided above, will be paid at the contract price per unit of measurement for the pay items listed in the bid schedule. Payment will be full compensation for the work prescribed in this Section. See Subsection 109.05, Scope of Payment.

**Section 703. – AGGREGATE**

**703.03.Granular Backfill.** Furnish aggregate for the following installations.

- (a) **Granular Rock Backdrain.** Furnish granular rock backdrain conforming to Table 703-A.

**Table 703-A  
Granular Rock Backdrain Gradation**

Opening or Sieve Size	Percent by Mass Passing Designated Sieve (AASHTO T 27 & T 11)
150 mm (6 in.)	100
100 mm (4 in.)	0.0 – 25
19.0 mm (3/4 in.)	0.0 – 15
4.75 mm (No. 4)	0.0 – 5.0
75 µm (No. 200)	0.0 – 2.0

**Section 705. – ROCK**

**705.07 Rock for Rockeries.**

- (a) **General.** Furnish hard, angular, and durable rock that consists of intact blocks without open fractures, foliation, or other planes of weakness. Conform to the following:

- (1) Rock has sufficient hardness so that it cannot be scratched with a knife or scratched only with difficulty
- (2) Apparent specific gravity, AASHTO T 85 2.5 min.
- (3) Absorption, AASHTO T 85 4.2% max.
- (4) Los Angeles abrasion, AASHTO T 96 (500 rev) 40% max.
- (5) Coarse durability index, AASHTO T 210 50 min.
- (6) Sodium sulfate soundness (5 cycles), AASHTO T 104 10% max.
- (7) Freeze-thaw loss (12 cycles), AASHTO T 103 10% max.

- (b) **Sizes and shapes.** Furnish angular rocks that are generally cubical, tabular, or rectangular in shape. Conform to the following:

- (1) The minimum rock length is shown in the Rockery Design Schedule in the plans.
- (2) Rock width and height are greater than or equal to one-third of the rock length.

(3) The minimum rock dimension is 450 millimeters (18 inches).

(4) The minimum cap rock weight is 90 kilograms (200 pounds).

(c) **Color.** Furnish rocks with a color indigenous to the area. Submit at least three (3), 300 mm (12 in) samples of rock to be used for rockery facing that are representative of rock color for approval by the CO. Furnish rocks free of machine-made scratches, mars, or other damage to the visible face

**Section 714. – GEOTEXTILE AND GEOCOMPOSITE DRAIN MATERIAL**

**714.01 Geotextile.** For rockery construction with retained soil that has between 15 and 50% passing the 0.075  $\mu\text{m}$  (No. 200) sieve, furnish Type I-B geotextile. If the fines content is outside this range, type I-C geotextile may be required.

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APPENDIX A – ALTERNATIVE SEISMIC DESIGN PROCEDURE

**ALTERNATIVE DESIGN PROCEDURE**

Research by Richards and Elms<sup>(10)</sup> in the late 1970s suggests that application of traditional seismic design procedures, such as the Mononobe-Okabe procedure, does not adequately capture the performance of gravity walls during seismic loading. The method proposed by Richards and Elms presents an alternative and rational method for selection of the horizontal seismic coefficient,  $k_h$ . Using a Newmark-type displacement analysis, Richards and Elms developed equations for determination of  $k_h$  that assume some permanent displacement of a wall (in this case, the rockery) is acceptable. Richards and Elms came to this conclusion after realizing that most seismic evaluations of retaining structures neglected inertial forces for the structure itself. When the mass of the structure is large, such as the case with rockeries, the results can be unconservative. However, they concluded it is impractical to size a retaining structure such that it has sufficient mass to prevent all movement during a seismic event, and, therefore, it makes more sense to design to a tolerable level of displacement.

Richards and Elms defined a permanent displacement,  $\Delta$ , which the wall can tolerate without significant distress or failure. The value of  $\Delta$  is typically assumed to be about 5% of the base width of the rockery. For a 3-m-tall rockery, this is about 75 mm. In comparison, AASHTO suggests  $\Delta$  is equal to about  $250A$  (in mm) when using the AASHTO seismic design charts.<sup>(19)</sup> For  $A$  values of 0.3 and 0.4, this results values of  $\Delta$  of 75 and 100 mm, respectively. The actual tolerable displacement should be selected by the Geotechnical Engineer based on the sensitivity of any improvements retained behind the rockery.

Once the tolerable displacement has been selected, the seismic coefficient is determined using the equation presented in Figure 77. The value of  $\Delta$  must be input in inches. The values  $A_a$  and  $A_v$  represent acceleration coefficients developed by the Applied Technology Council (ATC).<sup>(24)</sup> The values are presented on regional maps, and, therefore, should not be considered to be site-specific.

$$k_h = A_s \left( \frac{0.2A_v^2}{A_s \cdot \Delta} \right)^{0.25}$$

**Figure 77. Equation. Determination of horizontal seismic coefficient,  $k_h$  ( $\Delta$  in inches).**

Richards and Elms concluded the wall should be designed with sufficient weight to resist the total thrust ( $F_{AE}$ ) on the wall (static plus seismic) multiplied by an inertial coefficient ( $C_{IE}$ ). A later study by Whitman<sup>(11)</sup> concluded that if the actual weight of the wall is at least 10% greater than the weight required (that is, a factor of safety of 1.1), then there is a greater than 90% probability that  $\Delta$  will not be exceeded. Higher factors of safety result in a greater confidence level. Figures 78, 79, and 80 present the equations required to calculate the factor of safety.

$$C_{IE} = \frac{\cos(\delta - \psi) + \tan \phi \cdot \sin(\delta - \psi)}{(1 - k_v)(\tan \phi - \tan \theta)}$$

**Figure 78. Equation. Computation of inertial thrust coefficient.**

$$W_{req} = F_{AE} \cdot C_{IE}$$

**Figure 79. Equation. Computation of weight required to resist seismic forces.**

$$FS_{seismic} = \frac{W}{W_{req}}$$

**Figure 80. Equation. Seismic factor of safety with regard to wall movement.**

If  $FS_{seismic}$  is greater than 1.1, there is a 90% probability that the wall displacement will not exceed  $\Delta$ , provided the level of ground shaking does not exceed the estimated values. The Geotechnical Engineer should also use the value of  $k_h$  obtained from Figure 77 to check the factors of safety with respect to seismic overturning, sliding, and bearing capacity described in Chapter 4.

