Design and Construction Guidelines for Geosynthetic Reinforced Soil Abutments and Integrated Bridge Systems

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Turner-Fairbank Highway Research Center
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FOREWORD

This manual provides information on the design and construction of Geosynthetic Reinforced Soil (GRS) abutments and the Geosynthetic Reinforced Soil-Integrated Bridge System (GRS-IBS). It serves as a follow-up to the *Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide*. As such, it includes several updates based on continued research conducted at the Federal Highway Administration's Turner-Fairbank Highway Research Center, as well as user feedback resulting from the use of GRS in the construction of more than 300 bridges.

Revisions include additional details on the design, construction, quality assurance, in-service inspection, maintenance, and repair of the GRS-IBS, along with revised material specifications, site selection criteria, and hydraulic guidance. The manual will benefit owners, designers, and construction personnel interested in implementing this technology.

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Director, Office of Infrastructure
Research and Development

Cover photo sources (clockwise from top left): FHWA; FHWA; Defiance County, OH.

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This manual outlines the state of the art and recommended practice for designing and constructing geosynthetic reinforced soil (GRS) technology for the application of abutments and the Integrated Bridge System (IBS). It was developed to provide engineers with the necessary background knowledge of GRS technology and its fundamental characteristics as an alternative to other construction methods. This manual presents step-by-step guidance on the design of GRS-IBS using the Load and Resistance Factor Design methodology. Material specifications for standard GRS-IBS are also provided. Detailed construction guidance is presented along with methods for the inspection, performance monitoring, maintenance, and repair of GRS-IBS. Quality assurance and quality control procedures are also covered. The procedures presented are based on 40 years of State and Federal research and deployment efforts focused on GRS technology for bridge support.</td></tr>
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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
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")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
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
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*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 AND SYMBOLS

Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
AS	abutment scour
ASD	Allowable Stress Design
B/H	base-to-height ratio
BR	vehicular braking force
CE	vehicular centrifugal force
CF	check flood
CIP	cast-in-place
CMU	concrete masonry unit
COB	center of bearing
CR	force effects due to creep
CS	contraction scour
CT	vehicular collision force
CV	vessel collision force
DC	dead load of structural components and nonstructural attachments
DD	downdrag force
DF	design flood
DL	dead load
DW	dead load of wearing surfaces and utilities
EDM	electronic distance measurement
EH	horizontal earth loads
EL	miscellaneous lock-in force effects
EQ	earthquake load
ES	earth surcharge load
EV	vertical pressure from dead load of earth fill
FHWA	Federal Highway Administration
FR	friction load
GMSE	geosynthetic mechanically stabilized earth
GRS	geosynthetic reinforced soil
HDPE	high-density polyethylene

HEC	Hydraulic Engineering Circular
IBS	Integrated Bridge System
IC	ice load
IM	vehicular dynamic load allowance
IPI	in-place inclinometer
LL	live load
LRFD	Load and Resistance Factor Design
LS	live load surcharge
LTD	long-term degradation
MARV	minimum average roll value
MSE	mechanically stabilized earth
NHS	National Highway System
PET	polyethylene terephthalate
PL	pedestrian live load
PP	polypropylene
PS	secondary forces from post-tensioning for strength limit states; or total prestress forces for service limit states
QA	quality assurance
QC	quality control
RSF	reinforced soil foundation
SE	force effect due to settlement
SH	force effects due to shrinkage
SPT	standard penetration test
SRW	segmental retaining wall
TG	force effect due to temperature gradient
TU	force effect due to uniform temperature
WA	water load and stream pressure
WL	wind on live load
WS	wind load on structure

Symbols

α	angle between the projections of the inner and outer edge lines of the surcharge to the wall face (radians)
α_b	angle between the wall face and projection of the midline of the bridge surcharge to the wall face (radians)
β	angle between the wall face and projection of the midline of the surcharge to the wall face (radians)
β_b	angle between the projections of the inner and outer edge lines of the bridge surcharge to the wall face (radians)
γ_b	unit weight of retained backfill (F/L^3)
$\gamma_{DC\ MAX}$	maximum dead load load factor
$\gamma_{DC\ MIN}$	minimum dead load load factor
$\gamma_{EH\ MAX}$	maximum horizontal earth pressure load factor
$\gamma_{EH\ MIN}$	minimum horizontal earth pressure load factor
$\gamma_{EV\ MAX}$	maximum vertical earth pressure load factor
$\gamma_{EV\ MIN}$	minimum vertical earth pressure load factor
γ_f	unit weight of foundation soil (F/L^3)
γ'_f	effective unit weight of the foundation soil (F/L^3)
γ_{LL}	bridge live load surcharge load factor
γ_{LS}	traffic live load surcharge load factor
γ_p	load factor for permanent loading
γ_r	unit weight of reinforced backfill (F/L^3)
γ_{rb}	unit weight of road base material (F/L^3)
γ'_{RSF}	effective unit weight of the reinforced soil foundation backfill
γ_{SE}	load factor for settlement
γ_{TG}	load factor for temperature gradient
ΔP_{AE}	dynamic (pseudostatic) force
ε_L	lateral strain
ε_V	vertical strain
μ	friction factor between the abutment wall base and the reinforced soil foundation
μ_{RSF}	friction factor between the base of the reinforced soil foundation and the foundation soils
σ_c	confining stress due to the facing. (F/L^2)

σ_h	lateral pressure within the geosynthetic reinforced soil abutment at a given depth and location (F/L^2)
$\sigma_{h,b}$	equivalent lateral stress distribution due to the retained soil behind the geosynthetic reinforced soil abutment (F/L^2)
$\sigma_{h,bridge}$	lateral pressure due to the bridge dead load surcharge within geosynthetic reinforced soil
$\sigma_{h,bridge,eq}$	lateral pressure due to the equivalent bridge load (F/L^2)
$\sigma_{h,bridge,f}$	factored lateral pressure due to the equivalent bridge load (F/L^2)
$\sigma_{h,f}$	factored the factored total lateral pressure within the GRS abutment at a given depth and location (F/L^2)
$\sigma_{h,LL}$	lateral stress distribution due to the equivalent superstructure live load pressure (F/L^2)
$\sigma_{h,q}$	lateral pressure due to surcharge loading (F/L^2)
$\sigma_{h,rb}$	lateral pressure due to road base surcharge within geosynthetic reinforced soil (F/L^2)
$\sigma_{h,rb,f}$	factored lateral pressure due to road base surcharge within geosynthetic reinforced soil (F/L^2)
$\sigma_{h,t}$	lateral pressure due to traffic live load surcharge (F/L^2)
$\sigma_{h,t,f}$	factored lateral pressure due to traffic surcharge within geosynthetic reinforced soil (F/L^2)
$\sigma_{h,total}$	total lateral pressure due to loads on geosynthetic reinforced soil mass (F/L^2)
$\sigma_{h,W}$	lateral pressure due to the weight of the reinforced backfill in the GRS abutment (F/L^2)
$\sigma_{h,W,f}$	factored lateral stress due to the weight of geosynthetic reinforced soil (F/L^2)
$\sigma_{v,base,n}$	nominal vertical pressure at the base of the geosynthetic reinforced soil mass (F/L^2)
$\sigma_{v,base,R}$	factored vertical pressure at the base of the geosynthetic reinforced soil mass (F/L^2)
ΣM_D	total driving moment (L-F/L)
$\Sigma M_{D,R}$	total factored driving moment (L-F/L)
ΣM_R	total resisting moment (L-F/L)
$\Sigma M_{R,R}$	total factored resisting moment (L-F/L)
ΣV	total vertical load (F/L)
ΣV_R	total factored vertical load (F/L)

ϕ	friction angle of soil (degrees)
ϕ_b	friction angle of retained backfill (degrees)
ϕ_{crit}	critical friction angle (degrees)
ϕ_f	friction angle of foundation soil (degrees)
ϕ'_f	effective friction angle (degrees)
ϕ_r	friction angle of reinforced backfill (degrees)
ϕ_{rb}	friction angle of road base material (degrees)
$\Phi\tau$	sliding resistance factor
Φ_{bc}	bearing resistance factor
Φ_{cap}	nominal resistance factor
Φ_{reinf}	reinforcement strength resistance factor
a_b	setback distance between the back of the face and the beam seat (L)
b	bearing width of the bridge/beam seat
b_{block}	width of the facing element (L)
b_q	width of surcharge loading (L)
$b_{q,vol}$	width of the load along the top of the wall including the setback (L)
$b_{rb,t}$	width of the traffic and road base surcharges over the GRS abutment (L)
B	base length of the reinforcement not including the wall face (L)
B'	effective foundation width (L)
B_{min}	minimum length of the reinforcement lengths at the base of the abutment
B_{RSF}	base width of the reinforced soil foundation (L)
B_{total}	total base width of the geosynthetic reinforced soil abutment including the width of the facing (L)
c_b	cohesion of the retained backfill (F/L^2)
c'_f	effective cohesion of the foundation soil (F/L^2)
c_r	cohesion of the reinforced backfill (F/L^2)
c_{rb}	cohesion of the road base (F/L^2)
c_u	undrained shear strength of the foundation soil (F/L^2)
C_C	compression index
C_R	recompression index
d_e	minimum clear space (L)
d_{max}	maximum grain size (L)
D_{50}	median material size (L)

D_f	depth of embedment (L)
D_L	maximum lateral displacement (L)
D_{RSF}	depth of the reinforced soil foundation (L)
D_v	vertical settlement in the geosynthetic reinforced soil mass (L)
$e_{B,n}$	nominal eccentricity for bearing resistance
$e_{B,R}$	factored eccentricity for bearing resistance (L)
$f_{b,rb}$	inertia force of the retained backfill and road base (F/L)
F_b	nominal lateral force behind the geosynthetic reinforced soil abutment due to the retained backfill (F/L)
$F_{b,RSF}$	nominal lateral force behind the geosynthetic reinforced soil abutment and reinforced soil foundation due to the retained backfill (F/L)
F_{DL}	horizontal inertia of the dead load (F/L)
F_{LL}	horizontal inertia of the live load (F/L)
F_n	nominal driving force behind the geosynthetic reinforced soil abutment for direct sliding (F/L)
$F_{n,RSF}$	total nominal driving force on the RSF (F/L)
F_{rb}	nominal lateral force behind the geosynthetic reinforced soil abutment due to the road base surcharge (F/L)
$F_{rb,RSF}$	nominal lateral force behind the geosynthetic reinforced soil abutment and reinforced soil foundation due to the road base surcharge (F/L)
F_R	factored driving force for direct sliding calculations at the base of the geosynthetic reinforced soil abutment (F/L)
$F_{R,RSF}$	factored driving force for direct sliding calculations at the base of the reinforced soil foundation (F/L)
F_t	nominal lateral force behind the geosynthetic reinforced soil abutment due to live load on the roadway (F/L)
$F_{t,RSF}$	nominal lateral force behind the geosynthetic reinforced soil abutment and reinforced soil foundation due to live load on the roadway (F/L)
$FS_{bearing}$	factor of safety against bearing failure
$FS_{capacity,emp}$	factor of safety for vertical capacity using the empirical method
$FS_{capacity,an}$	factor of safety for vertical capacity using the analytical method
FS_{reinf}	factor of safety for required reinforcement strength
$FS_{slide,GRS}$	factor of safety against direct sliding at the base of the geosynthetic reinforced soil
$FS_{slide,RSF}$	factor of safety against direct sliding at the base of the reinforced soil foundation

g	acceleration due to gravity [F/T^2]
h_{eq}	equivalent height of overburden for traffic surcharge (L)
H	height of the geosynthetic reinforced soil abutment including the clear space distance (L)
H_{abut}	height of the geosynthetic reinforced soil abutment (L)
H_{rb}	height of the road base (or bridge beam) (L)
k	coefficient of permeability (L/T)
K_a	coefficient of active earth pressure
K_{ab}	coefficient of active earth pressure for the retained backfill
K_{ar}	coefficient of active earth pressure for the reinforced backfill
K_{pr}	coefficient of passive earth pressure for the reinforced backfill
L_{abut}	abutment length (L)
L_{block}	length of a facing block (L)
L_{span}	span length of the bridge (L)
N_γ, N_c, N_q	bearing capacity coefficients (dimensionless)
N_{block}	number of facing blocks in a column along the cross sectional height of the abutment
P_{ir}	inertia force of the reinforced fill beneath the abutment (F/L)
$P_{ir,b}$	inertia force of the integrated approach road base behind the superstructure (F/L)
q	surcharge load (F/L^2)
q_{DL}	superstructure dead load pressure (F/L^2)
$q_{DL,allow@\varepsilon=1\%}$	prescribed service bearing pressure for dead load pressure (F/L^2)
q_{LL}	superstructure live load pressure (F/L^2)
q_n	in LRFD, the nominal bearing resistance of the foundation soil; in ASD, the bearing bearing capacity of the foundation (F/L^2)
$q_{n,an}$	nominal bearing resistance of the geosynthetic reinforced soil abutment using the analytical method (F/L^2)
$q_{n,emp}$	nominal bearing resistance of the geosynthetic reinforced soil abutment using the empirical method (F/L^2)
q_R	factored bearing resistance (F/L^2)
q_{rb}	surcharge due to the structural backfill of the integrated approach (also referred to as "road base") (F/L^2)
q_t	roadway live load surcharge due to traffic (F/L^2)

Q_{100}	storm-event having a probability of occurrence of one every 100 years (F^3/T)
Q_{500}	storm-event having a probability of occurrence of one every 500 years (F^3/T)
Q_{DL}	dead load due to the bridge superstructure on each abutment (F)
Q_{LL}	live load due to the bridge superstructure on each abutment (F)
Q_{rb}	dead load due to the road base on each abutment (F)
RF_{global}	global reduction factor
R_n	nominal resisting force for direct sliding at the base of the geosynthetic reinforced soil abutment (F/L)
$R_{n,RSF}$	nominal resisting force for direct sliding at the base of the reinforced soil foundation (F/L)
R_R	factored resisting force for direct sliding at the base of the geosynthetic reinforced soil abutment (F/L)
$R_{R,RSF}$	factored resisting force for direct sliding at the base of the reinforced soil foundation (F/L)
S_v	reinforcement spacing (L)
$T_{@\varepsilon = 2\%}$	reinforcement strength at 2 percent reinforcement strain (F/L)
T_{allow}	allowable reinforcement strength (F/L)
T_f	ultimate reinforcement strength (F/L)
$T_{f,f}$	factored reinforcement strength (F/L)
T_{req}	required reinforcement strength (F/L)
$T_{req,f}$	factored required reinforcement strength (F/L)
$V_{allow,emp}$	factored applied stress on top of the geosynthetic reinforced soil mass using the empirical method (F/L^2)
$V_{applied}$	applied stress on top of the geosynthetic reinforced soil mass (F/L^2)
$V_{applied,f}$	factored applied stress on top of the geosynthetic reinforced soil mass (F/L^2)
W	weight of the geosynthetic reinforced soil abutment backfill (F/L)
W_2	bottom width of the contracted section (L)
W_{block}	weight of an individual facing block (F)
W_{face}	weight of the facing elements (F/L)
W_{Roll}	width of the reinforcement roll (L)
W_{RSF}	weight of the reinforced soil foundation (F/L)

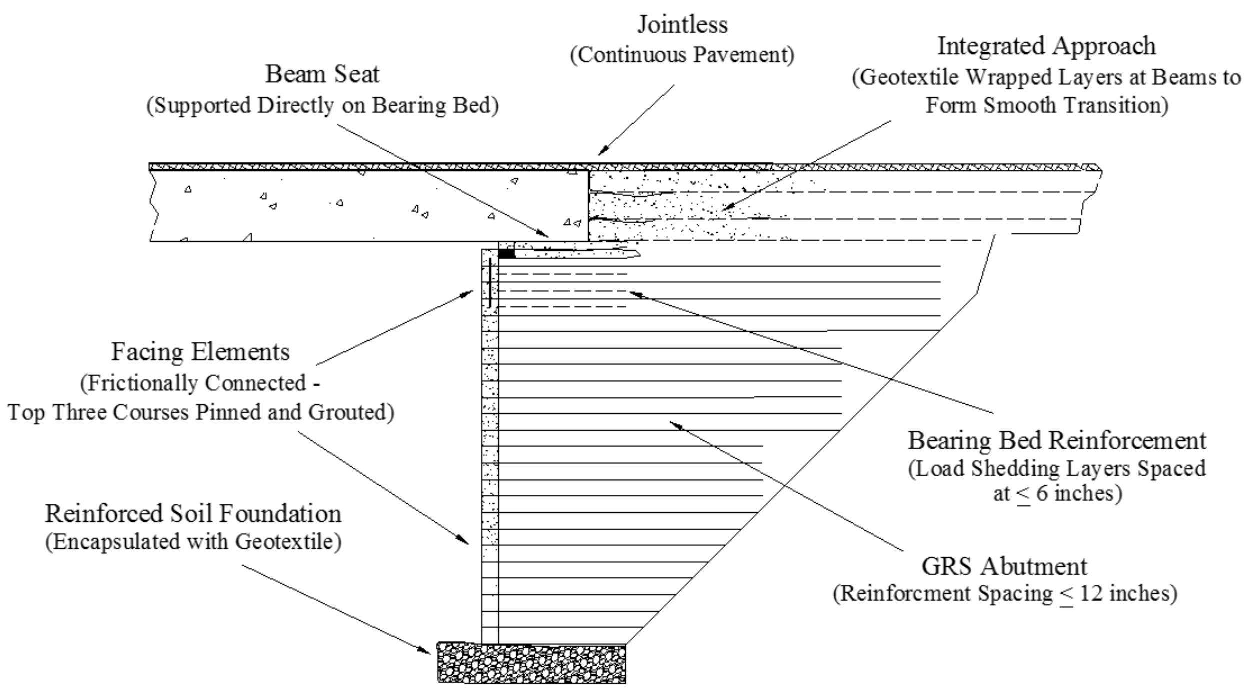
W_t	total weight (weight of the geosynthetic reinforced soil plus the weight of the bridge beam plus the weight of the road base over the geosynthetic reinforced soil mass only) (F/L)
$W_{T,R}$	factored total resisting weight (weight of geosynthetic reinforced soil plus weight of bridge beam plus weight of the road base over the geosynthetic reinforced soil mass only) (F/L)
$W_{T,R,RSF}$	total factored resisting weight including the reinforced soil foundation (F/L)
x	distance from the edge of the load to the point of interest for lateral pressure (L)
y_o	main channel flow depth through the bridge (L)
$y_{o(CF)}$	flow depth in the bridge opening during the check flood (L)
$y_{o(CF/DF)}$	flow depth in the bridge opening during either the check flood or design flood, depending on the requirements of the scour analysis (L)
$y_{o(DF)}$	flow depth in the bridge opening during the design flood (L)
y_1	average flow depth in the upstream section (L)
y_{s+LTD}	depth from the original river bed to the abutment scour elevation for the check flood and long term degradation (L)
$y_{U(CF)}$	flow depth on the upstream side of the bridge opening
x_{RSF}	length of the reinforced soil foundation in front of the abutment wall face (L)
z	depth from the top of the wall (L)

CHAPTER 1. TECHNOLOGY OVERVIEW

1.1 INTRODUCTION

Geosynthetic reinforced soil (GRS) consists of alternating layers of compacted fill and closely spaced (≤ 12 inches) geosynthetic reinforcement. The technology has many applications, with bridge support being the focus of this design and construction manual, which serves as a complete guide that supersedes the 2011 *Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide*.(1) Updates are largely based on transitioning to Load and Resistance Factor Design (LRFD), recent research results, performance data, and feedback resulting from implementation of the GRS Integrated Bridge System (IBS) through the Federal Highway Administration's (FHWA) Every Day Counts initiative.

GRS can be used for a standalone abutment or within an IBS. The GRS-IBS is a fast, cost-effective method of bridge support that blends the roadway into the superstructure to create a jointless interface between the bridge and the approach (figure 1). The same principles of design and construction discussed throughout this manual can be applied for standalone GRS abutments, but for simplicity, the guidance herein is focused on the IBS.



Source: FHWA.

Figure 1. Illustration. Typical GRS-IBS cross section.

The IBS consists of three main components, all utilizing GRS technology: (1) the reinforced soil foundation (RSF), (2) the GRS abutment, and (3) the integrated approach. The RSF is composed of granular fill material that is compacted and encapsulated with a geotextile. It provides embedment and increases the bearing width and capacity of the GRS abutment. The GRS abutment provides direct support for the bridge without the need for deep foundation elements. The integrated approach also uses GRS to blend the roadway to the superstructure, creating a jointless transition. The IBS alleviates the "bump at the bridge" problem often encountered with conventional bridge systems.

GRS-IBS was initially developed by FHWA almost 20 years ago to help meet the demand for the next generation of single-span bridges in the United States. They can be built quicker and at a lower cost and can provide improved performance.[2] Along with realizing time and cost savings, GRS-IBS also has many other distinct and innovative qualities. For example, GRS technology is extremely durable and performs well in earthquakes when designed and constructed, as outlined in this manual. GRS abutments can be built with readily available material using common construction equipment without the need for specialized labor. Construction of the abutment is contained within its footprint for a reduction of environmental impact as well as a reduced work zone. Additional benefits include convenience and design flexibility, as GRS-IBS can be built in variable weather conditions and adapted easily in the case of unforeseen site conditions.

This guidance manual addresses the design and construction of GRS-IBS. In-service performance, inspection, maintenance, and repair are also described, along with design requirements and considerations for hydraulic and seismic conditions. Finally, procedures for quality assurance (QA) and quality control (QC), including necessary construction documents, are provided. The purpose of this manual is to allow designers and contractors to effectively design and construct a GRS-IBS that will have an estimated service life of at least 100 years.

1.2. GRS COMPOSITE BEHAVIOR

GRS is often misconstrued as the same technology as geosynthetic mechanically stabilized earth (GMSE). While both GRS and GMSE utilize the same basic ingredients (e.g., structural backfill, geosynthetic reinforcement, and a facing element), the close reinforcement spacing (≤ 12 inches) of GRS results in more effective composite behavior as compared to larger-spaced (typically 18–24 inches) GMSE systems. GRS is analogous to concrete, whereby the combination of materials produces a new material with its own properties and characteristics. To take advantage of this composite behavior, a different design procedure from GMSE is required for GRS, as outlined in this manual. The differences largely lie in the internal stability design.[3]

The internal stability design method recommended herein accounts for this composite behavior and takes into consideration the backfill and reinforcement spacing comprising the GRS composite material. In this method, connection, reinforcement pullout, and other mechanisms associated with the traditional design model used for GMSE are not modes of failure for closely spaced GRS.[3] This is because closer reinforcement spacing creates more soil–geosynthetic interaction. In GRS, the reinforcement not only serves to resist tensile forces but also functions to restrain lateral deformation of the soil, increase lateral confinement, simulate the effect of cohesion in a granular fill (while maintaining all desirable characteristics of granular soil), suppress dilation of the soil, enhance compaction-induced stresses, increase ductility, and reduce migration of fines. These added benefits develop because of the close reinforcement spacing, and they all contribute to improved overall performance.

Adding to the understanding of the composite behavior, a series of large-scale laboratory load tests have been conducted on GRS, allowing for the failure surface of GRS composites to be visually observed.[4] For tests with reinforcement spacing less than 12 inches, a shear surface formed through the composite, leading to rupture of the reinforcement (figure 2). The same mode of failure was not seen for the larger-spaced systems where failure was determined by the soil between the sheets of geosynthetic, and the full strength of the reinforcement was never developed.

A. $S_V \approx 16$ inches. B. $S_V \approx 8$ inches. C. $S_V \approx 4$ inches.

Source of subfigure images: FHWA.
S_v = Reinforcement spacing.

Figure 2. Photos. Composite behavior of GRS versus GMSE as shown in tests conducted at reinforcement spacings of 16, 8, and 4 inches.

It is important to note that the transition into GRS behavior is not dependent solely on reinforcement spacing; the aggregate size, friction angle, facing element, and other elements are also likely contributing factors. Research is ongoing to investigate the relationship between these factors; however, in the meantime, limiting spacing to 12 inches and using the materials specified in this manual will result in reliable composite behavior.

1.3 SITE SELECTION

GRS technology can be used for many bridges on all types of roads, both on and off the National Highway System (NHS). GRS-IBS was developed to meet the needs for the majority of common, bread-and-butter–type bridges, but it is not suitable for every location. Because it is a shallow foundation system, the technology is ideally suited for grade separations; however, it can also be applied over water crossings. The feasibility of GRS-IBS or any other foundation type should be evaluated and based on economy, time, safety, and other factors important to the transportation agency. Previous recommendations on GRS-IBS in the initial interim guidance included abutment heights less than 30 ft, span lengths less than 140 ft, and design service limit state pressures up to 4,000 lb/ft^2.[1] These criteria were established based on experience to date, not limitations of the technology itself. Performance data (based on the authors' experience) suggest that these boundaries can be expanded if designed appropriately following this guide.

For water crossings, careful attention must be placed on the hydraulic analysis to ensure long-term stability under potential scour conditions. With scour being the leading cause of bridge failures, it is important to assess the site and ensure stability in a storm event. The optimum stream-crossing site is a fully stable channel, which is characterized by banks and a bed that are not prone to change.(5) If excessive scour depths are estimated, then the use of GRS-IBS will likely be too expensive or difficult to construct, and an alternative foundation system should be considered. The presence of water, however, does not preclude the use of GRS-IBS. More information specific to water crossings is provided in appendix D, including the use of designed scour countermeasures to protect the GRS abutment. In areas of highly compressible foundation soils, ground improvement techniques can be evaluated with GRS-IBS in comparison to deep foundations.

A summary of key site selection considerations within this manual includes the following:

- Single-span bridges.
- Grade separations or feasible water crossings.
- Competent or improved foundation soils.
- No limitations on average daily traffic, span length, or abutment height.

CHAPTER 2. TERMINOLOGY

2.1 TERMINOLOGY

Common terminology used throughout this manual and their definitions are as follows:

- **Biaxial**: Reinforcement strength is approximately equal in both the machine and the cross-machine directions.

- **Clear space**: Clear space is the vertical distance between the top of the wall face (block) and base superstructure. Typically, this distance is 3 inches or at least 2 percent of the wall height.

- **Facing**: The outer wall facing of a GRS wall or abutment can be built with any material, including, but not limited to, natural rock, concrete modular blocks, gabions, timber, or geosynthetic wrapped face. GRS is generic and can be built with any combination of geosynthetic reinforcement, compacted granular fill, and facing, although some combinations of the three components are more compatible than others.

- **GRS**: GRS includes alternating layers of compacted granular fill reinforced with geosynthetic reinforcement (e.g., geotextiles and geogrids). The primary reinforcement spacing in GRS is less than or equal to 12 inches. Facing elements can be frictionally connected to the reinforcement layers to form the outer wall. Depending on the facing, mechanical connections to other facing elements or the layers of reinforcement are not needed.

- **GRS abutment**: This is a GRS system designed and built to support a bridge. Usually, GRS abutments have three sides: the abutment face wall and two wing walls. All GRS abutments must have the abutment face wall. In some circumstances, depending on the layout, a GRS abutment can be built with one or none of the wing walls.

- **GRS abutment face wall**: A GRS abutment face wall is the vertical or near-vertical wall parallel to the center of bearing and designed to support the bridge. The length of a GRS abutment face wall is typically the total width of the bridge structure plus any additional width necessary to accommodate the structure (e.g., guardrail deflection distance).

- **GRS-IBS**: A unique application of GRS technology in the specific context of bridge abutments, GRS-IBS is different from other, more general GRS abutments that use many common elements associated with traditional bridge abutments. GRS-IBS bridge abutments are built to economically support a bridge on the granular fill directly behind the block face. GRS-IBS can be used to integrate the bridge structure with the bridge approach to create a jointless bridge system. One version of GRS-IBS uses adjacent concrete box beams or voided slabs supported directly on the GRS abutments without a concrete footing or elastomeric pads. The bridge has no cast-in-place (CIP) concrete or approach slab. A typical IBS cross section shows a GRS mass compacted directly behind the bridge beams to form the approach way and to create a smooth transition from the roadway to the bridge. Another version of GRS-IBS uses steel girders with either a CIP footing or a precast sill. The footing or sill is placed directly on the GRS abutment. The reinforcement layers behind the beam ends are wrapped to confine the compacted approach fill against the beam ends and the adjacent side slopes to prevent lateral spreading. Since the wrapped-face GRS mass behind the beam ends is freestanding, the active lateral pressure against the beam ends is considered negligible. The wrapped-face fill also prevents migration of fill during thermal bridge cycles and vehicle live load (LL).

- **GRS composite**: A composite mass built with GRS creates a freestanding internally supported structure with reduced lateral earth pressures with considerable strength. A GRS composite is not rigid and is therefore tolerant to differential foundation settlement.

- **GRS wall**: This includes any wall built with GRS.

- **GRS wing wall**: A GRS wing wall is a wall attached and adjacent to the abutment face wall. The wing walls are built at the same time as the abutment face wall and at a right or other angle to the abutment face wall. The wing walls are built to support the roadway and the approach embankment. They must be designed to retain the soil fill in the core of the approach embankment and to protect the abutment from erosion.

- **Minimum average roll value (MARV)**: MARV is the average geosynthetic tensile strength less two standard deviations. MARV values are used to determine geosynthetic conformance with specifications.

- **Performance test**: A performance test is a large-element GRS load test to provide the designer with material strength properties of a particular GRS composite mass built with a unique combination of reinforcement, compacted fill, and facing elements. The procedure involves axially loading the GRS mass while measuring vertical settlement and lateral deformation to monitor performance. This information can then be used to aid in the design process to predict performance of a full-scale GRS abutment. This is also referred to as a “mini-pier experiment.”

- **Setback**: A setback is the lateral distance from the back of the wall face to the front of the bearing area. This distance must be a minimum of 8 inches.

- **Uniaxial**: Reinforcement strength is larger in one direction than the other.

CHAPTER 3. MATERIALS

3.1 INTRODUCTION

The materials used in GRS, primarily structural backfill, geosynthetic reinforcement, and a facing, are generic and should be selected to meet the required material specifications presented herein. There are also several miscellaneous materials needed for GRS-IBS. Consideration should be made for locally or readily available materials that meet the specifications to reduce project cost. Aesthetics and other project-specific requirements may also play a role in material selection.

3.2 BACKFILL MATERIAL

Backfill selection is important because it is a major structural component for GRS, comprising the majority of the volume within the IBS. The backfill must be properly compacted (as specified in chapter 7) or according to minimum agency requirements. Locally sourced aggregates are the most economical choice for GRS construction as long as they meet the material specifications. Most State specifications for aggregate, which are usually met by local quarries and aggregate suppliers, will satisfy the material requirements. It should be noted that some backfill materials are easier to work with than others, and certain backfills are more suitable for compacting behind a given facing element than others. Backfill selection will therefore have an impact on construction efforts. In GRS-IBS construction, areas to consider for backfill selection are the abutment, the RSF, and the integrated approach.

3.2.1 GRS Abutment Backfill

Abutment backfill should consist of crushed, hard, durable particles or fragments of stone or gravel. These materials should be free from organic matter or deleterious material such as shale or other soft particles that have poor durability. The backfill should follow the size and quality requirements for crushed aggregate normally used locally in the construction and maintenance of highways by Federal or State agencies.

Abutment backfill typically consists of either well- or open-graded aggregates (discussed in section 3.2.1.1 and section 3.2.1.2, respectively). At the time that this manual was written, open-graded aggregates had been selected on most GRS-IBS projects, as observed by FHWA, due to the ease of construction, lower weight, and favorable drainage characteristics. If the abutment will be submerged at any point in time, open-graded aggregates are recommended because they are free-draining and will not build up hydrostatic pressure. Well-graded backfills have different advantages, including their stiffness characteristics, availability, and familiar compaction control techniques. Regardless of the selected gradation, the measured friction angle of the backfill should be greater than or equal to 38 degrees.

Natural fill materials can be used if the amount of fines is limited to 12 percent to minimize the potential for frost susceptibility and if other material requirements are met (i.e., primarily maximum size and friction angle). Engineers should specify structural backfill to ensure sufficient strength, serviceability, and constructability of the abutment. Additionally, site and environmental conditions should be considered in the selection of the backfill.

Backfill selection is dependent on the following factors:

- The backfill should meet the minimum recommended requirements described in section 3.2.1.1 or section 3.2.1.2 of this manual for well- or open-graded materials, respectively.

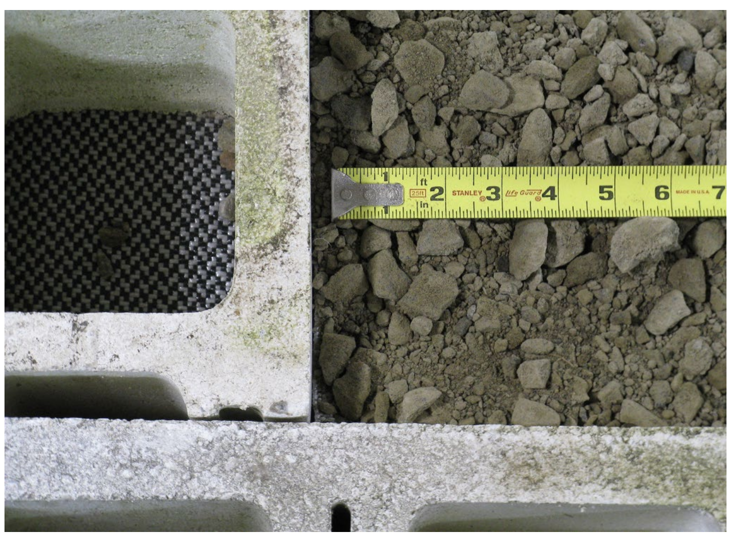
- Open-graded backfill is recommended for an abutment located in a flood zone to facilitate the flow of water out of the abutment without loss of material.

- For open-graded fine aggregates, consider specifying a maximum diameter of 0.5 to 0.75 inch to make it easier to spread, level, and compact than well-graded fill.

- Angular particles are recommended to maximize the shear strength of the GRS composite.

3.2.1.1 Well-Graded Backfill

Well-graded backfill has a mixture of various particle sizes. Most transportation departments have a specification for well-graded backfill. An example of a typical well-graded abutment backfill is shown in figure 3, while table 1 provides recommended criteria for a suitable well-graded material that would be appropriate for a GRS abutment or IBS.



Source: FHWA.

Figure 3. Photo. Sample well-graded structural backfill.

Table 1. GRS well-graded backfill specifications.

Parameter	Test Method	Criteria
Maximum aggregate size	AASHTO T 27[6]	Between 0.5 and 2 inches
Percent passing No. 200 sieve	AASHTO T 11[7]	≤ 12 percent
Coefficient of uniformity	ASTM D6913[8]	≥ 4
Coefficient of curvature	ASTM D6913[8]	Between 1 and 3
Plasticity index	AASHTO T 90[9]	≤ 6
Friction angle	AASHTO T 236[10]	≥ 38 degrees
Soundness	AASHTO T 104[11]	The backfill shall be substantially free of shale or other poor durability particles. The material shall have a sodium sulfate soundless loss of less than 15 percent after five cycles.

A maximum grain size limitation of 2 inches is recommended by FHWA for efficient compaction behind the abutment wall facing. The amount of fines passing the No. 200 sieve shall be less than or equal to 12 percent based on a washed sieve per AASHTO T 11.[7] A dry sieve analysis may not produce a representative value of fines content, which could lead to frost susceptibly and should be avoided, especially in areas with seasonally cold climates. The friction angle is one of the most important parameters, defining the strength of the backfill. Most structural backfills will have high measured friction angles, but verifying that it is 38 degrees or more will ensure adequate strength and serviceability for the application intended.

3.2.1.2 Open-Graded Backfill

Open-graded backfill comprises aggregates of primarily one size. Recommended open-graded backfill material consists of clean, crushed, angular stone. An example of a typical open-graded abutment backfill is shown in figure 4, and table 2 provides recommended criteria for a suitable open-graded material that should be used in the IBS. Typically, this represents a range of AASHTO M 43 aggregates from No. 5 to No. 89. which have friction angles greater than 40 degrees.[12,13]

Source: FHWA.

Figure 4. Photo. Sample open-graded aggregate.

Table 2. GRS open-graded backfill specifications.

Parameter	Test Method	Criteria
Minimum maximum aggregate size	AASHTO T 27(6)	≥ 0.5 inch
Maximum aggregate size	AASHTO T 27(6)	≤ 2 inches
Percent passing No. 50 sieve	AASHTO T 11(7)	≤ 5 percent
Friction angle	AASHTO T 236(10)	≥ 38 degrees
Soundness	AASHTO T 104(11)	The backfill shall be substantially free of shale or other poor durability particles. The material shall have a sodium sulfate soundless loss of < 15 percent after five cycles.