## **COMMENTARY ON ATC MAPS**

The Richards and Elms procedure provides a rational method for determination of  $k_h$  and reasonable factors of safety for seismic conditions. However, the method relies on the values of  $A_a$  and  $A_v$  from the ATC maps, which were developed in the late 1970s. These maps are still referenced by many designers and regulatory agencies, including the State of California Office of Statewide Health Planning and Development (OSHPD),<sup>(25)</sup> and are likely acceptable in areas of relatively low seismicity (UBC Seismic Zones 1, 2, and portions of 3). However, recent building codes have introduced near-source amplification factors to increase the PGA in seismically active areas near fault lines, a concept that is not captured by the older ATC maps. Therefore, the authors believe they may be insufficient in areas of higher seismicity.

The authors have developed a method that may prove to be a rational, feasible way to scale up the values of  $A_a$  and  $A_v$  in areas of higher seismicity. The method consists of deriving new values of  $A_a$  and  $A_v$  using a site-specific elastic response spectrum. If a site-specific response spectrum is not available, a design spectrum can be constructed utilizing methods proposed by Newmark and Hall, as described in Chopra.<sup>(26)</sup>

To develop the design spectrum, the peak ground acceleration ( $a_{peak}$ , equivalent to PGA) should be obtained from the Geotechnical Engineer for a 10% probability of exceedance in 50 years. This value is equivalent to the value of  $A$  on the AASHTO maps. The peak velocity is determined from the standard ratio proposed by Newmark and Hall:

- Competent Soil:  $v_{peak}/a_{peak} = 122 \text{ (cm/sec)/g}$
- Rock:  $v_{peak}/a_{peak} = 91 \text{ (cm/sec)/g}$

Although not required, the peak displacement can also be determined utilizing the ratio  $(a_{peak})(d_{peak})/(v_{peak})^2 = 6$ . The values for peak acceleration, peak velocity, and peak displacement are then multiplied by amplification factors developed by Newmark and Hall for 5% critical damping.<sup>(26)</sup> Newmark and Hall developed amplification factors for both a median spectrum and a median plus one standard deviation. Based on the authors' review of the ATC report, we believe the median values were originally used by ATC, and, therefore, the median values should

be used for consistency. These amplification factors are  $\alpha_A = 2.12$ ,  $\alpha_V = 1.65$ , and  $\alpha_D = 1.39$ . Therefore:

- $A^* = \alpha_A * a_{\text{peak}}$
- $V^* = \alpha_V * v_{\text{peak}}$
- $D^* = \alpha_D * d_{\text{peak}}$

Once the values of  $A^*$  and  $V^*$  have been determined, they can be used to determine the effective peak acceleration (EPA) and effective peak velocity (EPV). The EPA is defined as the average spectral acceleration over the period range 0.1 to 0.5 seconds divided by 2.5. The average spectral acceleration can be taken as equal to the value of  $A^*$  computed above.

The EPV is defined as the average spectral velocity at a period of 1 second divided by 2.5. This value can be taken as equal to  $V^*$  from the Newmark and Hall analysis.

The effective peak acceleration coefficient,  $A_a$ , is numerically equal to the EPA when expressed as a decimal fraction of gravity. The effective peak velocity-related acceleration coefficient,  $A_v$ , is numerically equal to the EPV/76.2 when the EPV is expressed in cm/sec. For example,  $A_v=0.2$  when EPV = 15.24 cm/sec. In summary:

- $A_a = \text{EPA}/g$
- $A_v = \text{EPV}/(76.2 \text{ cm/sec})$

These values can then be used in the equation presented in Figure 77 for the determination of  $k_h$  and the seismic design performed in accordance with the Richards and Elms procedure. This value can also be used to determine the factors of safety with respect to seismic overturning, sliding, and bearing capacity described in Chapter 4.

At the time this report was prepared, the authors had successfully completed substantial design work using the Richards and Elms method. However, the method scaling of  $A_a$  and  $A_v$ , as described previously, has been used on very few designs, and has not been identified in the

literature. Therefore, we recommend this scaling method be viewed as a preliminary design concept requiring further study.



## APPENDIX B – SAMPLE DESIGN PROBLEM

The following is a sample design problem illustrating the design procedure described in Chapter 4. The hypothetical design case involves construction of a 2400-mm-tall (8-ft-tall) rockery with level back slope and toe slopes. A standard AASHTO vehicle surcharge is assumed to act behind the rockery. In order to simplify the analysis for the example problem, a temporary concrete traffic barrier (K-rail) was used instead of a guardrail. In addition, the following assumptions were made:

- The project geotechnical investigation has determined the rockery will retain medium dense clayey sand with a friction angle ( $\phi$ ) of  $33^\circ$  and no long-term cohesion.
- For short-term conditions, undrained cohesion will allow excavation of the rockery back cut at an inclination of 8V:1H.
- Passive pressure will be neglected at the toe of the rockery.
- Friction between the base rock and soil subgrade can be computed using the equation  $\mu = \tan\phi$ . Ultimate friction can be used because passive pressure is neglected.
- $FS_{OT} = 2.0$ ,  $FS_{SL} = 1.5$ , and  $FS_{BC} = 2.5$ .
- Inter-rock sliding will be satisfied through the plans and specifications, which will require the outermost bearing point to be within 150 mm (6 in) of the rockery face.
- The rockery face batter is 4V:1H.
- The site is located in Seismic Performance Category (SPC) C with an Acceleration Coefficient (A) of 0.25.