The maximum grain size to efficiently achieve compaction behind the abutment wall face is 0.5–1.5 inches. While larger diameter aggregates theoretically create a stronger GRS composite, there is a tradeoff in their use in terms of constructability. The larger the aggregate size, the more difficult it is to place and compact the material in the confines of an abutment. The amount of fines passing the No. 50 sieve should be less than or equal to 5 percent based on FHWA recommendations. Typical aggregates meeting these requirements from AASHTO M 43 are a No. 89 to No. 5 aggregate.(12) Information about the strength properties of open-graded aggregates is available in *Strength Characterization of Open-Graded Aggregates for Structural Backfills*.(13)

3.2.2 RSF Backfill

The backfill for the RSF can be either well-graded (see section 3.2.1.1) or open-graded aggregate (see section 3.2.1.2) and can be densely compacted to form a level working platform to place the first course of facing block. It is desirable from a constructability perspective that the RSF match the abutment backfill material, but it is not required.

3.2.3 Integrated Approach Backfill

The wrapped GRS located directly behind the beam end is important to provide a smooth, integrated transition from the approach to the bridge. This component of GRS-IBS is called the "integrated approach." The fill material used for this transition should be a well-graded structural backfill (see section 3.2.1.1). Material that is similar to an aggregate blend specified by State transportation departments for an unbound base course in pavement construction will typically meet the specifications. Note that attention should be made with respect to the frost susceptibility of the soil. Depending on the region, backfills that tend to heave may lead to cracks at the bridge approach interface during freeze-thaw cycles.

3.2.4 Riprap

Because of its flexibility, availability, and relative cost, riprap is often selected as the scour countermeasure to protect against erosion and ensure foundation and embankment stability. For a water crossing, riprap protection should be sized for the specific site conditions. The stone used should be hard, durable, angular, free of organic and spoil material, and resistant to weathering and water action. It should be free of clay or soft shale seams that can slake when exposed to water. Hydraulic Engineering Circular (HEC) No. 23 should be used to adequately size riprap or other scour countermeasures.[14]

3.3 GEOSYNTHETIC

Since GRS is generic, there are many types of geosynthetic materials of various strengths available for abutment construction. While any geosynthetic reinforcement meeting the design requirements for a GRS abutment can be selected (see chapter 4), woven polypropylene (PP) geotextiles with a MARV of 4,800 lb/ft ultimate reinforcement strength (T_f) have been traditionally used to build in-service GRS abutments. Chapter 4 provides design guidance on the required reinforcement strength for the strength and service limits, which is a function of the lateral stress, reinforcement spacing, and backfill properties.

A geotextile is often selected for several reasons, including cost, ease of placement, and compatibility with the frictional connection that is used between a block facing and the GRS composite. Geogrids have also been successfully used for GRS abutments. It is important to note that a geotextile must be used for the RSF and the integrated approach to fully encapsulate the material.

Geosynthetics can be either uniaxial or biaxial, meaning the reinforcement either has more strength in one direction or it has equal strength in both directions along its length, respectively. The term "machine direction" (or "warp direction") refers to the strength along the length of the roll, and the term "cross-machine direction" (or "fill/weft direction") refers to the strength along the width of the roll (figure 5). The strong direction is typically in the cross-machine direction for biaxial geosynthetics and the machine direction for uniaxial products. If a biaxial product is used for the abutment, having greater strength in the cross-machine direction allows for easy placement, as the geosynthetic can be rolled out parallel to the wall. When using a uniaxial product in the machine direction, the placement must be perpendicular to the wall. It is recommended, however, that biaxial reinforcement be specified to avoid construction placement errors and to simplify corner details by eliminating conflicts of multiple overlapping layers.

Source: FHWA.

Figure 5. Illustration. Geosynthetic roll direction.

The following should be specified for geosynthetic reinforcement:

- Laboratory test results documenting the MARV ultimate tensile strength along with the measured MARV strength at 2 percent strain in accordance with ASTM D4595 for geotextiles or ASTM D6637 for geogrids.[15,16] Tests should be conducted at a strain rate of 10 percent/min.
- Manufacturing QC program and data indicating minimum test requirements, test methods, test frequency, and lot size for each product.[17] Further minimum conformance requirements as prescribed by the manufacturer should be indicated (see table 3).

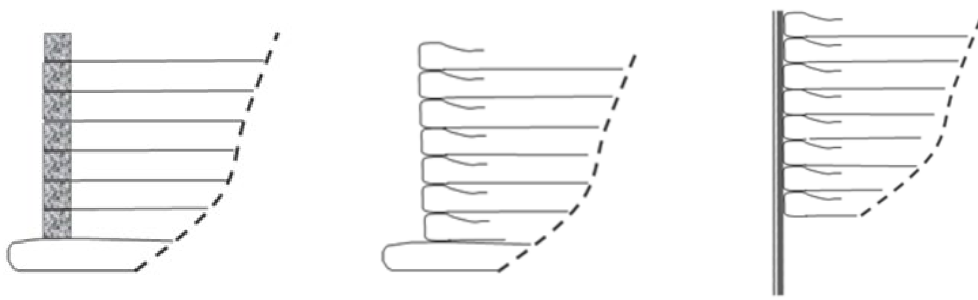
Table 3. Geosynthetic conformance criteria.

Test	Test Procedure
Wide-width tensile (geotextiles)	ASTM D4595[15]
Wide-width tensile (geogrids)	ASTM D6637[16]
Single-rib tensile (geogrids)	ASTM D6637[16]
Specific gravity (high-density polyethylene (HDPE) only)	ASTM D1505[18]
Melt flow index (PP and HDPE)	ASTM D1238[19]
Inherent viscosity (polyethylene terephtalate (PET) only)	ASTM D4603[20]
Carboxyl end group (PET only)	ASTM D7409[21]

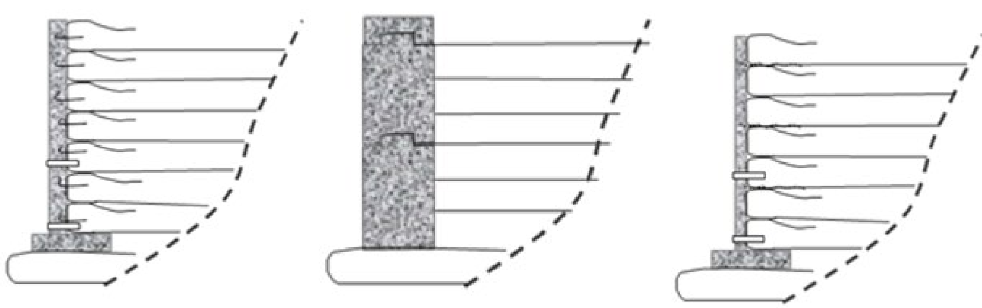
3.4 FACING ELEMENTS

The most common facing element used to build generic GRS abutments is a concrete modular block, either a normal weight concrete masonry unit (CMU) or a segmental retaining wall (SRW) unit. Based on internal data collected from FHWA, many projects have utilized a CMU that is frictionally connected to the GRS composite; SRW units have also been used and serve the same purpose. Other facing elements are possible because the facing is not considered a structural component of GRS, and its contribution to strength is not accounted for in the internal stability design (see chapter 4). While the facing does contribute to the stiffness and strength of the composite, its primary purpose for GRS abutments is to provide a form for compaction, serve as a façade, and protect against loss of the granular fill from outside weathering. (See references 4 and 22–24.)

Since the facing is not considered a structural element of a GRS abutment, it is up to the user to select a facing that meets project requirements for durability, cost, availability, aesthetics, etc. The facing can be composed of various materials such as concrete, timber, natural rock, metal, automobile tires, shotcrete, gabion baskets, or the existing abutment. Figure 6 illustrates the cross sections of various facing elements that have been used in the construction of GRS walls; other options not shown are also available. For simplicity, the focus throughout this manual is on the use of concrete modular blocks as facing elements. Construction and connection details will change if other facing elements are selected; however, the general design procedure as presented in chapter 4 will remain unchanged.



A. Modular block. B. Wrapped face. C. Sheet pile.



D. Cast-in-place. E. Wet cast concrete block. F. Full-height panel.

Source of subfigure images: FHWA.

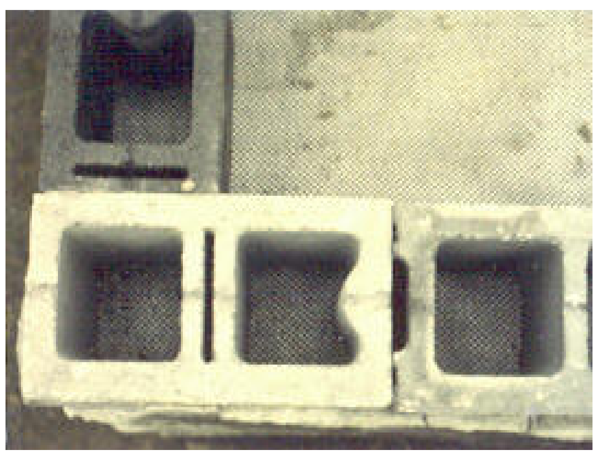
Figure 6. Illustrations. GRS walls with different facing elements.

As previously stated, the most common facing element for vertical GRS walls and abutments is the split-faced CMU with actual dimensions of $7^{5}/_{8}$ by $7^{5}/_{8}$ by $15^{5}/_{8}$ inches. In addition to the height of $7^{5}/_{8}$ inches, there are $3^{5}/_{8}$ inches of solid CMU blocks that can be used as spacers to form the beam seat (see chapter 7). They are compatible with the frictional connection to the recommended reinforcement. Again, because the facing element is not a structural component of the GRS abutment, any facing element can be selected.

It is important to use the actual dimensions of the specified facing element in designing the layout and details of the IBS. An advantage of the generic CMU block is that it is lightweight (about 42 lb), making it easy to place; however, depending on the combination of the CMU block and backfill, there may be some difficulty maintaining face alignment with lighter blocks. The nature of this type of concrete modular block also allows flexibility related to design modifications that can result from unforeseen field conditions. As seen in figure 7 and figure 8, the reinforcement extends directly between each layer of CMU blocks to create a simple frictional connection. Figure 8 and figure 9 also show split-faced corner blocks that can be used so that the exposed sides are aesthetically similar.

Source: FHWA.

Figure 7. Photo. Split-faced CMU blocks.



Source: FHWA.

Figure 8. Photo. Top view of split-faced CMU blocks.

Source: FHWA.

Figure 9. Photo. Split-faced corner CMU.

Another common facing element are SRW blocks (figure 10). They come in many shapes and sizes. Some of these units are designed to have a mechanical connection to the reinforcement; however, the use of these types of connection is not necessary. As illustrated in figure 10, SRW blocks typically require the wall to be battered. Depending on the abutment layout with respect to the wing walls, a battered face may require that the SRW units be trimmed on each course to maintain the wall face position. Additionally, a batter slightly increases the bridge span length, which should be considered in the design and cost of the project.

Figure 10. Photo. GRS abutment constructed with an SRW block facing.

The CMU or SRW units should be produced for the specific environment depending on the requirement for freeze-thaw durability and their subsequent availability. Figure 11 shows a U.S. map that was developed by the authors of this manual based off a figure from *Segmental Retaining Walls Best Practices Guide for the Specification, Design, Construction, and Inspection of SRW Systems*.[25] It is divided into three temperature zones, as generated by the National Concrete Masonry Association based on average daily temperatures from weather data.[25] The climate zone for a particular project (table 4) should be determined using the 30-year average low temperature from December to February at the site.

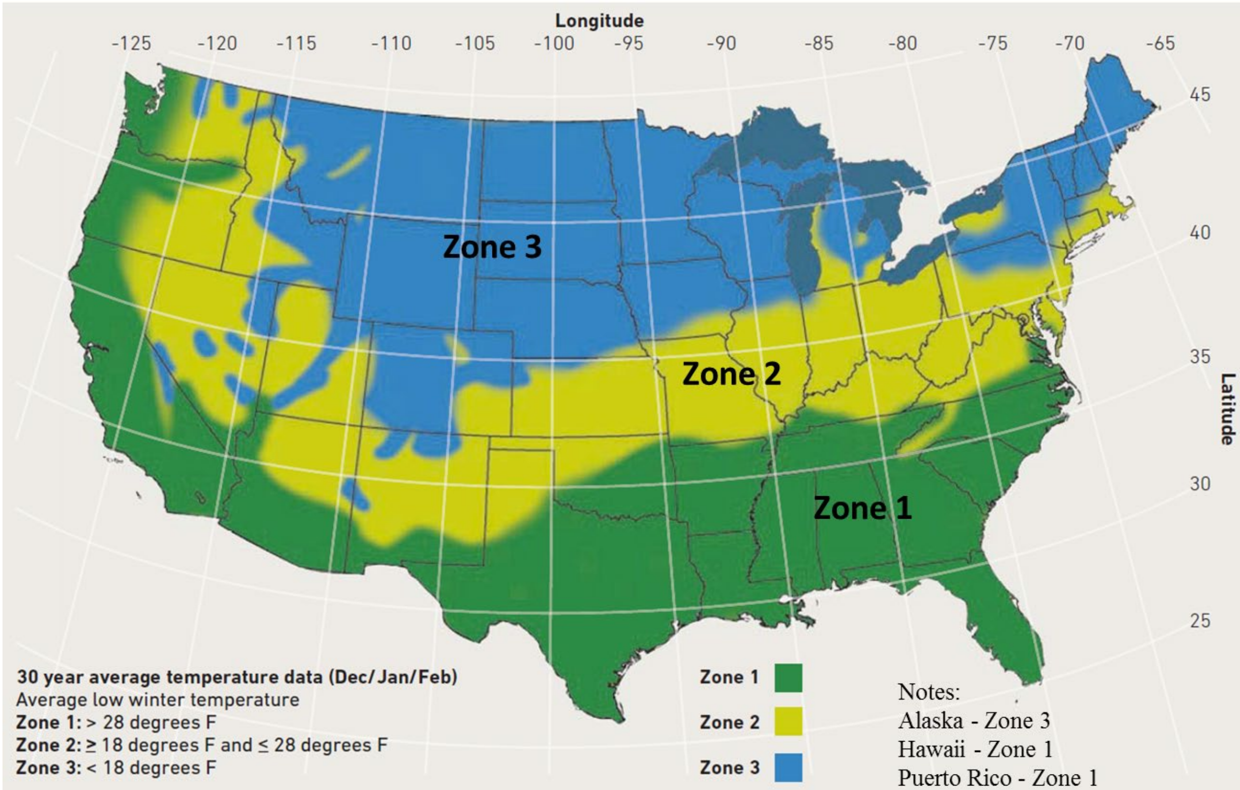


Figure 11. Illustration. Climate exposure zones for freeze-thaw durability of dry-cast modular blocks.[25]

Table 4. Climate exposure zone classification.

Zone	Exposure	Average Low Winter Temperature (°F)
1	Negligible	> 28
2	Moderate	18–28
3	Severe	< 18

Table 5, which was created based on information found in *Segmental Retaining Walls Best Practices Guide for the Specification, Design, Construction, and Inspection of SRW Systems*, provides specifications for compressive strength and freeze-thaw durability testing of modular blocks for each of the zone temperature zones.[25] In colder climates, a freeze-thaw test according to ASTM C1262 should be conducted to assess the durability of the concrete modular block.[26] One method to ensure the overall quality of the CMU or SRW units is to review the QA/QC process of a particular producer.

Table 5. Compressive strength and freeze-thaw durability recommendations.

Zone	Exposure	Minimum Compressive Strength	Minimum Freeze-Thaw (Durability Criteria)
1	Negligible	CMU per ASTM C90[27] SRW per ASTM C1372[28]	No freeze-thaw testing required.
Units Not Exposed to De-icing Salts			
2	Moderate	3,000 psi	Less than 1 percent weight loss after 100 cycles for 5 of 5 specimens or less than 1.5 percent weight loss after 150 cycles for 4 of 5 specimens tested using ASTM C1262 in water.[26]
3	Moderate	4,000 psi and maximum water absorption of 5 percent	Less than 1 percent weight loss after 100 cycles for 5 of 5 specimens or less than 1.5 percent weight loss after 150 cycles for 4 of 5 specimens tested using ASTM C1262 in water.[26]
Units Exposed to De-icing Salts			
2	Severe	4,000 psi and maximum water absorption of 5 percent	Less than 1 percent weight loss after 20 cycles for 5 of 5 specimens or less than 1.5 percent weight loss after 30 cycles for 4 of 5 specimens tested using ASTM C1262 in 3 percent saline solution.[26]
3	Severe	4,000 psi and maximum water absorption of 5 percent	Less than 1 percent weight loss after 40 cycles for 5 of 5 specimens or less than 1.5 percent weight loss after 50 cycles for 4 of 5 specimens tested using ASTM C1262 in 3 percent saline solution.[26]

There are several types of CMU that are commonly used in GRS-IBS construction: solid face, hollow core, and corner block. Typically, solid block is specified for below-grade construction where the block integrity may be affected due to construction activities or extreme events. If using an SRW unit, a solid block is not needed depending on the front wall thickness of the unit; however, because the face of CMUs is relatively thin (about $1^5/_8$ inch), solid blocks are preferred in this location. Solid CMU blocks weigh about 82 lb and are often smooth-faced; however, split-faced units are available by some manufacturers. By comparison, standard split-faced hollow core CMUs weigh about 42 lb, making them easier to place during construction. All of these CMU blocks come in the standard dimensions previously described.

3.5 MISCELLANEOUS MATERIALS

The three main materials involved in GRS construction are the backfill, the geosynthetic reinforcement, and the facing element. Other miscellaneous materials that are also necessary during construction include the following:

- **Concrete block wall fill**: Concrete block wall fill, along with rebar, is used to fill in and bind together the top three courses of facing blocks (see chapter 7). It is also used for runoff coping and, if necessary, to connect the wing wall to the abutment face when a vertical seam is located at the corners. The concrete used should be class A concrete with a compressive strength of 4,000 psi.

- **Rebar**: No. 4 rebar (0.5-inch diameter), preferably epoxy-coated, is used in the concrete block wall fill to pin the top three courses of facing blocks. If necessary, it can also be used to connect the wing wall to the abutment face at the corners (see chapter 7).

- **Flashing**: Optional flashing (e.g., aluminum flashing) can be used for two main purposes: (1) to serve as a drip edge within the clear space depending on the superstructure type (e.g., noncomposite adjacent box beams), which can help shed potentially corrosive fluids off the dry-cast block as a precaution, and (2) to prevent animals from burrowing into the abutment (see chapter 7). Typical dimensions of the aluminum fascia are 4 by 1.5 inches. The use of flashing is a decision left to the engineer.

- **Foam board**: Rigid foam insulation boards are used to help build the beam seat and create the setback. It also provides a bearing buffer between the superstructure and the wall face (see chapter 7). As the beams settle, there will be pressure on the foam board; therefore, the compressive strength of the foam should be less than the pressure produced by the superstructure to allow it to compress. The typical foam board used is extruded polystyrene, which is a common thermal insulation that is usually blue or pink. Expanded polystyrene insulation, which is typically white, is another commonly available foam board. A single foam board is usually 2 inches thick and 12 inches wide.

- **Corrosion protection**: A corrosion protection package is recommended for concrete and steel beam ends that will be embedded within the GRS abutment (figure 12 through figure 14). Many coating systems can be more efficiently installed with better quality in the shop than onsite. Any form of approved corrosion protection is possible and will not impact the GRS itself. Bitumen can also be shop-installed to prevent corrosion of the embedded concrete. Other methods can also be used to protect the embedded beams, such as polyurea spray coatings or other similar polymer coating systems.

- **Waterproof membrane**: A waterproof bridge deck membrane is recommended to prevent water infiltration through the pavement layer into the superstructure. The membrane should extend at least 3 ft from the deck onto the approach (figure 15).

Copyright: Definance County, OH.

Figure 12. Photo. Bitumen coating on concrete beam ends.

Copyright: Definance County, OH.

Figure 13. Photo. Bitumen coating under voided slabbeam.

Source: FHWA.

Figure 14. Photo. Coating over weathering steel girders.

Source: FHWA.

Figure 15. Photo. Waterproof membrane extending over adjacent box beams onto the integrated approach.

CHAPTER 4. DESIGN METHODOLOGY

4.1 OVERVIEW OF GRS-IBS DESIGN

Over the past 40 years, GRS technology has been used to build walls, shallow foundations, culverts, bridge abutments, and rock fall barriers. The technology has also been used to stabilize slopes and repair roadways. This chapter focuses on the LRFD methodology used for GRS-IBS. Note that the design of standalone GRS abutments follows the same procedure presented herein, just without the other components of the IBS (e.g., RSF and integrated approach). While GRS abutments can be utilized on their own with good performance, it is recommended to incorporate both the RSF (unless bearing on bedrock) and integrated approach in GRS bridge projects to ensure a jointless bridge system and better integrate the approach roadway with the bridge to mitigate the bump at the end of the bridge. The design and use of additional features and add-ons that are found on other types of bridge systems, such as bearings, approach slabs, and sleeper slabs, are not included because they are not necessary elements of the IBS; however, attention should be made to State and local transportation agency policies and specifications.

A general and identifying feature of the GRS-IBS design is a composite built with alternating layers of compacted granular fill material and closely spaced reinforcement (i.e., less than or equal to 12 inches). In nearly all of the GRS abutments built in the United States, either full-scale experiments or in-service structures were designed with a nominal 8-inch layered system. Most GRS abutments have been designed and constructed with a concrete modular block facing, but, as noted in chapter 3, many different types of facing elements are possible with GRS. The choice of facing will not change the basic design methodology presented.

An interdisciplinary team primarily of structural, geotechnical, and hydraulic engineers should be fully engaged in bridge scoping, design, and construction processes. The scoping and design process starts with establishing the project requirements from which the feasibility and preliminary geometry of a GRS-IBS is determined for its environment. Once the geometry is defined, it is then evaluated against external and internal modes of failure. An iterative process is used to assess the geometry and make adjustments as necessary to facilitate construction and ensure long-term performance. Note that economy should also be a consideration when evaluating each design alternative (e.g., deeper embedment versus larger footing).

The design of GRS-IBS is based on the following:

- The spacing of the reinforcement (12 inches or less) is a principal factor in the design and performance of GRS-IBS.
- GRS is a composite material that is internally stabilized.
- Both the compacted granular fill and the reinforcement layers strain laterally together in response to vertical stress until the system approaches a failure condition.
- A GRS composite is not supported externally, and, therefore, the facing system is not considered a structural element in design.

- Lateral earth pressure at the face of a GRS mass (i.e., thrust) is not significant, eliminating connection failure as a possible limit state.

- The facing elements are typically frictionally connected to the geosynthetic reinforcement.

- Under the prescribed granular fill and reinforcement conditions, reinforcement creep is not a major concern for the applied in-service loads. Therefore, an individual reduction factor for reinforcement creep is not necessary. Creep is accommodated within the prescribed reduction factor used for long-term design, which also accounts for durability and installation damage.

As described in greater detail in subsequent sections of this manual, GRS-IBS design and construction procedures follow from these basic assumptions and principles. Note that the design of the superstructure supported by GRS abutments or integrated with the IBS is no different from bridges supported by other foundation types. Superstructure design is not addressed in this manual and should follow the guidelines outlined in the *AASHTO LRFD Bridge Design Specifications*.(29)

4.1.1 External Stability

A GRS abutment is a type of gravity structure; therefore, external stability must be evaluated. External stability checks for GRS include direct sliding, bearing resistance of the foundation soil, and global stability. Limiting eccentricity is considered a serviceability check and is not required for GRS abutments because GRS composites are relatively ductile compared to conventional gravity structures. Overturning about the toe, in a strict sense, is not a possible response to earth pressures at the back of the mass or loading on its top.(3,30) Other attributes of GRS abutments also tend to preclude overturning as a mode of failure. A bridge supported by GRS consists of two abutments supporting a superstructure; the superstructure functions as a strut to resist overturning. For the IBS, each GRS abutment also has the integrated approach above its heel, resisting the overturning mode of failure. Consequently, while direct sliding, bearing resistance, and global stability are evaluated in conventional ways, overturning is sometimes addressed by inspection as a serviceability check in the field. Observations have shown that the combination of vertical and lateral loads applied, as limited by analysis of direct sliding, bearing resistance, and global stability, does not cause excessive deformation at the face of the GRS mass or other undesirable performance. While this combination of unique features and behavior eliminates the need to limit eccentricity for completed GRS abutments, engineers may choose to analyze potential overturning during an intermediate phase of construction if it is suspected that the length of time needed for an overturning mechanism to develop will lapse.

4.1.2 Internal Stability

GRS is inherently internally stable because of the interaction between the soil and the reinforcement layers. The strength and stiffness of GRS depend on the unique combination of compacted soil, reinforcement, and facing element. Internal stability checks include bearing resistance, deformations, and required reinforcement strength.

4.1.2.1 Allowable Bearing Resistance

The bearing resistance of the GRS abutment can be determined either empirically or analytically. Empirically, the bearing resistance is found using a stress–strain curve specific to the composite tested. The composite properties are a function of the reinforcement type and granular fill material, as determined through a GRS performance test.(31) If a designer uses a combination of the materials previously tested, then the appropriate stress–strain curve can directly be used for design. If the designer decides to change the materials from those already tested, then a performance test can be performed to obtain an applicable stress–strain curve for the empirical method. Guidelines on how to conduct a performance test, along with stress–strain curves for various GRS composites, are given in *Geosynthetic Reinforced Soil Performance Testing: Axial Load Deformation Relationship*.(4)

Alternatively, the designer can predict the bearing resistance of the GRS abutment by using a semi-empirical equation based on traditional earth pressure theory and adjusted to fit measured laboratory and field data. The design equation is a function of reinforcement spacing, soil, geosynthetic strengths, and soil grain size. This procedure is based on the results of many full-scale experiments and verified using case history performance data collected on several in-service GRS structures more than 20 years old.(3)

4.1.2.2 Deformations

While GRS abutments are very strong, conventional practice is to limit nominal bearing pressures to 4,000 lb/ft^2 for the service limit state. For design service loads resulting in applied pressures larger than 4,000 lb/ft^2, the project-specific performance criteria for deformations must be checked against the applicable stress–strain curve resulting from a performance test. Otherwise, a bearing pressure can be prescribed semi-empirically to limit deformations depending on the performance criteria (discussed later in this chapter). The recommended performance criteria for GRS-IBS consist of a tolerable vertical strain of 1 percent of the abutment height and a lateral strain of 2 percent of the bearing width and setback; however, the owner may decide to limit or expand these criteria depending on the requirements for the project.

4.1.2.3 Reinforcement Strength

A typical default value for the ultimate reinforcement strength (T_f) needed in GRS abutments is 4,800 lb/ft; however, the specified reinforcement may be weaker or stronger depending on the project (see section 4.3.7.3 later in this chapter). A semi-empirical equation is provided in section 4.3.7.3.1 (see equation 40) to determine the required strength of the reinforcement in a GRS abutment. It is a function of the applied lateral pressures, backfill parameters, and reinforcement spacing. The equation serves as a failure envelope and determines the strength needed (which may be more or less than the initial default of 4,800 lb/ft) to prevent a strength limit failure of the GRS composite given the applied loads. Variability and long-term strength losses are also considered in the design.

Along with the strength check, a service limit state check is also needed to define the stiffness of the reinforcement. The strength at 2 percent reinforcement strain (according to ASTM D4595 for geotextiles or ASTM D6637 for geogrids) must satisfy the design requirements discussed later in this chapter.[15,16] Note that the strength at 2 percent strain is not equivalent to the load on the reinforcement under service conditions; the actual load will vary depending on the strength and stiffness of the reinforcement specified. This service limit criterion ensures that the actual load on the reinforcement is less than what is calculated to limit lateral strain.

4.2 LRFD

Note that the design philosophy illustrated in this chapter is based on LRFD. It is FHWA policy that design for all Federal aid-funded projects be conducted using LRFD. Guidelines to design GRS-IBS in an Allowable Stress Design (ASD) format are presented in appendix B. In LRFD, several limit states must be evaluated, including strength, service, extreme event, and fatigue. This chapter focuses on the Strength I and Service I limit states for GRS abutment design.

For LRFD, AASHTO defines the various load factors and load combinations that need to be considered in the design of bridge and transportation structures.[29] Table 6 and table 7 reproduce these load combinations and load factors. Resistance factors for external stability are also given by AASHTO for mechanically stabilized earth (MSE), gravity, and semi-gravity walls, which are adopted for the GRS design presented herein.[29] For direct sliding, a sliding resistance factor ($\Phi\tau$) is included. For sliding of soil on soil, $\Phi\tau$ is equal to 1.0. For bearing resistance, the resistance factor (Φ_{bc}) is equal to 0.65. Finally, for global stability, the resistance factor is 0.65.

Table 6. Typical load combinations and load factors.[29]

Load Combination Limit State	DC, DD, DW, EH, EV, ES EL, PS, CR, and SH	LL, IM, CE, BR, PL, and LS	WA	WS	WL	FR	TU	TG	SE	EQ*	IC*	CT*	CV*
Strength I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Strength II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Strength III	γ_p	—	1.00	1.40	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Strength IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—
Strength V	γ_p	1.35	1.00	0.40	1.00	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Extreme Event I	γ_p	γ_{EQ}	1.00	—	—	1.00	—	—	—	1.00	—	—	—
Extreme Event II	γ_p	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00
Service I	1.00	1.00	1.00	0.30	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Service II	1.00	1.30	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—
Service III	1.00	0.80	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Service IV	1.00	—	1.00	0.70	—	1.00	0.50/1.20	—	1.0	—	—	—	—
Fatigue I—LL, IM, and CE only	—	1.50	—	—	—	—	—	—	—	—	—	—	—
Fatigue II—LL, IM, and CE only	—	0.75	—	—	—	—	—	—	—	—	—	—	—

—This term is not included in the load combination.
*Use one at a time.
γ_p = load factor for permanent loading; γ_{TG} = load factor for temperature gradient; γ_{SE} = load factor for settlement
DC = Dead load (DL) of structural components and nonstructural attachments; DD = downdrag force; DW = DL of wearing surfaces and utilities; EH = horizontal earth pressure load; EV = vertical pressure from DL of earth fill; ES = earth surcharge load; EL = miscellaneous locked-in force effect; PS = secondary forces from post-tensioning for strength limit states or total prestress forces for service limit states; CR = force effects due to creep; SH = force effects due to shrinkage; LL = vehicular live load; IM = vehicular dynamic load allowance; CE = vehicular centrifugal force, BR = vehicular braking force; PL = pedestrian LL; LS = LL surcharge; WA = water load and stream pressure; WS = wind load on structure; WL = wind on LL; FR = friction load; TU = force effect due to uniform temperature; TG = force effect due to temperature gradient; SE = force effect due to settlement; EQ = earthquake load; IC = ice load; CT = vehicular collision force; and CV = vessel collision force.

Table 7. Load factors for permanent loads of interest to GRS design.[29]

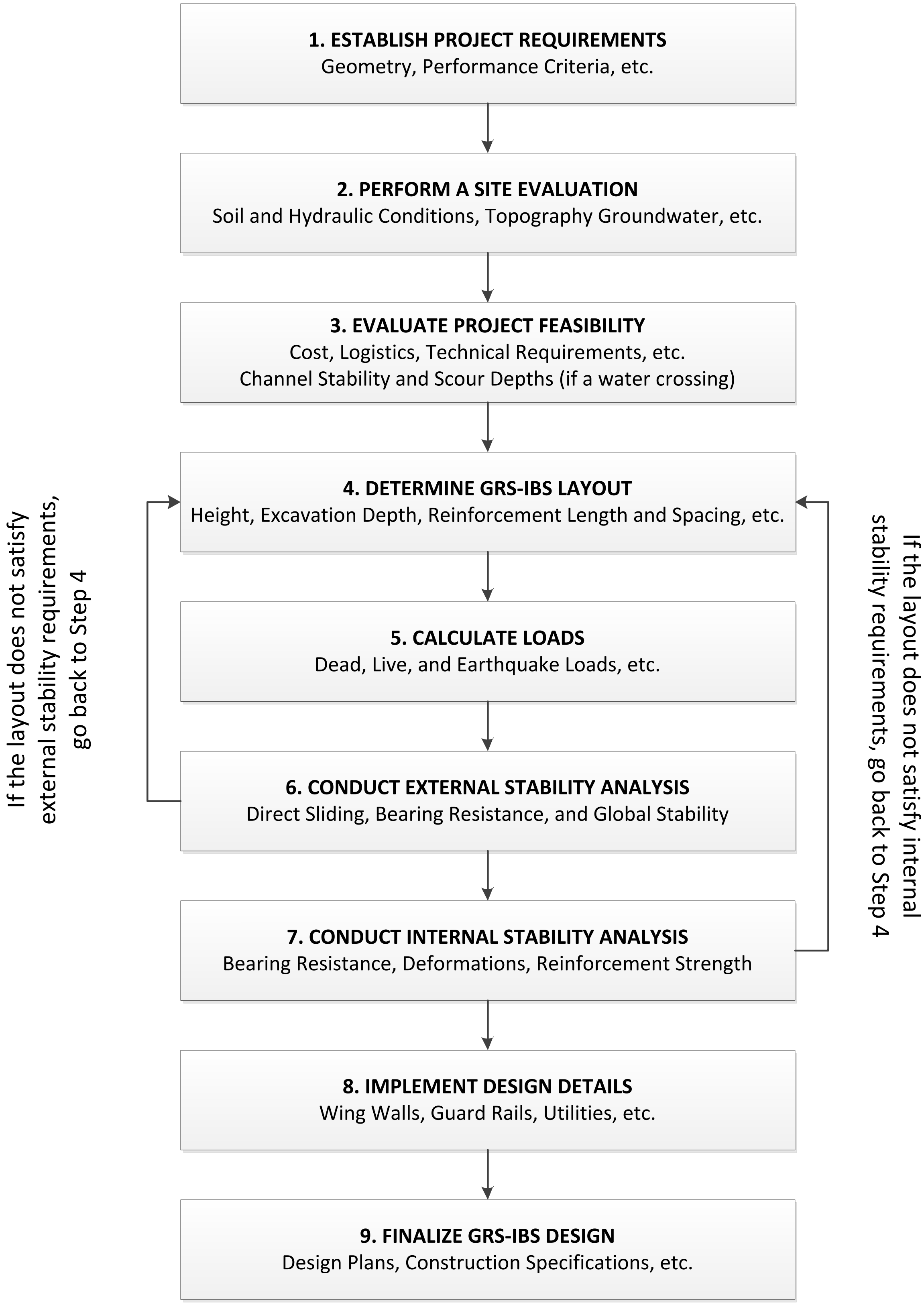
Type of Load, Foundation Type, and Method Used to Calculate Downdrag	Load Factor	
	Maximum	Minimum
DC: Component and Attachments	1.25	0.90
DC: Strength IV only	1.50	0.90
EH: Horizontal Earth Pressure		
Active	1.50	0.90
At-Rest	1.35	0.90
AEP for anchored walls	1.35	N/A
EV: Vertical Earth Pressure		
Overall Stability	1.00	N/A
Retaining Wall and Abutments	1.35	1.00
Rigid Buried Structure	1.30	0.90
Rigid Frames	1.35	0.90
Flexible Buried Structures other than Metal Box Culverts	1.95	0.90
Flexible Metal Box Culverts and Structural Plate Culverts with Deep Corrugations	1.50	0.90
ES: Earth Surcharge	1.50	0.75

N/A = not applicable.

The resistance factors recommended were, for the most part, calibrated to traditional ASD safety factors. Exceptions are the bearing resistance and required reinforcement strength of the GRS abutment, which have been statistically calibrated using the results of 35 large-scale GRS load tests; however, the results are consistent with the traditional ASD safety factors previously prescribed in the *Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide*.[1,4]

4.3 BASIC DESIGN STEPS FOR GRS-IBS

There are nine basic steps in the design of GRS-IBS (figure 16). The basic design guidelines are the same whether using LRFD (presented in this chapter) or ASD (presented in appendix B). Design examples are provided in appendix C. The following subsections describe the design guidelines for LRFD in further detail.



Source: FHWA.

Figure 16. Flowchart. Steps for GRS-IBS design.

4.3.1 Step 1—Establish Project Requirements

The following parameters must be defined:

- Geometry of abutment and wing walls.
- Height.
- Length.
- Batter (vertical or near vertical).
- Wall placement with respect to ground conditions (e.g., back slope and toe slope).
- Skew.
- Grade.
- Superelevation.
- Performance criteria.
- Tolerable movements, including vertical settlement of both the GRS abutment and the native foundation soil, lateral displacement, differential settlement, and angular distortion between abutments.
- Design life.
- Constraints including environmental, construction, scour, and stream stability.