The design geometry and computation of the required base width and factors of safety follow:

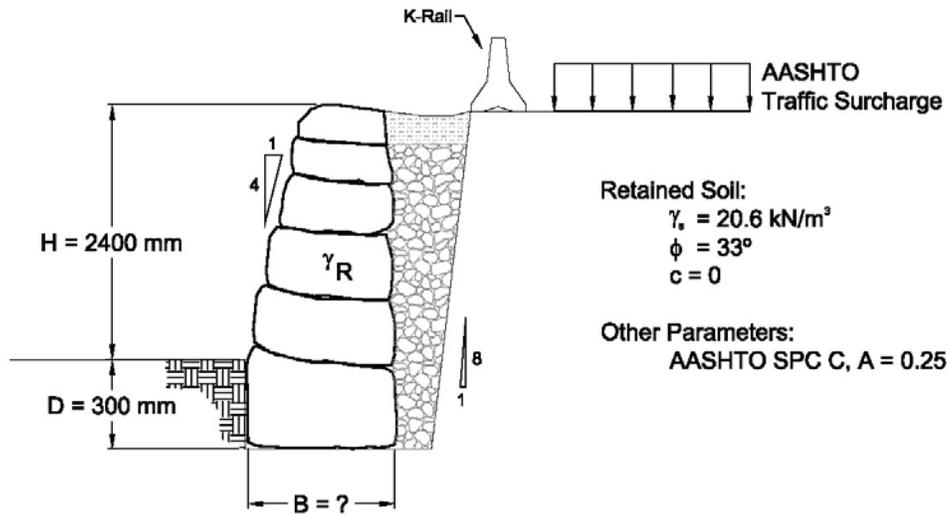


Figure 81. Graphic. Assumed Geometry for Example Problem—2400 mm (8 ft) rockery retaining medium dense clayey sand and subjected to a vehicle surcharge.

Parameters:

$H = 2.7\text{m}$ (total)	$\beta = 0$ (level backslope)
$H_R = 2.4\text{m}$ (exposed)	$\delta = \frac{2}{3}\phi = 22^\circ$
$D = 0.3\text{m}$	$\alpha = \tan^{-1}\left(\frac{8}{1}\right) = 82.9^\circ$
$\phi = 33^\circ$	$\psi = 90^\circ - \alpha = 7.13^\circ$
$\gamma_s = 20.6 \frac{\text{kN}}{\text{m}^3}$	$\mu = \tan \phi = 0.649$
$\gamma_R = 23.5 \frac{\text{kN}}{\text{m}^3}$	

Assume  $B = 1.2$  m for initial analyses.

Lateral Earth Pressure Coefficient:

$$K_A = \frac{\cos^2(\psi + \phi)}{\cos^2(\psi) \cdot \cos(\delta - \psi) \cdot \left[ 1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \beta)}{\cos(\delta - \psi) \cdot \cos(-\psi - \beta)}} \right]^2}$$

$$K_A = 0.217$$

**Surcharge:**

$$q_s = \gamma_s(0.6m) = 12.36 \frac{kN}{m^2}$$

**Total Horizontal Force:**

$$F_H = F_{A,H} + F_S = \frac{1}{2} \gamma_s K_A H^2 \cos(\delta - \psi) + q_s K_A H$$

$$F_H = 23.0 \frac{kN}{m}$$

**Wall Weight:**

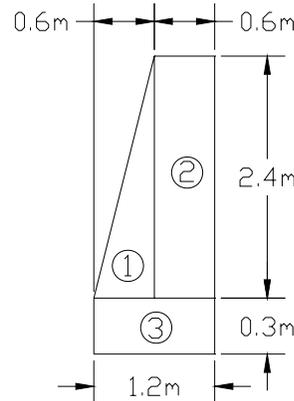
$$\gamma_R = 23.5 \frac{kN}{m^3}$$

$$W_1 = \frac{1}{2}(0.6m)(2.4m)(23.5 \frac{kN}{m^3}) = 16.9 \frac{kN}{m}$$

$$W_2 = (0.6m)(2.4m)(23.5 \frac{kN}{m^3}) = 33.8 \frac{kN}{m}$$

$$W_3 = (0.3m)(1.2m)(23.5 \frac{kN}{m^3}) = 8.46 \frac{kN}{m}$$

$$\sum_i W_i = 59.2 \frac{kN}{m}$$


**Frictional Resistance:**

$$F_\mu = \mu \cdot (W + F_{A,V}) = \mu \cdot \left( \sum_i W_i + \frac{1}{2} \gamma_s K_A H^2 \sin(\delta - \psi) \right)$$

$$F_\mu = 41.1 \frac{kN}{m}$$

**Factor of Safety against External Sliding:**

$$FS_{SL} = \frac{F_\mu}{F_H} = 1.8 \quad \text{OK}$$

**Factor of Safety against External Overturning:**

$$M_o = \frac{1}{2} \gamma_s K_A H^2 \cos(\delta - \psi) \left( \frac{H}{3} \right) + q_s K_A H \left( \frac{H}{2} \right)$$

$$M_o = 23.9 \frac{kN \cdot m}{m}$$

$$M_r = \sum_i W_i x_i + \frac{1}{2} \gamma_s K_A H^2 \sin(\delta - \psi) \left( \frac{H}{3} \cdot \tan(\psi) + B \right)$$

$$\sum_i W_i x_i = (16.9 \frac{kN}{m})(0.4m) + (33.8 \frac{kN}{m})(0.9m) + (8.46 \frac{kN}{m})(0.6m) = 42.3 \frac{kN \cdot m}{m}$$

$$M_r = 47.8 \frac{kN \cdot m}{m} \quad \therefore \quad FS_{OT} = \frac{M_r}{M_o} = 2.0 \quad \text{OK}$$