4.3.2 Step 2—Perform a Site Evaluation

To properly assess conditions at the site, a site visit must be conducted. During this visit, the following must be performed by the agency and/or its designer:

- Study the existing topography with respect to the proposed GRS-IBS.
- Check any existing structures/roads for problems to aid in the assessment and design.
- Conduct a subsurface investigation. Refer to AASHTO's *Standard Practice for Conducting Geotechnical Subsurface Investigations* or FHWA's *Soils and Foundations Reference Manual* for more information.[32,33] The following should be investigated:
    - Foundation soil properties (e.g., unit weight of foundation soil (γ_f), friction angle of the foundation soil (ϕ_f), cohesion of the foundation soil (c'_f), undrained shear strength of foundation (c_u), compression index (C_C), recompression index (C_R), and coefficient of permeability (k)).

- Retained soil properties (unit weight of the retained backfill (γ_b) and friction angle of the retained backfill (ϕ_b)).

- Ground water conditions.

- Evaluate soil properties for the anticipated reinforced backfill (e.g., unit weight of the reinforced backfill (γ_r), friction angle of the reinforced backfill (ϕ_r), cohesion of the reinforced backfill (c_r), and the maximum grain size (d_{max})). In addition to the basic soil properties typically collected, d_{max} is necessary to evaluate the bearing resistance and required reinforcement strength of the GRS abutment. The gradation of the reinforced backfill is also important and should conform to the material specifications presented in chapter 3.

- Evaluate hydraulic conditions and scour potential. This should be accomplished through consultation with a qualified hydraulic engineer.

4.3.3 Step 3—Evaluate Project Feasibility

The feasibility of the project should be evaluated in terms of cost, logistics, technical requirements, and performance objectives. In particular, in the case of abutments for bridges constructed over water, the potential for scour, sedimentation, and/or channel instability must be evaluated in accordance with the policies and procedures of both FHWA and AASHTO. (See references 1, 5, 14, 29, and 34.)

It is necessary to determine the potential for scour at all bridges constructed over water. Adverse flow conditions generate complex hydraulics and increase the potential scour and stream instability at a bridge site. If the abutment may be impacted by scour, additional design requirements are necessary (see appendix D). These additional design requirements can be determined and implemented through a hydraulic and scour analysis of the site by a qualified hydraulic engineer. Once the risk and scour potential are determined, a countermeasure can be designed to protect the abutment against failure during a hydraulic event. A designed countermeasure will also protect the abutment from lateral channel migration that could undermine the foundation.

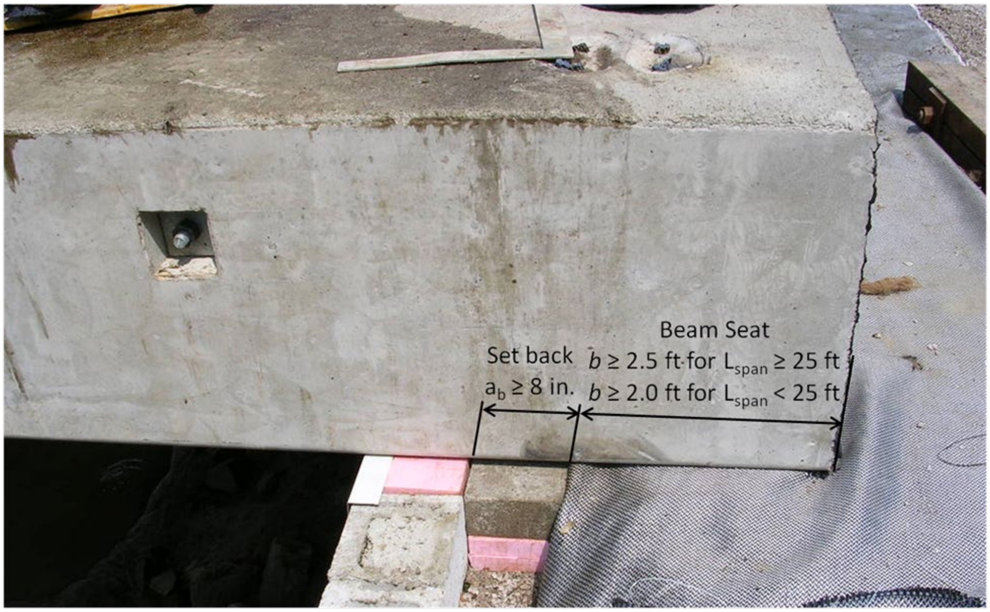
4.3.4 Step 4—Determine Layout of GRS-IBS

The layout of the GRS-IBS is ultimately based on site conditions (e.g., desired road alignment, right-of-way, geotechnical issues, and hydraulic considerations). A survey should be conducted to determine the location of the GRS abutments and the layout. The layout of the abutment face wall needs to coincide with the wing walls because the system is built from the bottom up, one course at a time. Both abutment and wing walls are built at the same time. The following steps shall be used to design the abutment:

1. Define the geometry of the abutment face wall and wing walls.

2. Lay out the abutment with respect to the superstructure, including any skew, superelevation, or grade requirements as follows:

- The recommended minimum bearing width (b) for the superstructure is 2.5 ft for span lengths (L_{span}) greater than or equal to 25 ft, as shown in figure 17. For L_{span} less than 25 ft, the minimum bearing width is 2 ft.



Source: FHWA.

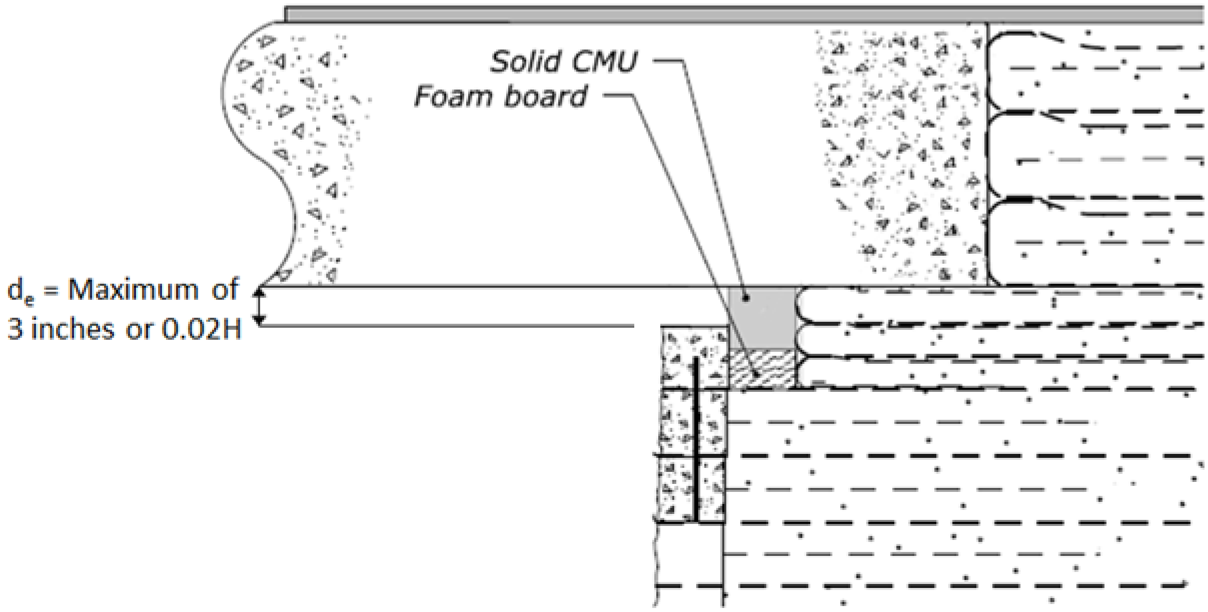
Figure 17. Photo. Beam seat and setback distances.

- If the superstructure consists of spread girders, a concrete slab or footing should be designed and placed on the GRS abutment and integrated with the girders to transfer a uniform bearing pressure. The footing shall be sized to support the applied loads and have the recommended minimum bearing width as noted previously. In addition, a backwall and cheek walls should be designed to create a solid form that the integrated approach can be built against.

3. Account for setback and clear space to calculate the elevation of the abutment face wall and the span length of the bridge as follows:

    - The setback distance between the back of the face and the beam seat (a_b) should be nominally 8 inches or more, as shown in figure 17.

    - The minimum clear space (d_e), which is defined as the distance from the top of the uppermost facing block to the bottom of the superstructure, should be 3 inches or 2 percent of the abutment height (H) (i.e., height of the GRS abutment including the clear space distance), whichever is greater (figure 18). The gap is to ensure that the superstructure does not bear directly on the facing block due to long-term settlement or an unforeseen event.



Source: FHWA.

Figure 18. Illustration. Clear space distance.

4. Determine the depth and volume of excavation necessary for construction. A GRS abutment is inherently stable and therefore can be built with a truncated base to reduce the excavation. Truncation also reduces the quantity of backfill and reinforcement needed for the project, thus reducing cost. The depth and volume should be determined as follows:

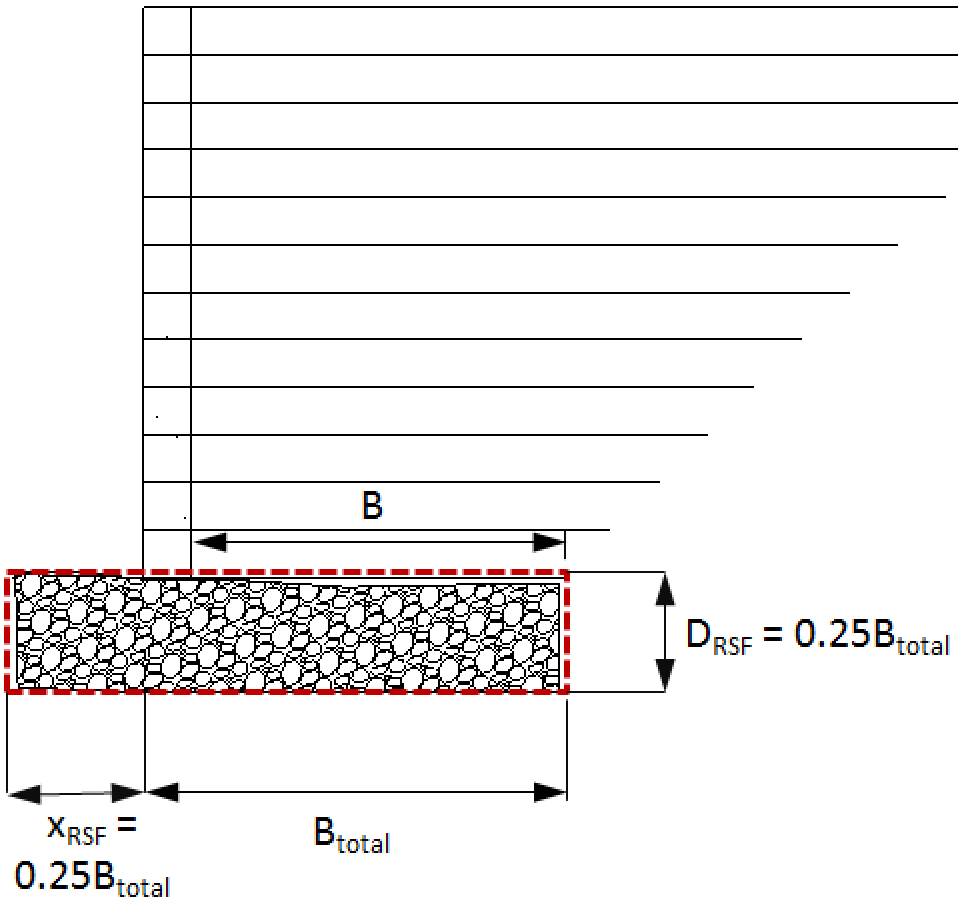
    - For L_{span} greater than or equal to 25 ft, a minimum wall base width of 6 ft, including the total base width of the GRS abutment including the width of the facing (B_{total}), should initially be chosen (figure 19). For L_{span} less than 25 ft, a minimum B_{total} of 5 ft should initially be chosen. Whether a cut or fill situation, there should be a minimum base-to-height ratio (B/H) of 0.3.

    - If b is larger than 5–6 ft, depending on the span length, then the base length of the reinforcement not including the wall face (B) should be a minimum of the width of the bearing area plus twice the setback distance ($b + 2a_b$).

    - Excavation of one-quarter of the total width of the base of the abutment, including the length of the RSF in front of the abutment wall face (x_{RSF}), should be made in front of the face of the wall to accommodate for construction of the RSF (figure 19).

    - For water crossings, the base of the GRS abutment should be located at the computed scour depth (see appendix D). If an IBS, the RSF is founded below the abutment at a depth discussed in step 5.

    - The depth of the RSF (D_{RSF}) should equal 18 inches minimum, up to one-quarter the total width of the base of the GRS abutment including the facing (figure 19); however, additional excavation may be necessary depending on the soil conditions (e.g., compressible soils) and should be determined by the engineer. If bedrock (or soil defined as very stiff) is relatively shallow, an RSF may not be necessary. In this case, the abutment can either be keyed in to the bedrock, or a thin layer of concrete or structural backfill can be placed to provide a level working surface to place the facing elements (see chapter 7). Constructability should be considered with regard to the total depth of the RSF should the base length of reinforcement be large.



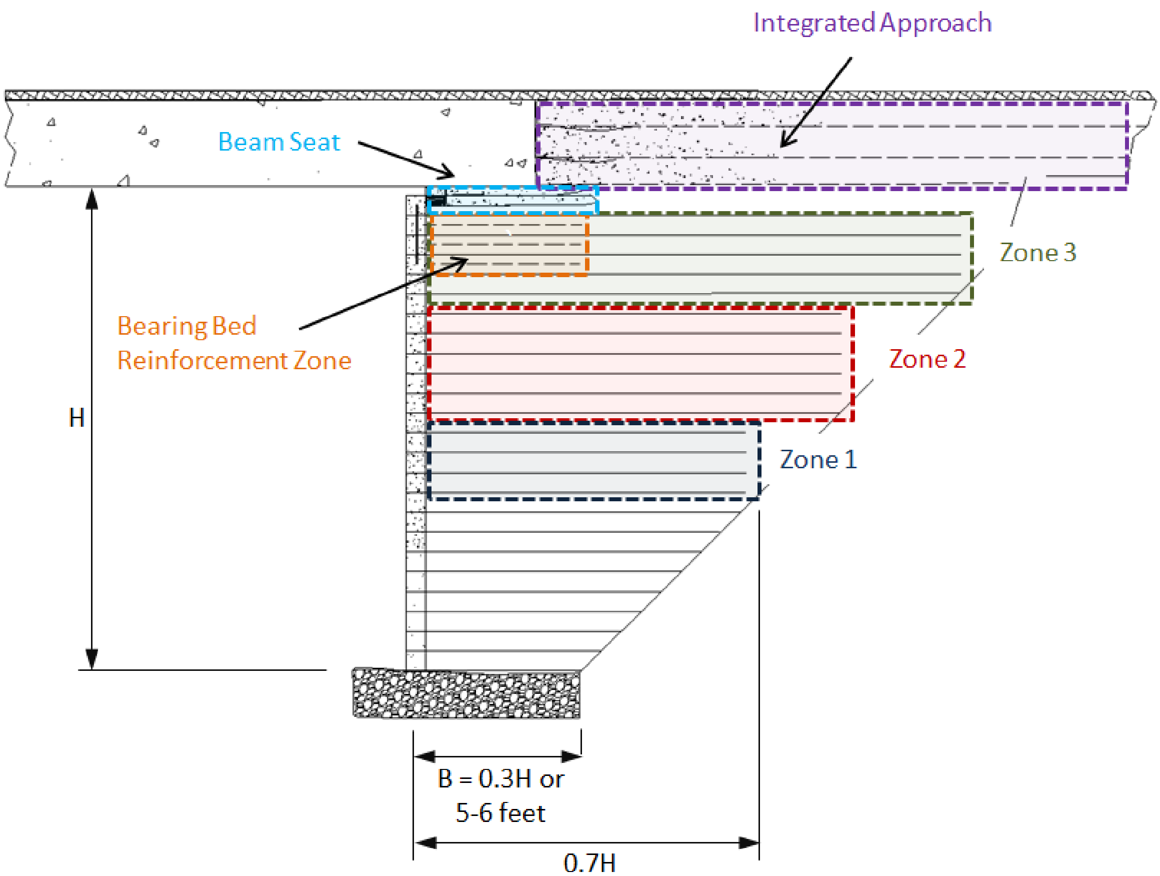
Source: FHWA.

Figure 19. Illustration. Abutment base width and RSF dimensions.

5. Select the length of reinforcement throughout the height of the abutment as follows:

    - As previously mentioned, the minimum reinforcement length at the lowest level should extend B_{total} and have a minimum B/H (not including the facing) of 0.3, or 5–6 ft.

    - Once the base length of the reinforcement is chosen, the reinforcement schedule should follow the cut slope, if applicable, up to a B/H of 0.7. From there, the reinforcement length can get progressively longer in reinforcement zones (figure 20). Not every layer will need to extend fully to the cut slope. The progressively longer lengths of reinforcement serve to improve the quality of construction and overall stability of the GRS abutment. The reinforcement zones also serve to provide a gradual transition from the substructure to the superstructure. The exact details of the reinforcement zones, such as number of layers and length, are left to the designer, as this will not have a significant impact on performance.



Source: FHWA.

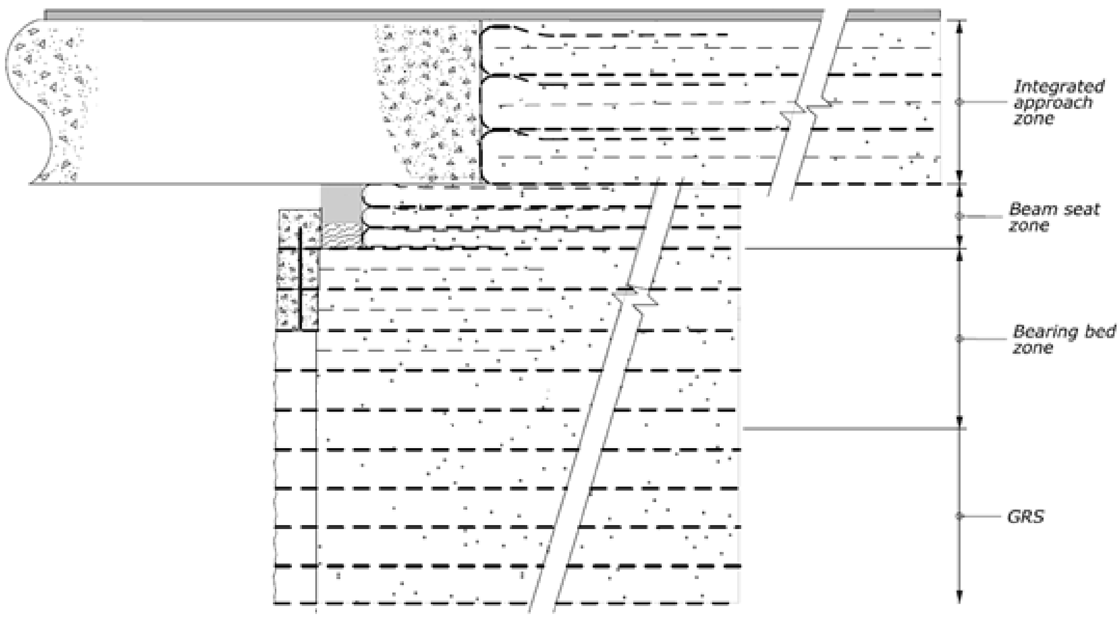
Figure 20. Illustration. Reinforcement schedule for a GRS abutment.