**Factor of Safety against Individual Rock Overturning:**

$$H - H' = 1.2\text{m}$$

$$x_1 = 0.2\text{m}$$

$$W_1 = \frac{1}{2}(0.3\text{m})(1.2\text{m})(23.5\frac{\text{kN}}{\text{m}^3}) = 4.23\frac{\text{kN}}{\text{m}}$$

$$x_2 = 0.6\text{m}$$

$$W_2 = (0.6\text{m})(1.2\text{m})(23.54\frac{\text{kN}}{\text{m}^3}) = 16.9\frac{\text{kN}}{\text{m}}$$

$$x' = 0.15\text{m}$$

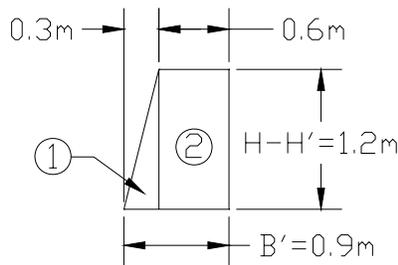
$$M_{o\_int} = \frac{1}{2}\gamma_s K_A (H - H')^2 \cos(\delta - \psi) \left( \frac{H - H'}{3} \right) + q_s K_A (H - H') \left( \frac{H - H'}{2} \right)$$

$$M_{o\_int} = 3.18\frac{\text{kN}\cdot\text{m}}{\text{m}}$$

$$M_{r\_int} = \sum_i W_{i\_top} (x_i - x') + \frac{1}{2}\gamma_s K_A (H - H')^2 \sin(\delta - \psi) \left( \frac{H - H'}{3} \cdot \tan(\psi) + B' \right)$$

$$M_{r\_int} = 8.60\frac{\text{kN}\cdot\text{m}}{\text{m}}$$

$$FS_{OT\_int} = \frac{M_{r\_int}}{M_{o\_int}} = 2.7 \quad \text{OK}$$



**Factor of Safety against Bearing Capacity:**

$$e = \frac{B}{2} - \frac{M_r - M_o}{W + \frac{1}{2}\gamma_s K_A H^2 \sin(\delta - \psi)}$$

$$e = 0.310$$

$$|e| \geq \frac{B}{6} = .200 \quad \text{NO GOOD; USE B=1.4m}$$

Wall Weight for B=1.4m:

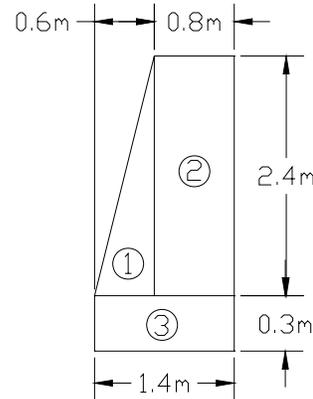
$$W_1 = 16.9 \frac{\text{kN}}{\text{m}}$$

$$W_2 = (0.8\text{m})(2.4\text{m})(23.5 \frac{\text{kN}}{\text{m}^3}) = 45.1 \frac{\text{kN}}{\text{m}}$$

$$W_3 = (0.3\text{m})(1.4\text{m})(23.5 \frac{\text{kN}}{\text{m}^3}) = 9.87 \frac{\text{kN}}{\text{m}}$$

$$\sum W_i = 71.9 \frac{\text{kN}}{\text{m}}$$

$$\bar{y} = \frac{16.9(0.3+0.8)+45.1(0.3+1.2)+9.87(0.15)}{71.9} = 1.22\text{m}$$



Factor of Safety against External Overturning:

$$M_o = \frac{1}{2} \gamma_s K_A H^2 \cos(\delta - \psi) \left( \frac{H}{3} \right) + q_s K_A H \left( \frac{H}{2} \right)$$

$$M_o = 23.9 \frac{\text{kN}\cdot\text{m}}{\text{m}}$$

$$M_r = \sum W_i x_i + \frac{1}{2} \gamma_s K_A H^2 \sin(\delta - \psi) \left( \frac{H}{3} \cdot \tan(\psi) + B \right)$$

$$\sum W_i x_i = (16.9 \frac{\text{kN}}{\text{m}})(0.4\text{m}) + (45.1 \frac{\text{kN}}{\text{m}})(1.0\text{m}) + (9.87 \frac{\text{kN}}{\text{m}})(0.7\text{m}) = 58.8 \frac{\text{kN}\cdot\text{m}}{\text{m}}$$

$$M_r = 65.1 \frac{\text{kN}\cdot\text{m}}{\text{m}} \quad \therefore \quad \text{FS}_{\text{OT}} = \frac{M_r}{M_o} = 2.7 \quad \text{OK}$$

Factor of Safety against Bearing Capacity for B=1.4m:

$$e = \frac{B}{2} - \frac{M_r - M_o}{W + \frac{1}{2} \gamma_s K_A H^2 \sin(\delta - \psi)}$$

$$e = 0.158$$

$$|e| \leq \frac{B}{6} = .233 \quad \text{OK}$$

$$q_{\text{max}} = \frac{W + \frac{1}{2} \gamma_s K_A H^2 \sin(\delta - \psi)}{B} \cdot \left( 1 + \frac{6e}{B} \right)$$

$$q_{\text{max}} = 91.1 \text{kPa}$$

$$q_{ult} = cN_c + 0.5\lambda_s B'' N_\lambda + qN_q \quad \text{From AASHTO Section 4.4.7.1}$$

$$\text{Assume, } c = 0 \quad B'' = B - 2e \quad \text{and} \quad N_{\lambda_{33}} = 35.19 \quad \text{From AASHTO Table 4.4.7.1A}$$

$$qN_q = 0 \quad (\text{neglected})$$

$$\text{therefore, } q_{ult} = 0.5(20.6)(1.4 - 2(0.158))(35.19)$$

$$q_{ult} = 393 \text{ kPa}$$

$$\text{therefore, } FS_{BC} = \frac{q_{ult}}{q_{max}} = 4.3 \quad \text{OK}$$

**Seismic Earth Pressure Coefficient:**

$$k_h = 0.5A = 0.125$$

$$k_v = 0$$

$$\theta = \tan^{-1} \left( \frac{k_h}{1 - k_v} \right) = 7.13$$

$$K_{AE} = \frac{\cos^2(\phi - \theta + \psi)}{\cos(\theta) \cdot \cos^2(-\psi) \cdot \cos(\delta - \psi + \theta) \cdot \left[ 1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \theta - \beta)}{\cos(\delta - \psi + \theta) \cdot \cos(\beta + \psi)}} \right]^2}$$

$$K_{AE} = 0.295$$

**Seismic Surcharge Pressure:**

$$F_A = \frac{1}{2} \gamma_S K_A H^2 = 16.29 \frac{\text{kN}}{\text{m}}$$

$$F_{AE} = \frac{1}{2} (1 - K_v) K_{AE} \gamma_S H^2 = 22.15 \frac{\text{kN}}{\text{m}}$$

$$\Delta F_{AE} = F_{AE} - F_A = 5.86 \frac{\text{kN}}{\text{m}}$$

**Factor of Safety against Seismic Overturning:**

$$M_{O,S} = M_O + \Delta F_{AE} \cos(\delta - \psi) \cdot 0.6H + k_h W \bar{y}$$

$$M_{O,S} = 44.0 \frac{kN \cdot m}{m}$$

$$M_{r,s} = M_r + \Delta F_{AE} \sin(\delta - \psi) [(0.6H) \tan \psi + B]$$

$$M_{r,s} = 67.5 \frac{kN \cdot m}{m}$$

$$FS_{OT,S} = \frac{M_{r,s}}{M_{O,S}} = 1.5 \quad \text{OK}$$

**Factor of Safety against Seismic Sliding:**

$$F_{H,S} = F_{A,H} + F_S + \Delta F_{AE} \cos(\delta - \psi) + k_h W$$

$$F_{H,S} = 37.7 \frac{kN}{m}$$

$$F_{R,S} = \mu (W + F_{A,V} + \Delta F_{AE} \sin(\delta - \psi))$$

$$F_{R,S} = 50.4 \frac{kN}{m}$$

$$FS_{SL,S} = \frac{F_{R,S}}{F_{H,S}} = 1.3 \quad \text{OK}$$

**Factor of Safety against Seismic Bearing Capacity:**

$$e_s = \frac{B}{2} - \frac{M_{r,s} - M_{O,S}}{W + F_{AE} \sin(\delta - \psi)} = 0.397$$

$$q_{\max,S} = \frac{W + F_{AE} \sin(\delta - \psi)}{B} \cdot \left( 1 + \frac{6e_s}{B} \right)$$

$$q_{\max,S} = 150 kPa$$

$$FS_{BC} = \frac{q_{ult}}{q_{\max}} = 2.9 \quad \text{OK}$$

Check seismic stability using the alternate approach by Richards and Elms:

Recompute  $k_h$  and  $K_{AE}$

$$k_h = A_a \left( \frac{0.2 A_v^2}{A_a \cdot \Delta} \right)^{0.25}$$

Assume  $\Delta = 0.05B = 0.07m = 2.75 \text{ in}$

Estimate  $A_a, A_v$  :

$$\alpha_{peak} = A = 0.25g$$

$$v_{peak} = 122 \frac{cm}{sec \cdot g} \cdot \alpha_{peak} = 30.5 \frac{cm}{s}$$

$$\alpha_A = 2.12$$

$$\alpha_V = 1.65$$

Calculate the Spectral Ordinates:

$$A^* = \alpha_A \cdot \alpha_{peak} = 0.53g$$

$$V^* = \alpha_V \cdot v_{peak} = 50.3 \frac{cm}{s}$$

Calculate EPA and EPV:

$$EPA = A^* / 2.5 = 0.212g$$

$$EPV = V^* / 2.5 \frac{cm}{s} = 20.1 \frac{cm}{s}$$

Calculate  $A_a$  and  $A_v$ :

$$A_a = EPA / g = 0.212$$

$$A_v = EPV / 76.2 \frac{cm}{s} = 0.264$$

$$\text{therefore, } k_h = 0.083$$

$$\theta = \tan^{-1} \left( \frac{k_h}{1 - k_v} \right) = 4.74$$

$$K_{AE} = \frac{\cos^2(\phi - \theta + \psi)}{\cos(\theta) \cdot \cos^2(-\psi) \cdot \cos(\delta - \psi + \theta) \cdot \left[ 1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \theta - \beta)}{\cos(\delta - \psi + \theta) \cdot \cos(\beta + \psi)}} \right]^2}$$

$$K_{AE} = 0.266$$

**Factor of Safety against Wall Movement:**

$$F_{AE} = \frac{1}{2} (1 - k_v) K_{AE} \gamma_s H^2 = 20.0 \frac{kN}{m}$$

$$C_{IE} = \frac{\cos(\delta - \psi) + \tan \phi \cdot \sin(\delta - \psi)}{(1 - k_v)(\tan \phi - \tan \theta)} = 2.00$$

$$W_{req} = F_{AE} \cdot C_{IE} = 40.0 \frac{kN}{m}$$

$$FS_{seismic} = \frac{W}{W_{req}} = 1.80 \quad \mathbf{OK}$$

Therefore, since  $FS_{seismic} > 1.1$ , there is at least a 90% probability that the observed movement following a design earthquake with the estimated level of shaking will be less than 70 mm