- For cut slopes flatter than 1:1, reinforcement zones with lengths larger than 1.0*H* may not be necessary. The backfill between the reinforced zone and the cut slope or retained soil must be the same structural backfill as the reinforced fill and compacted to the same effort (see chapter 3 and chapter 7). The reinforcement spacing should be no more than 12 inches throughout each zone.

6. Add a bearing bed reinforcement zone underneath the beam seat to ensure adequate backfill compaction, support the increased loads due to the bridge, and reduce lateral deformations at the face (figure 20). The bearing bed serves as an embedded footing in the reinforced soil abutment. The spacing of the bearing bed reinforcement zone directly underneath the beam seat should be, at a minimum, half the primary spacing (e.g., for an 8-inch primary spacing, the bearing bed reinforcement spacing will equal 4 inches).

   In general, the bearing bed reinforcement should extend twice the setback plus the width of the beam seat ($b + 2a_b$). The depth of the bearing reinforcement zone is determined based on internal stability design for required reinforcement strength (see section 4.3.7.3.3); however, a minimum of three bearing bed reinforcement layers should be specified (figure 20).

7. Place a beam seat above the bearing bed reinforcement zone to support the superstructure (figure 20). The purpose of the beam seat is twofold: (1) to ensure that the superstructure bears on the GRS abutment and not the wall facing and (2) to provide the necessary clear space between the superstructure and the wall face (see step 3). In general, the total thickness of the beam seat consists of two to three 4-inch lifts of wrapped-face GRS at the established setback distance (figure 21). The exact height will depend on the required clear space distance for the superstructure. The minimum reinforcement length shall be the same as that placed in the bearing bed. The superstructure, or its footing, can then be placed directly on the beam seat.



Source: FHWA.

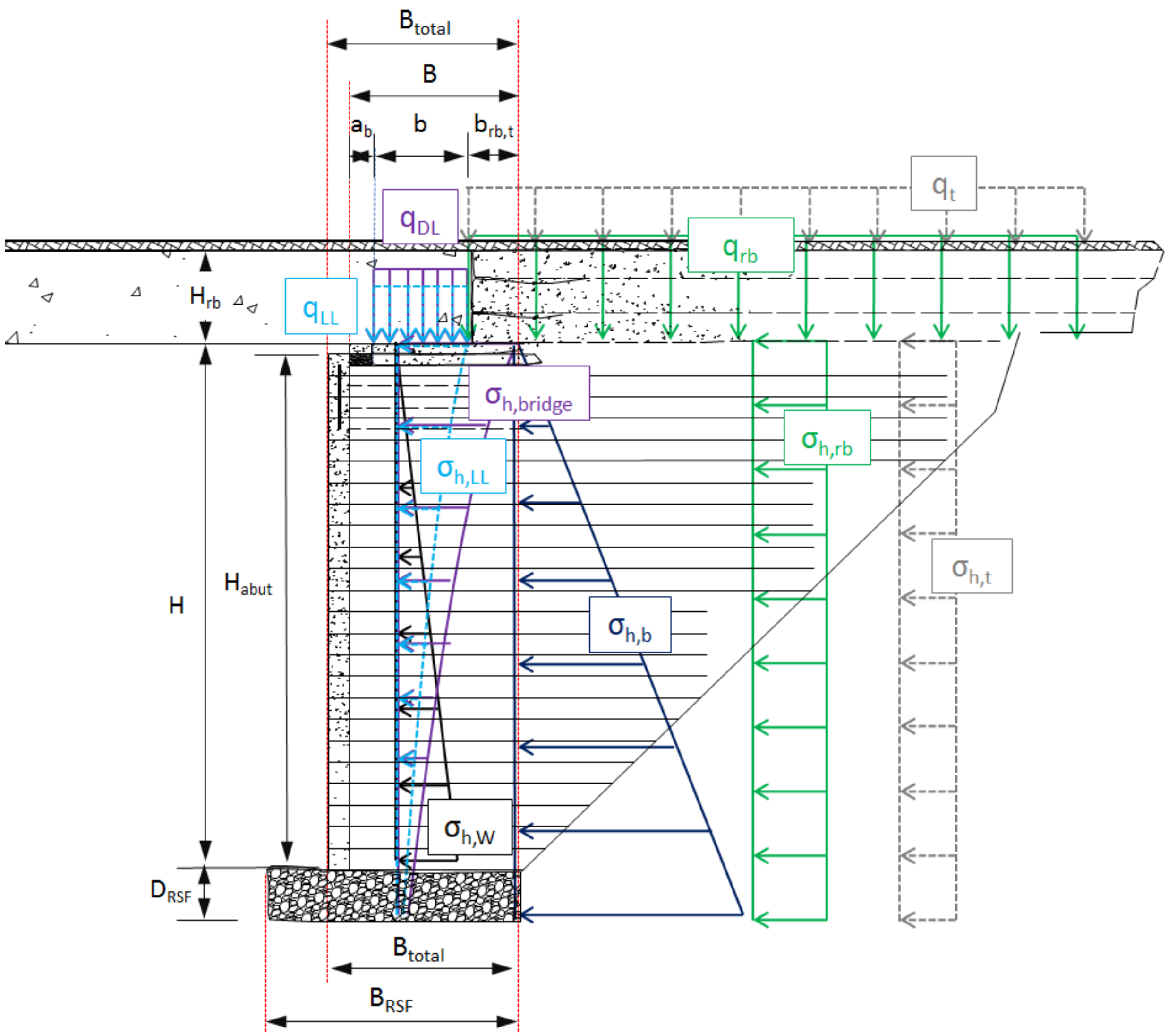
Figure 21. Illustration. Bearing bed, beam seat, and integrated approach details.

8. Extend the reinforcement layers of the integrated approach behind the superstructure or backwall to the cut slope, if applicable, with the exception of the top reinforcement layer. The top two layers should extend beyond the cut slope by 3 ft to tie the abutment with the existing grade, limit the development of a tension crack at the cut slope and reinforced soil interface, and blend the approach way on to the roadway to create a smooth transition.

   The number of reinforcement layers in the integration zone depends on the depth of the superstructure, but each wrapped layer should be no more than 12 inches in height, with secondary layers placed at no more than 6 inches high to aid compaction. Additional construction work is needed to integrate the substructure with the superstructure within the integration zone, as described in chapter 7.

4.3.5 Step 5—Calculate Applicable Loads and Pressures

The nominal pressures and loads (permanent and transient) applied on the GRS abutment and RSF should be calculated and then factored appropriately. The most common pressures (which may be resolved into forces) on GRS-IBS for stability computations are depicted in figure 22.



Source: FHWA.

Figure 22. Illustration. Vertical and lateral pressures on a GRS abutment.

Common nominal vertical and lateral service pressures on a GRS abutment as well as remaining variables shown in figure 22 are defined as follows:

- q_{DL} = Equivalent superstructure DL pressure.
- q_{LL} = Superstructure LL pressure.
- q_{rb} = Surcharge due to the structural backfill of the integrated approach (termed "road base" herein).
- q_t = Roadway LL surcharge due to traffic.
- $\sigma_{h,bridge}$ = Lateral pressure due to bridge DL surcharge within GRS.

- $\sigma_{h,LL}$ = Lateral stress distribution due to the equivalent superstructure superstructure LL pressure.
- $\sigma_{h,rb}$ = Lateral pressure due to the road base surcharge within GRS.
- $\sigma_{h,b}$ = Equivalent lateral stress distribution due to the retained soil behind the GRS soil abutment.
- $\sigma_{h,t}$ = Lateral pressure due to traffic LL surcharge.
- $\sigma_{h,W}$ = Lateral stress distribution due to the weight of the reinforced backfill in the GRS abutment.

4.3.5.1 Lateral Pressures

Lateral earth pressure can be calculated according to classical soil mechanics for active earth pressure. For vertical walls, the active earth pressure coefficient (K_a) is calculated according to equation 1.

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \tan^2\left(45 \text{ degrees} - \frac{\phi}{2}\right) \tag{1}$$

Where ϕ is the friction angle of interest (e.g., substitute the friction angle of the retained backfill (ϕ_b) when calculating the coefficient of active earth pressure for the retained backfill (K_{ab}) for the retained soil). $\sigma_{h,W}$ is found using Rankine's active stress condition shown in equation 2.

$$\sigma_{h,W} = \gamma_r z K_{ar} \tag{2}$$

Where:

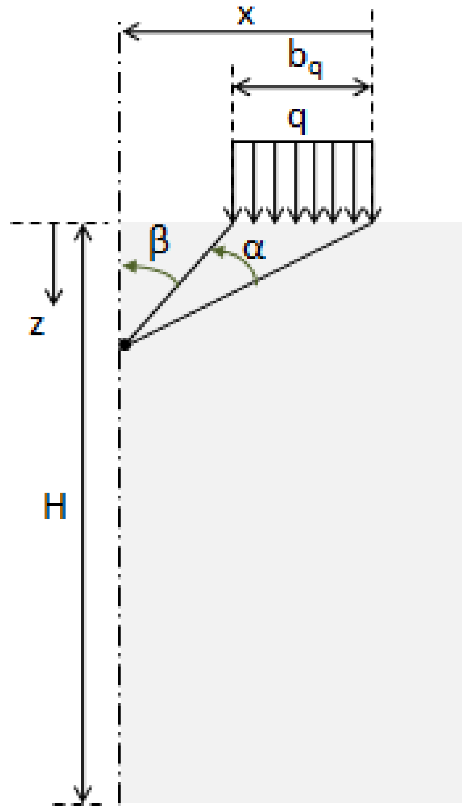
γ_r = unit weight of the reinforced backfill.
z = depth from the top of the wall.
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

$\sigma_{h,t}$ and $\sigma_{h,rb}$ are solved for according to equation 3 and equation 4, respectively. Note that both equations assume that the loading is continuous across the retained soil.

$$\sigma_{h,t} = q_t K_{ab} \tag{3}$$

$$\sigma_{h,rb} = q_{rb} K_{ab} \tag{4}$$

When loads are not continuous across the GRS abutment or retained soil (e.g., loads from the bridge), the lateral pressure is evaluated at the location of interest (a distance from the edge of the load to the point of interest for lateral pressure (x); see figure 23 based on the Boussinesq theory for stress distribution through a soil mass for an area transmitting a uniform surcharge.(35) For required reinforcement strength calculations, the assumed location of interest is directly underneath the beam seat centerline (e.g., $x = b_q/2$), where b_q is the width of the surcharge loading for the bridge DL and LL.



Source: FHWA.

Figure 23. Illustration. Boussinesq load distribution with depth for a strip load.

Where:

q = surcharge load (e.g., q_{DL} for the bridge DL pressure).
α = angle between the projections of the inner and outer edge lines of the surcharge to the wall face (radians).
β = angle between wall face and projection of the midline of the surcharge to the wall face.

Using the Boussinesq theory, the lateral pressure due to bridge surcharge loading ($\sigma_{h,q}$) is calculated according to equation 5.

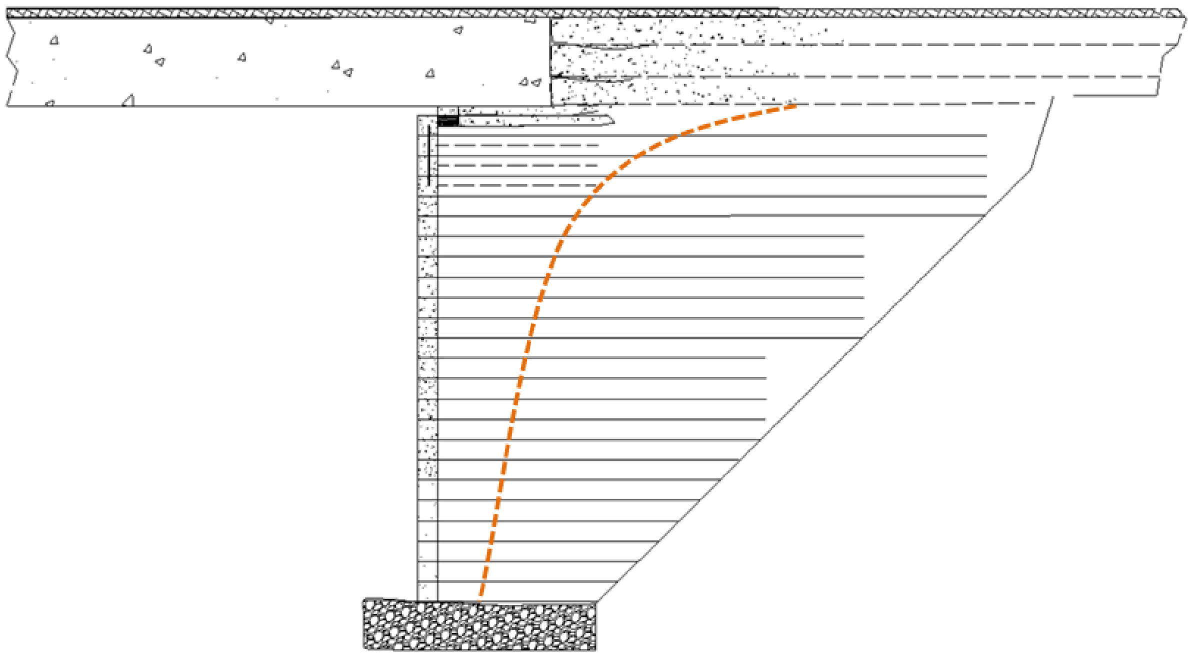
$$\sigma_{h.q} = \frac{q}{\pi}[\alpha + \sin(\alpha)\cos(\alpha + 2\beta)]K_a \tag{5}$$

Note that α and β must be input in radians in equation 5. In addition, α (figure 23) is found using equation 6, while β (figure 23) is found in equation 7.

$$\alpha = \tan^{-1}\left(\frac{x}{z}\right) - \beta \tag{6}$$

$$\beta = \tan^{-1}\left(\frac{x - b_q}{z}\right) \tag{7}$$

The lateral pressure in the GRS abutment due to the superstructure DL and LL has a trend similar to that shown in figure 24, where the stress is highest at the top of the GRS abutment and lowest at the base. The unfactored and factored DL and LL from the superstructure will be provided by the bridge designer based on *AASHTO LRFD Bridge Design Specifications*.[29]



Source: FHWA.

Figure 24. Illustration. Internal lateral stress in the GRS abutment due to bridge loading.

Note that other load distributions are available besides Boussinesq. For example, Westergaard may be more applicable to a GRS mass than Boussinesq; however, it gives lower stresses than Boussinesq, and therefore, using Boussinesq will provide a more conservative estimate of stresses.

4.3.5.2 DLs

The GRS abutment must support a variety of permanent DLs, including those resulting from the bridge superstructure itself, the road base behind the superstructure (or integrated approach), and, when considering the stability of the RSF, the weight of the facing.

4.3.5.2.1 Bridge

In a GRS-IBS design with adjacent concrete box beams, the bridge superstructure bears directly on the GRS abutment. For superstructures with spread girders, a footing (which bears directly on the GRS abutment) is necessary to ensure even load distribution on the GRS abutment. The DL design pressure on the bridge seat includes the DLs due to the bridge beams, asphalt, overlay, guardrail, and any other applicable permanent loads related to the superstructure.

4.3.5.2.2 Road Base

Behind the bridge beams, the road base is wrapped in geotextile (i.e., the integrated approach). The wrapped face controls lateral load from the road base on the beam or backwall but adds weight and lateral pressures to the GRS abutment.

4.3.5.2.3 Facing

The weight of the facing elements selected (W_{face}) per unit length adds pressure on the RSF and underlying foundation soil, but it also helps resist the applied lateral pressures. For a modular block facing, the weight can be calculated according to equation 8.

$$W_{face} = N_{block} \frac{W_{block}}{L_{block}} \tag{8}$$

N_{block} = number of blocks in a single column along the cross sectional height of the abutment.
W_{block} = weight of an individual facing block.
L_{block} = length of a facing block (e.g., 15.625 inches for a CMU).

4.3.5.3 LLs

There are two applications of LL that affect the design of GRS-IBS: (1) LL on the approach pavement and (2) LL on the superstructure. Both of these LLs are defined by AASHTO and should be appropriately quantified and factored by the design engineer.[29]

4.3.5.3.1 LL on the Approach Pavement

An LL surcharge (q_t) is used to account for the traffic load on the approach pavement leading up to the superstructure. Computation of this load consists of assuming an equivalent height of overburden for traffic surcharge (h_{eq}) of earth that would produce an equivalent lateral effect on the abutment resulting from the application of the traffic LL surcharge. The equivalent height of earth is dependent on the total height of the GRS abutment including the clear space (H) (table 8)

Table 8. Equivalent height of soil for vehicular loading on abutments.[29]

H **(ft)**	$\boldsymbol{h_{eq}}$ **(ft)**
5.0	4.0
10.0	3.0
≥ 20.0	2.0

4.3.5.3.2 LL on the Superstructure

The vehicular LL on the superstructure is currently determined by applying the HL-93 LL model to the superstructure.[29] This model consists of appropriately locating a design truck or design tandem in combination with a design lane load in each design lane of the bridge to create the maximum force effect at each abutment. The vehicular portion of the LL model is also amplified for dynamic load allowance (impact). The governing LL delivered by the bridge engineer is distributed to the abutment by multiplying by the number of design lanes and dividing by the beam seat bearing area.