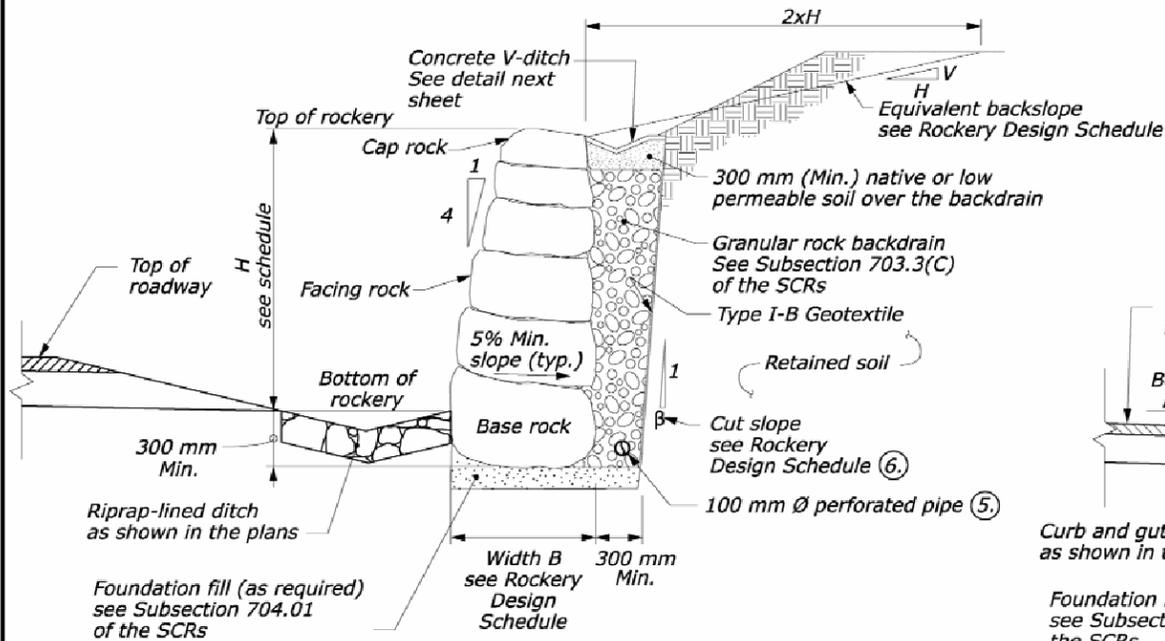
(2.75 in). Because the factor of safety is actually much greater than 1.1, the probability that the observed movement will not exceeded 70 mm (2.75 in) is likely greater than 95%.



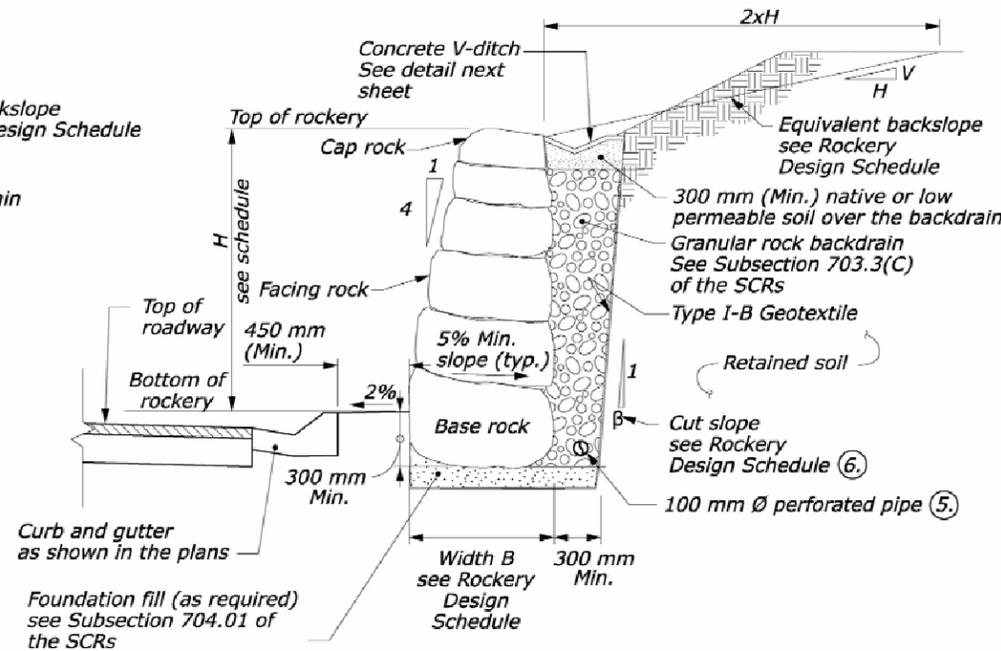
**APPENDIX C – SAMPLE SECTIONS AND DETAILS**

The following sample sections and details have been developed in conjunction with the FLH for use on FLH projects that will include rockery construction. The plans should be used in association with the design methods recommended in Chapter 4 and the guide specifications presented in Chapter 8.





**ROCKERY WITH RIPRAP-LINED DITCH  
TYPICAL SECTION**



**ROCKERY WITH CURB AND GUTTER  
TYPICAL SECTION**

**NOTE:**

1. Construct rockery and place base, facing, and cap rocks according to Section 252 of the SCRs. Place each rock individually by equipment suitable for lifting, manipulating, and placing rocks of the size and shape specified. Ensure that each rock is firmly set and supported by underlying materials and adjacent rocks. Reposition or replace loose rocks.
2. A maximum tolerance of 150 mm may be applied toward the total base rock width. Use rock with minimum L of 1700 mm. When L exceeds 1700 mm, two approximately equal size base rocks may be used, provided rocks are in contact at two points or more. Do not consecutively place base rocks with widths less than B.
3. Place base, facing, and cap rocks so that their height dimension is not greater than their width. The longest dimension of the base, facing, and cap rocks is perpendicular to face of rockery.
4. Where loose, soft, or otherwise unsuitable foundation soil conditions are encountered, contact the CO for supplemental recommendations.
5. Surround the perforated pipe on all sides by at least 100 mm of permeable backfill according to Subsection 703.04.  
  
Discharge outlet pipes to a protected outlet or other permanent drainage structure at low points in the rockery and at 30 m (max.) spacing. Drain outlets should not empty into storm drains that are designed to back-up during heavy flows.
6. Stability of temporary cut slopes is the responsibility of the Contractor.
7. Do not construct rockeries or slopes exceeding the heights shown on the Rockery Design Schedule without prior written approval by the CO.
8. Construct rockeries parallel to curb grade unless otherwise noted.

**ROCKERY DESIGN SCHEDULE**

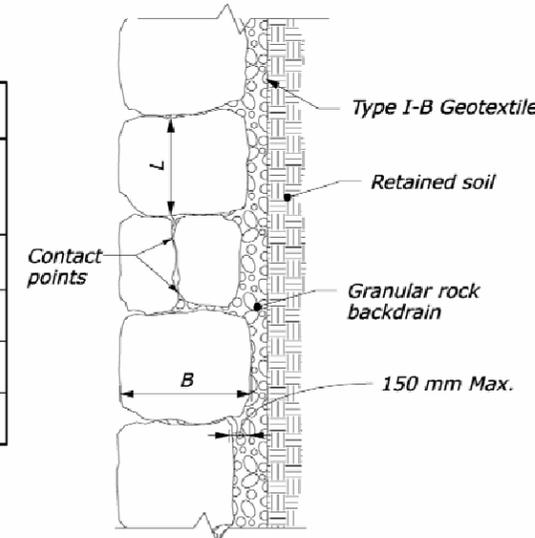
STATION	LT/RT TIER	MAX. HEIGHT H (m)	MIN. BASE ROCK WIDTH B (m)	MIN. CUT SLOPE BATTER β V:H	MAX. EQUIVALENT BACKSLOPE V:H	MIN. ROCK WEIGHT (kg)		DITCH TYPE	SURCHARGE TYPE (B)
						CAP ROCK	BASE ROCK		
BEGIN	END								

**Rockery Design Data:**

Friction angle,  $\phi = \text{---}^\circ$   
 Cohesion,  $c = 0$   
 Bulk unit weight,  $\gamma_R = 23.5 \text{ kN/m}^3$   
 Allowable bearing pressure =  $\text{---} \text{ KPa}$

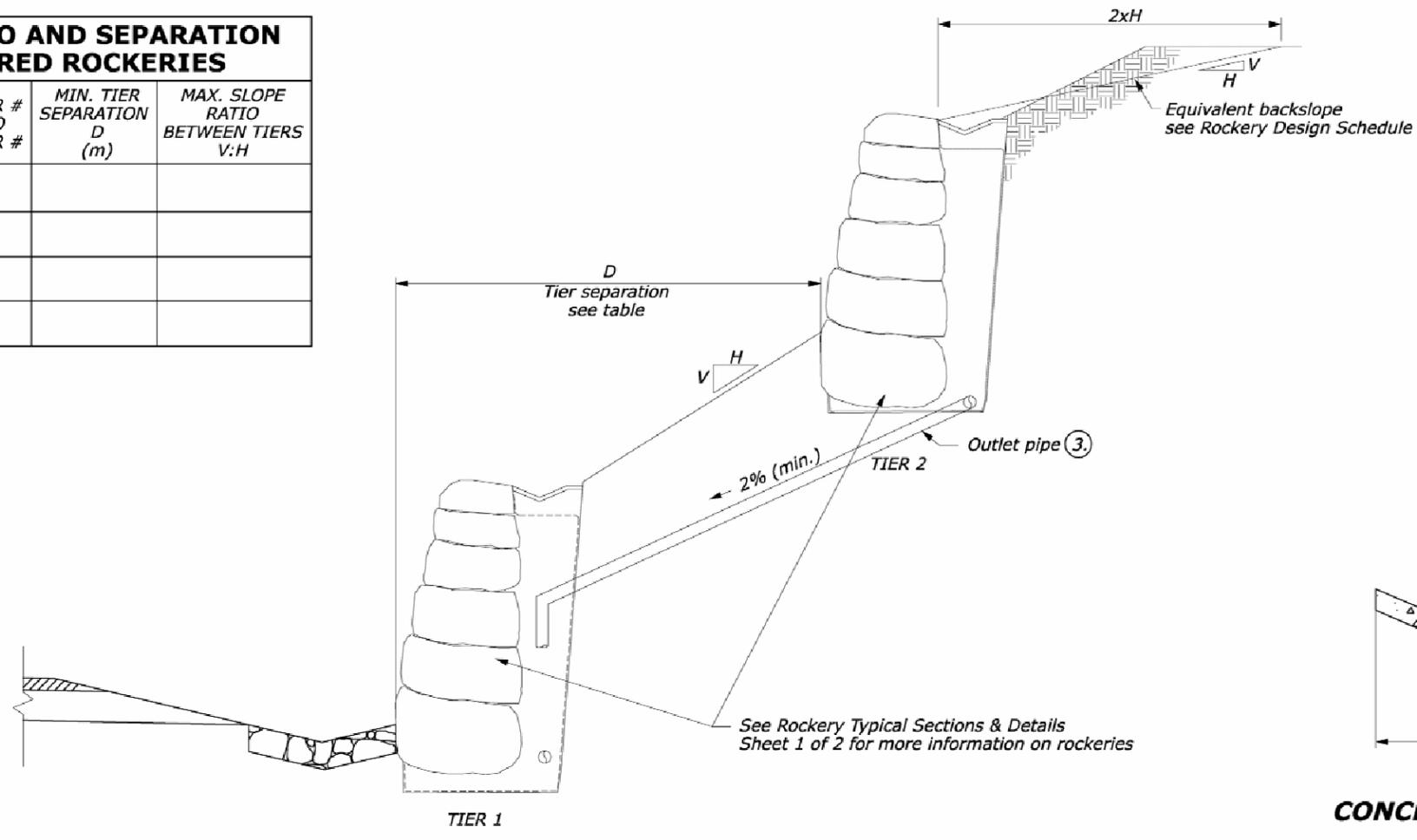
- (A) Minimum cut slope for design purposes only. Actual cut slope batter may be greater (B)  
 (B) Where "none" is indicated, no structures, vehicular traffic, or other surcharges can occur within a zone defined by an imaginary plane extending from the back of the base rock at an inclination of 1V:1.5H.