4.3.5.4 Design Beam Seat Pressure

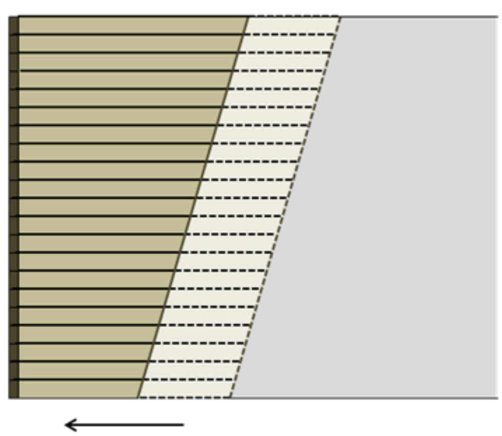
Adding the reaction due to bridge DL and LL per abutment will give the total load that the GRS abutment must support. Dividing this total load by the area of the beam seat will give the bearing pressure. For abutment applications, the service bearing pressure is often targeted to around 4,000 lb/ft^2; however, higher pressures are possible depending on the strength limit of the composite, but the tradeoff will be increased deformations. If these deformations are within the project criteria and the design meets other requirements, then higher combined service pressures can be applied.

4.3.6 Step 6—Conduct an External Stability Analysis

The external stability of GRS-IBS is evaluated by looking at the following potential external failure mechanisms:

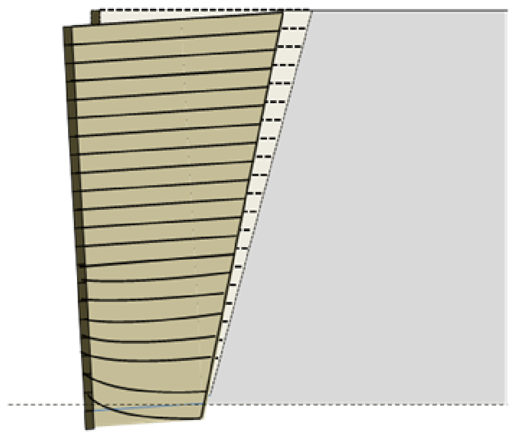
- Direct sliding (figure 25).
- Bearing capacity (figure 26).
- Global stability (figure 27).

In the external stability analysis presented, a simplistic method is provided whereby the reinforcement is assumed to be of uniform length throughout the height of the abutment, equal to the smallest base length, *B* (i.e., it does not include the more complex analysis with the reinforcement zones and trapezoidal wedge). This is a conservative measure, as the lateral stress will not be as high in reality as those computed with this simple alternative, especially if the cut slope is stable. More advanced computer analysis can also be used if the designer would like to take advantage of the entire GRS composite.



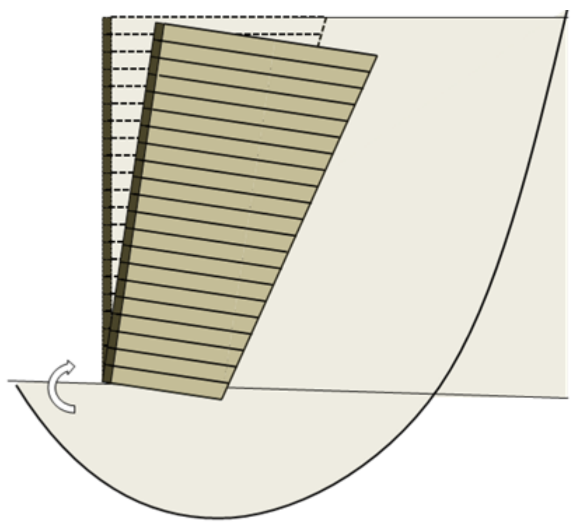
Source:FHWA.

Figure 25. Illustration. External stability—direct sliding.



Source: FHWA.

Figure 26. Illustration. External stability—bearing capacity.



Source: FHWA.

Figure 27. Illustration. External stability—global stability.

4.3.6.1 Direct Sliding

Lateral translation, or direct sliding, must be resisted for stability. For an IBS, direct sliding shall be evaluated at both the interface between the GRS abutment and RSF and between the RSF and the foundation soils. For a standalone GRS abutment, direct sliding shall be evaluated at the interface between the GRS abutment and the foundation soils.

4.3.6.1.1 Direct Sliding at the Base of the GRS Abutment

The LL surcharge on the approach pavement (q_t) is assumed to act only over the retained backfill and not the reinforced soil mass. The contribution of q_t (and superstructure LL pressure (q_{LL})) on the abutment is ignored because the loads are transient and cannot be counted on as stabilizing surcharges. The superstructure DL pressure (q_{DL}), however, is permanent and has a stabilization effect against direct sliding when considering an abutment. Because the integrated approach extends over the GRS abutment and the retained backfill, it acts to both stabilize and drive direct sliding. Regardless of whether an integrated approach is selected as part of the IBS or a traditional approach is selected with a standalone GRS abutment, contributions to both the driving force and to the resisting force from the approach road base must be taken into account because it is a permanent load.

The nominal lateral force behind the GRS abutment due to the retained backfill (F_b), the road base (F_{rb}), and the roadway LL surcharge (F_t) are calculated using equation 9 through equation 11, respectively.

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2 \tag{9}$$

$$F_{rb} = q_{rb} K_{ab} H \tag{10}$$

$$F_t = q_t K_{ab} H \tag{11}$$

Where:

γ_b = unit weight of the retained backfill.
K_{ab} = coefficient of active earth pressure for the retained backfill (equation 1).
H = height of the GRS abutment including the clear space distance.
q_{rb} = surcharge due to structural backfill of the integrated approach (i.e., road base).
q_t = roadway LL surcharge.

The total factored driving force for direct sliding calculations at the base of the GRS abutment (F_R) is calculated by summing each factored thrust force, as shown in equation 12. The load factors for the maximum horizontal earth pressure ($\gamma_{EH\,MAX}$) and traffic LL surcharge (γ_{LS}), which are determined using table 6 and table 7, are utilized for the retained backfill, road base, and traffic surcharges. Note that while the road base is a surcharge above the level of the GRS abutment, it is an engineered fill that is considered less variable than a typical surcharge load; hence, the load factor for the road base is equivalent to the factor used for horizontal earth pressure.

$$F_R = \gamma_{EH\,MAX}(F_b + F_{rb}) + \gamma_{LS} F_t \tag{12}$$

The factored resisting force at the base of the GRS abutment (R_R) is calculated according to equation 13.

$$R_R = \Phi_\tau \left(W_{T,R} \mu\right) \tag{13}$$

Where:

Φ_τ = sliding resistance factor (equal to 1.0).
$W_{T,R}$ = total factored resisting weight (calculated in equation 14).
μ = friction factor between the abutment wall and the RSF (taken as the tangent (i.e., tan) of the critical friction angle, ϕ_{crit}).

Because the RSF is encapsulated with a continuous geotextile, sliding at the base of the GRS abutment would occur between the abutment/RSF backfill and the geotextile reinforcement. ϕ_{crit} is equal to the interface friction angle between the reinforced backfill and the reinforcement. The interface friction angle should preferably be determined by conducting an interface direct shear test for the particular combination of geosynthetic and reinforced fill material.(36) If this information is not available during the design stage, it should be assumed that the friction factor is equal to two-thirds times the tangent of the reinforced backfill friction angle (i.e., $\mu = {}^{2}/_{3}\tan(\phi_r)$).

$$W_{T,R} = \gamma_{EV\,MIN} W + \gamma_{DC\,MIN}(q_{DL} b) + \gamma_{DC\,MIN}(W_{face}) + \gamma_{EH\,MIN}(q_{rb} b_{rb,t}) \tag{14}$$

Where:

$\gamma_{EV\,MIN}$ = minimum vertical earth pressure load factor.
W = weight of the GRS abutment backfill (calculated in equation 15).
$\gamma_{DC\,MIN}$ = minimum DL load factor.
q_{DL} = superstructure DL pressure.
b = bearing width of the bridge.
W_{face} = weight of the facing elements (equation 8).
$\gamma_{EH\,MIN}$ = minimum horizontal earth pressure load factor.
q_{rb} = surcharge due to structural backfill (road base) DL.
$b_{rb,t}$ = width of the traffic and road base surcharges over the GRS abutment (see figure 22).

Again, the LLs on the approach pavement and the superstructure are not included as resisting forces because they are transient loads.

$$W = \gamma_r HB \tag{15}$$

Where γ_r is the unit weight of the reinforced backfill.

For LRFD, the ratio of the factored resistance and the factored driving force must be greater than or equal to 1.0 (equation 16). If not, lengthening the reinforcement at the base should be considered. Alternatively, a more complex analysis including the full weight of the GRS abutment up to the cut slope can be performed.

$$\frac{R_R}{F_R} \geq 1.0 \tag{16}$$

4.3.6.1.2 Direct Sliding at the Base of the RSF

Direct sliding should also be checked at the interface between the RSF and the foundation soil. The check is similar to the previously performed check to evaluate sliding at the base of the GRS abutment; however, the weight of the RSF (W_{RSF}) is also included as a resisting force. The nominal lateral force behind the GRS abutment and RSF due to the retained backfill ($F_{b,RSF}$), the road base surcharge ($F_{rb,RSF}$), and the roadway LL surcharge ($F_{t,RSF}$) are determined along the height of the GRS abutment and depth of the RSF (equation 17 through equation 19).

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2 \tag{17}$$

$$F_{rb,RSF} = q_{rb} K_{ab}(H + D_{RSF}) \tag{18}$$

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF}) \tag{19}$$

The total factored driving force at the base of the RSF ($F_{R,RSF}$) is calculated by summing each factored thrust force, as shown in equation 20. The maximum horizontal earth pressure load factor ($\gamma_{EH\,MAX}$) and LL surcharge load factor (γ_{LS}), which are determined using table 6 and table 7, are utilized for the retained backfill, road base, and traffic surcharges.

$$F_{R,RSF} = \gamma_{EH\,MAX}\left(F_{b,RSF} + F_{rb,RSF}\right) + \gamma_{LS} F_{t,RSF} \tag{20}$$

The factored resisting force for direct sliding at the base of the RSF ($R_{R,RSF}$) is calculated according to equation 21.

$$R_{R,RSF} = \Phi_\tau\left(W_{T,R,RSF}\,\mu_{RSF}\right) \tag{21}$$

Where:

Φ_τ = sliding resistance factor (equal to 1.0).
$W_{T,R,RSF}$ = total factored resisting weight including the RSF (calculated in equation 22).
μ_{RSF} = friction factor between the base of the RSF and the foundation soils.

In the absence of interface shear testing, the friction factor at the base of the encapsulated RSF depends on the strength of the foundation soils and the RSF backfill. The tangent of the friction angle of the weakest soil (based on total shear strength, not just friction angle), or two-thirds times the tangent of the reinforced soil fill friction angle (i.e., $\mu_{RSF} = {}^2/_3 \tan(\phi_r)$), should be selected, whichever is lower.

$$W_{T,R,RSF} = W_{T,R} + \gamma_{EV\,MIN} W_{RSF} \tag{22}$$

Where:

$W_{T,R}$ = total factored resisting weight (calculated in equation 14).
$\gamma_{EV\,MIN}$ = minimum vertical earth pressure load factor.
W_{RSF} = weight of the RSF (equation 23).

$$W_{RSF} = \gamma'_{RSF} B_{RSF} D_{RSF} \tag{23}$$

Where:

γ'_{RSF} = effective unit weight of the RSF backfill.
B_{RSF} = base width of the RSF.
D_{RSF} = depth of the RSF.

In LRFD, the ratio of the factored resistance and the factored driving force must be greater than or equal to 1.0 (equation 24). If not, consideration should be given to widening the RSF. Alternatively, a more complex analysis including the full weight of the GRS abutment up to the cut slope can be performed. Note that the passive pressures due to any material in front of the RSF are not included as a conservative measure; however, the designer may elect to calculate this resistance assuming the material will remain in place throughout the life of the GRS-IBS.

$$\frac{R_{RSF,R}}{F_{RSF,R}} \geq 1.0 \tag{24}$$

External Bearing Resistance

To prevent bearing failure, the vertical pressure at the base of the RSF (or abutment, if standalone) must not exceed the allowable bearing resistance of the underlying soil foundation. In an IBS, the vertical pressure is a result of the weight of the GRS abutment, the weight of the RSF, the bridge DL, the road base load from the integrated approach, the LL on the superstructure, and the LL on the approach pavement. The factored vertical pressure at the base of the GRS $(\sigma_{v,base,R})$ is calculated according to a Meyerhof-type distribution, as shown in equation 25.

$$\sigma_{v,base,\,R} = \frac{\sum V_R}{B_{RSF} - 2e_{B,R}} \tag{25}$$

Where:

ΣV_R = total factored vertical load (equation 26).
B_{RSF} = base width of the RSF.
$e_{B,R}$ = factored eccentricity for bearing resistance (equation 27).

$$\sum V_R = \gamma_{EV\,MAX}(W) + \gamma_{EV\,MAX}(W_{RSF}) + \gamma_{DC\,MAX}(W_{face}) + \gamma_{LS}(q_t b_{rb,t}) + \gamma_{EH\,MAX}(q_{rb} b_{rb,t}) + \gamma_{DC\,MAX}(q_{DL} b) + \gamma_{LS}(q_{LL} b) \tag{26}$$

Where:

$\gamma_{EV\,MAX}$ = maximum vertical earth pressure load factor.
W = weight of the GRS abutment backfill (equation 15).
W_{RSF} = weight of the RSF (equation 23).
$\gamma_{DC\,MAX}$ = maximum DL load factor.
W_{face} = weight of the facing elements (equation 8).
γ_{LS} = traffic LL surcharge load factor.
q_t = roadway LL surcharge due to traffic.
$b_{rb,t}$ = width of the traffic and road base surcharges over the GRS abutment.
$\gamma_{EH\,MAX}$ = maximum horizontal earth pressure load factor.
q_{rb} = surcharge due to the structural backfill of the integrated approach (i.e., road base).
q_{DL} = superstructure DL pressure.

b = bearing width of the beam seat.
q_{LL} = bridge LL pressure.

$$e_{B,R} = \frac{\sum M_{D,R} - \sum M_{R,R}}{\sum V_R} \tag{27}$$

Where:

$\Sigma M_{D,R}$ = total factored driving moment (equation 28).
$\Sigma M_{R,R}$ = total factored resisting moment (equation 29).

Equation 28 and equation 29 for the moments are calculated about the bottom center of the width of the RSF; however, moments can be calculated about any point (e.g., the toe of the RSF, etc.) as long as the designer is consistent. Note that if $e_{B,R}$ is negative, $e_{B,R}$ equal to zero should be taken in equation 25.

$$\sum M_{D,R} = \gamma_{EH\,MAX} F_{b,RSF}\left(\frac{H + D_{RSF}}{3}\right) + \gamma_{LS} F_{t,RSF}\left(\frac{H + D_{RSF}}{2}\right) + \gamma_{EH\,MAX} F_{rb,RSF}\left(\frac{H + D_{RSF}}{2}\right) \tag{28}$$

$$\sum M_{R,R} = \left(\gamma_{DC\,MAX} q_{DL} b + \gamma_{LS} q_{LL} b\right)\left[\left(\frac{b}{2} + a_b\right) - \left(\frac{B_{RSF}}{2} - x_{RSF} - b_{block}\right)\right] + \left(\gamma_{LS} q_t b_{rb,t} + \gamma_{EV\,MAX} q_{rb} b_{rb,t}\right)\left(\frac{B_{RSF}}{2} - \frac{b_{rb}}{2}\right) + \gamma_{EV\,MAX} W\left(\frac{B_{RSF}}{2} - \frac{B}{2}\right) + \gamma_{DC\,MAX} W_{face}\left(B + \frac{b_{block}}{2} - \frac{B_{RSF}}{2}\right) \tag{29}$$

Where b_{block} is the width of the facing block.

The factored bearing resistance (q_R) of the foundation can be found using equation 30. Based on AASHTO, the bearing resistance factor (Φ_{bc}) is equal to 0.65.[29]

$$q_R = \Phi_{bc}\left(c'_f N_c + \frac{1}{2} B' \gamma'_f N_\gamma + \gamma'_f D_f N_q\right) \tag{30}$$

Where:

c'_f = cohesion of the foundation soil.
N_c, N_γ, and N_q = dimensionless bearing capacity coefficients (see table 9).
γ'_f = effective unit weight of the foundation soil.
B' = effective foundation width.
D_f = depth of the embedment.

The friction angle in table 9 should be taken as the foundation's effective friction angle (ϕ'_f). If groundwater is present, modifications to equation 30 may be necessary and are provided by AASHTO.[29]

Table 9. Bearing resistance factors.[29]

ϕ_f	N_c	N_q	N_γ	ϕ_f	N_c	N_q	N_γ
0	5.14	1.0	0.0	23	18.1	8.7	8.2
1	5.4	1.1	0.1	24	19.3	9.6	9.4
2	5.6	1.2	0.2	25	20.7	10.7	10.9
3	5.9	1.3	0.2	26	22.3	11.9	12.5
4	6.2	1.4	0.3	27	23.9	13.2	14.5
5	6.5	1.6	0.5	28	25.8	14.7	16.7
6	6.8	1.7	0.6	29	27.9	16.4	19.3
7	7.2	1.9	0.7	30	30.1	18.4	22.4
8	7.5	2.1	0.9	31	32.7	20.6	26.0
9	7.9	2.3	1.0	32	35.5	23.2	30.2
10	8.4	2.5	1.2	33	38.6	26.1	35.2
11	8.8	2.7	1.4	34	42.2	29.4	41.1
12	9.3	3.0	1.7	35	46.1	33.3	48.0
13	9.8	3.3	2.0	36	50.6	37.8	56.3
14	10.4	3.6	2.3	37	55.6	42.9	66.2
15	11.0	3.9	2.7	38	61.4	48.9	78.0
16	11.6	4.3	3.1	39	67.9	56.0	92.3
17	12.3	4.8	3.5	40	75.3	64.2	109.4
18	13.1	5.3	4.1	41	83.9	73.9	130.2
19	13.9	5.8	4.7	42	93.7	85.4	155.6
20	14.8	6.4	5.4	43	105.1	99.0	186.5
21	15.8	7.1	6.2	44	118.4	115.3	224.6
22	16.9	7.8	7.1	45	133.9	134.9	271.8

The ratio of the factored bearing resistance and the factored applied pressure must be greater than or equal to 1.0 (equation 31). If not, options include increasing the width of the GRS abutment and RSF by increasing the length of the reinforcement layers, replacing the foundation soil with a more competent soil, or adding embedment depth.

$$\frac{q_R}{\sigma_{v,base,\,R}} \geq 1.0 \qquad (31)$$

4.3.6.3 Global Stability

Global stability is evaluated according to the classical slope stability theory using either rotational or wedge analysis. To facilitate the global stability check, it is prudent to collect accurate soil property information. Standard slope stability computer programs can then be used to assess the global and compound stability of a GRS structure. The factor of safety for global stability should equal at least 1.5, which equates to a resistance factor of 0.65.