$\text{---}$  surcharge of  $\text{---} \text{ KPa}$  located  $\text{---}$  from back face of rockery.



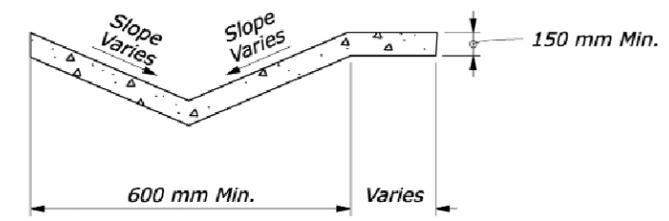
**BASE ROCK  
PLAN VIEW**  
See Note 2

SLOPE RATIO AND SEPARATION FOR TIERED ROCKERIES				
STATION		TIER # TO TIER #	MIN. TIER SEPARATION D (m)	MAX. SLOPE RATIO BETWEEN TIERS V:H
BEGIN	END			



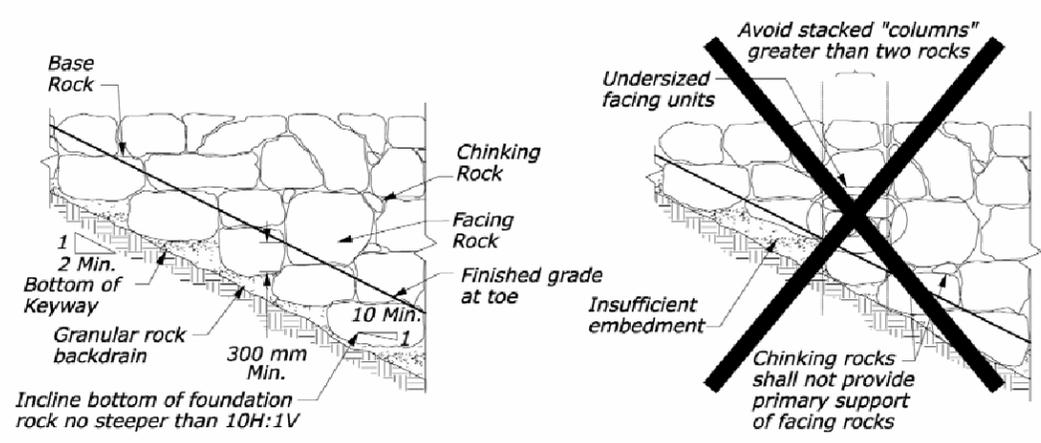
**NOTE:**

1. See Rockery Typical Sections & Details Sheet 1 of 2.
2. Construct riprap rundown at ends of rockery for catchment of V-ditch drainage.
3. Install 100 mm diameter solid outlet pipe at low points in the rockery and at 30 m (max.) spacing. Do not connect to drainage system for lower tier. Drainage systems for the upper and lower tiers outlet independently.



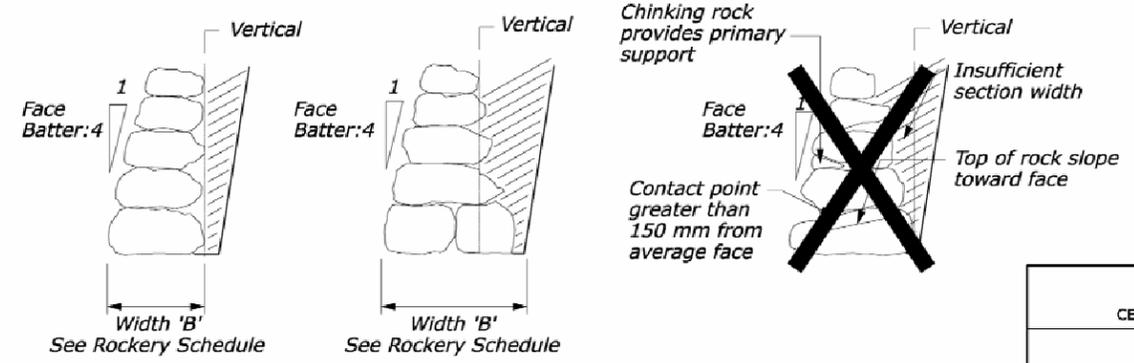
**CONCRETE V-DITCH DETAIL**

**ROCKERY WITH TIERS TYPICAL SECTION**



**CORRECT PARTIAL TYPICAL PROFILE**

**INCORRECT**



**CORRECT**

**CORRECT SECTION PROPERTIES**

**INCORRECT**

U.S. DEPARTMENT OF TRANSPORTATION  
 FEDERAL HIGHWAY ADMINISTRATION  
 CENTRAL FEDERAL LANDS HIGHWAY DIVISION

METRIC SPECIAL

**ROCKERY TYPICAL SECTIONS & DETAILS**

Sheet 2 of 2

SPECIAL M252-A

NOT TO SCALE