4.3.7 Step 7—Conduct Internal Stability Analysis

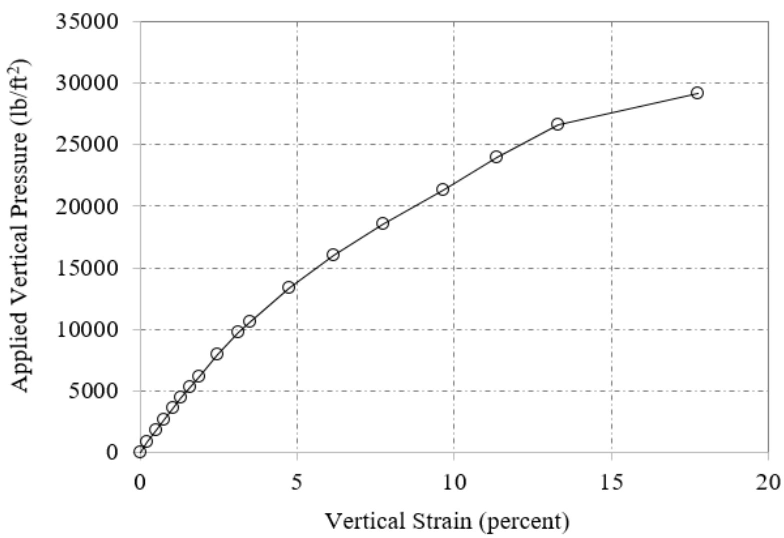
Internal stability for GRS includes ensuring adequate internal bearing resistance, tolerable deformations, and required reinforcement strength.

4.3.7.1 Internal Bearing Resistance

The nominal bearing resistance of a GRS abutment is determined either empirically through a GRS performance test or analytically through a semi-empirical equation.

4.3.7.1.1 Empirical Method

Empirically, the results of an applicable performance test using the same geosynthetic reinforcement and compacted granular backfill as planned for the site should be used. The bearing resistance in this case is defined as the stress at which the GRS composite fails at the strength limit (i.e., cannot sustain any more loading). An example of a performance test result and the corresponding bearing resistance is shown in figure 28. For this particular performance test, the nominal bearing resistance of the GRS abutment using the empirical method ($q_{n,emp}$) was equal to almost 30,000 lb/ft^2 at a vertical strain of about 17 percent.



Source: FHWA.

Figure 28. Graph. Stress–strain curve for a GRS composite.

Note that figure 28 represents the load–settlement behavior of a GRS composite with 4,800 lb/ft woven PP geosynthetic reinforcement spaced nominally at 8 inches and a well-compacted AASHTO A-1-a fill material with a maximum aggregate size of 1 inch, a friction angle of 54 degrees, and cohesion of 115 kPa. The use of this stress–strain curve is limited to these particular components; however, other curves are available for a variety of GRS composites.[4] If the materials used are outside of those previously tested, then a performance test can be performed to obtain the applicable stress–strain curve similar to figure 28. Alternate methods that do not require a performance test are discussed next in section 4.3.7.1.2.

The factored applied stress on top of the the GRS soil mass ($V_{applied,f}$) is equal to the sum of the vertical pressures on the bridge bearing area multiplied by their respective load factors (equation 32). The vertical pressures of interest include q_{DL} and q_{LL}. The surcharge due to q_{rb} and q_t due to the approach pavement are located behind the bearing area and are therefore not included in the bearing resistance related to the bridge superstructure.

$$V_{applied,f} = \gamma_{DC\,MAX} q_{DL} + \gamma_{LL} q_{LL} \tag{32}$$

The factored applied pressure must be less than or equal to the factored vertical resistance (equation 33). The resistance nominal factor (Φ_{cap}) is equal to 0.45.

$$\frac{\Phi_{cap}(q_{n,emp})}{V_{applied,f}} \geq 1.0 \tag{33}$$

4.3.7.1.2 Analytical Method

As an alternative to conducting a performance test, the nominal bearing resistance of a GRS abutment can also be evaluated using a semi-empirical formula (equation 34).[37] Note that the analytical method assumes that the backfill and reinforcement satisfy the criteria outlined in chapter 3.

$$q_{n,an} = \left[0.7^{\left(\frac{S_v}{6d_{max}}\right)} \frac{T_f}{S_v}\right] K_{pr} \tag{34}$$

Where:

$q_{n,an}$ = nominal bearing resistance of the GRS abutment using the analytical method.
S_v = reinforcement spacing.
d_{max} = maximum grain size.
T_f = ultimate reinforcement strength.
K_{pr} = coefficient of passive earth pressure for the reinforced fill (calculated in equation 35).

$$K_{pr} = \frac{1 + \sin\phi_r}{1 - \phi_r} = \tan^2\left(45 \text{ degrees} + \frac{\phi_r}{2}\right) \tag{35}$$

Where ϕ_r is the friction angle of the reinforced backfill. The friction angle should be determined from a large-scale direct shear device per ASTM D3080.[38] The factored applied pressure must be less than the factored bearing resistance (equation 36). Φ_{cap} is equal to 0.45.[4]

$$\frac{\Phi_{cap}(q_{n,an})}{V_{applied,f}} \geq 1.0 \tag{36}$$

4.3.7.2 Deformations

The general approach for determining vertical deformation of the GRS abutment involves empirically finding the strain from an applicable performance test curve. Alternatively, vertical strain can be targeted to 1 percent of the abutment height through a prescribed bearing resistance. In either case, the lateral strain is then determined analytically assuming the theory of zero volume change.[1,39]

Typically, if the materials used are within the specifications given in chapter 3, then designers can expect vertical strain to be about 1 percent of the abutment height and lateral strain to be about 2 percent of the width of the load along the top of the wall (including the setback). Engineers may select more or less stringent deformation criteria depending on the project. Methods to evaluate deformations prior to construction are presented in the following subsections.

4.3.7.2.1 Vertical

Empirically, the vertical strain of the GRS abutment is found from the intersection of the applied vertical stress due to the DL and the performance test design envelope for vertical strain (see figure 28). The vertical deformation, or settlement, of the GRS abutment is this vertical strain multiplied by the height of the GRS abutment (H_{abut}). Because the GRS abutment is built with a granular fill, the majority of settlement within the GRS abutment will occur immediately after the placement of DL and before the bridge is opened to traffic; therefore, transient loads due to LLs are not included in the empirical evaluation.

In the event an applicable performance test is not available, an alternative approach is provided whereby the vertical strain is limited to 1 percent by imposing a prescribed service bearing pressure for DL ($q_{DL,allow@\varepsilon=1\%}$) (equation 37).[22]

$$q_{DL,allow@\varepsilon=1\%} = 0.2\left[0.7^{S_v/6d_{max}}\left(\frac{T_f}{S_v}\right)\right]K_{pr} \tag{37}$$

If the estimated applied DL from the bridge exceeds $q_{DL,allow@\varepsilon=1\%}$, then the beam seat area can be widened or tolerable deformation can be reevaluated. For example, if only 0.5 percent vertical strain is allowed, then $q_{DL,allow@\varepsilon=1\%}$ would be 10 percent of the estimated capacity of the GRS abutment.

The settlement of the underlying foundation soils is determined separately using classic soil mechanics theory for immediate (elastic), consolidation, and secondary settlements. Design considerations such as the amount of previous load (if a bridge replacement project) and the reduction in pressure due to the excavation for the RSF should be evaluated for the particular project. Nevertheless, settlement of the foundation soil should be assessed as with any other shallow footings according to FHWA's *Soils and Foundations Reference Manual*.[33] Determining the criterion for tolerable foundation settlement is a decision for the engineer based on the superstructure, clearance requirements, risk, etc.

A stress history analysis should be conducted to ascertain settlement and stability prediction. Answers to the following questions will provide insight on the stress history for an efficient design:

- Is the site bridge a replacement project built in the same location?
- What was the performance of the existing bridge?
- Were there any chronic maintenance issues associated with the existing structure?
- What was the combined weight of the abutment and superstructure within the footprint of the new bridge foundation? How does that stress compare with the stress of the new structure?
- Does the site involve an excavation equivalent to the weight of the new GRS-IBS?

4.3.7.2.2 Lateral

In response to a vertical load, the composite behavior of a properly constructed GRS abutment is such that both the reinforcement and soil will tend to strain laterally together. This observation can be used to predict both the maximum lateral reinforcement strain and the maximum face deformation under a given service load. The recommended method conservatively assumes zero volume change in the GRS abutment.[1,39] The maximum lateral displacement (D_L) of the abutment face wall can be estimated using equation 38. The lateral strain (ε_L) is then found using equation 39 and is typically limited to 2 percent depending on the criterion established in step 1 (see section 4.3.1).

$$D_L = \frac{2b_{q,vol}D_v}{H} \tag{38}$$

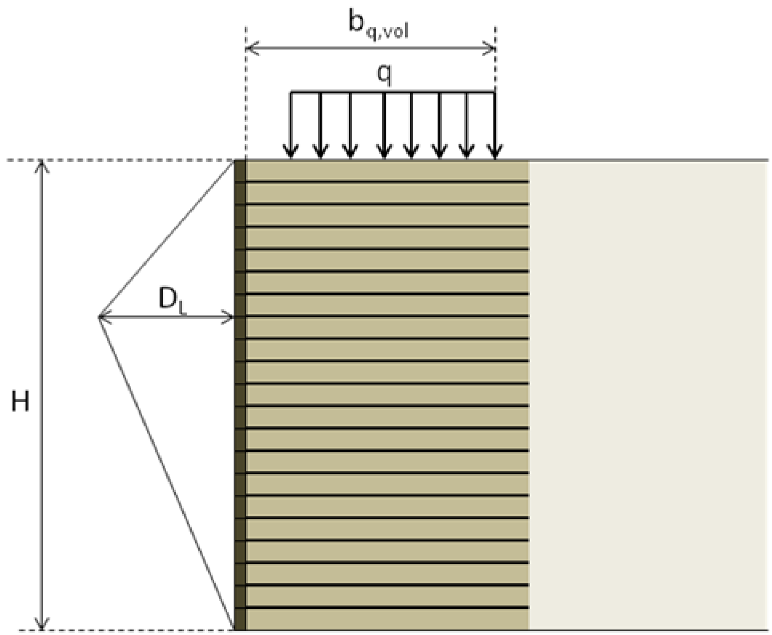
Where:

$b_{q,vol}$ = width of the load along the top of the wall including the setback distance.
D_v = vertical settlement in the GRS abutment mass (section 4.3.7.2.1).
H = height of the GRS abutment including the clear space distance.

$$\varepsilon_L = \frac{D_L}{b_{q,vol}} = \frac{2D_v}{H} = 2\varepsilon_v \tag{39}$$

Where ε_V is the vertical strain.

Note that equation 38 and equation 39 assume a triangular lateral deformation and a uniform vertical deformation (figure 29); the location of the maximum lateral deformation depends on the loading and fill conditions, but the volume gained will still equal the volume lost under service load conditions. The maximum deformation of a GRS abutment often occurs in the top third of the abutment/wall. (See references 22, 37, 39, and 40.)



Source: FHWA.

Figure 29. Illustration. Lateral deformation of a GRS structure.

4.3.7.3 Required Reinforcement Strength

The properties of the geosynthetic reinforcement (i.e., ultimate strength and stiffness) must meet both strength and serviceability requirements. This will prevent failure of the GRS composite throughout the life of the bridge and will also limit reinforcement strains under service conditions. Two criteria must be satisfied for the required reinforcement strength: (1) it must be less than the allowable reinforcement strength (T_{allow}), and (2) it must be less than the strength at 2 percent reinforcement strain ($T_{@\varepsilon=2\%}$) in the direction perpendicular to the abutment wall face.

4.3.7.3.1 Strength Limit

The factored required reinforcement strength ($T_{req,f}$) in the direction perpendicular to the abutment wall face can be determined analytically by equation 40. $T_{req,f}$ should be calculated at each layer of reinforcement to ensure adequate strength throughout the GRS abutment; however, it is recommended that only one type of reinforcement be specified throughout the abutment to simplify the design, avoid construction placement issues, and limit costs.

$$T_{req,f} = \left[\frac{\sigma_{h,f}}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}} \right] S_v \tag{40}$$

Where $\sigma_{h,f}$ is the factored total lateral pressure within the GRS abutment at a given depth and location (equation 41).

$$\sigma_{h,f} = \sigma_{h,W,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f} \tag{41}$$

Where:

$\sigma_{h,W,f}$ = factored lateral pressure due to weight of the GRS (equation 42).
$\sigma_{h,bridge,f}$ = factored lateral pressure due to the equivalent bridge load (equation 43).
$\sigma_{h,rb,f}$ = factored lateral pressure due to the road base surcharge within the GRS (see equation 44).
$\sigma_{h,t,f}$ = factored lateral pressure due to traffic surcharge within the GRS (equation 45).

$$\sigma_{h,W,f} = \gamma_{EH\,MAX}\left(\gamma_r z K_{ar}\right) \tag{42}$$

Where:

$\gamma_{EH\,MAX}$ = maximum horizontal earth pressure load factor.
γ_r = unit weight of reinforced backfill.
z = depth from the top of the wall.
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

$$\sigma_{h,bridge,f} = \frac{\left(\gamma_{DC\,MAX} q_{DL} + \gamma_{LL} q_{LL}\right) - \left(\gamma_{EH\,MAX} q_{rb} + \gamma_{LS} q_t\right)}{\pi}\left[\alpha_b + \sin(\alpha_b)\cos(\alpha_b) + 2\beta_b\right]K_{ar} \tag{43}$$

Where:

$\gamma_{DC\,MAX}$ = maximum DL load factor.
q_{DL} = superstructure DL pressure.
γ_{LL} = bridge LL surcharge load factor.
q_{LL} = bridge LL pressure.
q_{rb} = surcharge due to the structural backfill of the integrated approach (i.e., road base).
γ_{LS} = LL surcharge load factor.
q_t = roadway LL surcharge.
α_b = angle between wall face and projection of the midline of the bridge surcharge to the wall face found using equation 46 (see figure 23).
β_b = angle between the projections of the inner and outer edge lines of the bridge surcharge to the wall face found using equation 47 (see figure 23).

$$\sigma_{h,rb,f} = \gamma_{EH\,MAX} q_{rb} K_{ar} \tag{44}$$

$$\sigma_{h,t,f} = \gamma_{LS} q_t K_{ar} \tag{45}$$

$$\alpha_b = \tan^{-1}\left(\frac{b}{2z}\right) - \beta_b \tag{46}$$

$$\beta_b = \tan^{-1}\left(\frac{-b}{2z}\right) \tag{47}$$

For abutments, a typical default wide width tensile strength (T_f) of 4,800 lb/ft is often selected; however, the required strength may be more or less depending on the project requirements. To account for long-term strength losses of the geosynthetic, a global reduction factor (RF_{global}) of 2.25 is recommended. RF_{global} accounts for creep, durability, and installation damage. In addition, a resistance factor for reinforcement strength (Φ_{reinf}) of 0.9 should be applied to T_f to determine the factored reinforcement strength ($T_{f,f}$), as shown in equation 48.

$$T_{f,f} = \Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right) = 0.4T_f \tag{48}$$

$T_{f,f}$ must be less than or equal to the factored required reinforcement strength ($T_{req,f}$), as shown in equation 49, to satisfy the strength limit. If not, a stronger geosynthetic must be chosen or the reinforcement spacing must be decreased.

$$\frac{T_{f,f}}{T_{req,f}} \geq 1.0 \tag{49}$$

Conservatively, the strength requirements of the RSF and integrated approach are considered equal to that of the GRS abutment for simplicity in purchasing and construction placement, although similar computations can be performed to determine separate component-level needs if the designer chooses.

4.3.7.3.2 Service Limit

Since geosynthetic reinforcements of similar strength can have rather different load–deformation relationships depending on the manufacturing process and the polymer used, it is important that the nominal required reinforcement strength (T_{req}) is less than $T_{@\varepsilon = 2\%}$ in the strong direction for the particular reinforcement specified. $T_{@\varepsilon = 2\%}$ is often given by the geosynthetic manufacturer.

T_{req} in the direction perpendicular to the wall face is determined analytically by equation 50.[37] It should be calculated at each layer of reinforcement to ensure adequate strength throughout the GRS abutment.

$$T_{req} = \left[\frac{\sigma_h}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v \tag{50}$$

Where σ_h is the lateral pressure within the GRS abutment at a given depth and location and is calculated in equation 51.

$$\sigma_h = \sigma_{h,W} + \sigma_{h,bridge,eq} + \sigma_{h,rb} + \sigma_{h,t} \tag{51}$$

Where:

$\sigma_{h,W}$ = lateral pressue due to the weight of the reinforced backfill in the GRS abutment (see equation 2).
$\sigma_{h,bridge,eq}$ = lateral pressure due to the equivalent bridge load (equation 52).

$\sigma_{h,rb}$ = lateral pressure due to the road base surcharge within GRS (equation 53).
$\sigma_{h,t}$ = lateral pressure due to the traffic surcharge within GRS (equation 54).

$$\sigma_{h,bridge,eq} = \frac{(q_{DL} + q_{LL}) - (q_{rb} + q_t)}{\pi}\left[\alpha_b + \sin(\alpha_b)\cos(\alpha_b + 2\beta_b)\right]K_{ar} \tag{52}$$

Where:

q_{DL} = bridge DL pressure.
q_{LL} = bridge LL surcharge.
q_{rb} = surcharge due to the structural backfill of the integrated approach (i.e., road base).
q_t = roadway LL surcharge.

α_b = angle between wall face and projection of the midline of the bridge surcharge to the wall face found using equation 46 (see figure 23).
β_b = angle between the wall face and projection of the midline of the surcharge to the wall face found using equation 47 (see figure 23).
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

$$\sigma_{h,rb} = q_{rb}K_{ar} \tag{53}$$

$$\sigma_{h,t} = q_t K_{ar} \tag{54}$$

To simplify calculations, the approach LL and road base DL are extended across the abutment. The vertical components of these loads are then subtracted from the bridge DL pressure and bridge LL pressure, giving an equivalent bridge load. The lateral stress due to the equivalent bridge load is then calculated according to Boussinesq's theory. The location of interest to determine the maximum lateral pressure is directly underneath the centerline of the bridge bearing width.

If T_{req} is greater than $T_{@\varepsilon = 2\%}$ for the strong direction, a stiffer geosynthetic must be chosen or the reinforcement spacing must be decreased. Because bridges are often in a plane strain condition, the bridge loads will not be shed to the wing walls; therefore, $T_{@\varepsilon = 2\%}$ only needs to be specified in the direction perpendicular to the abutment wall face.

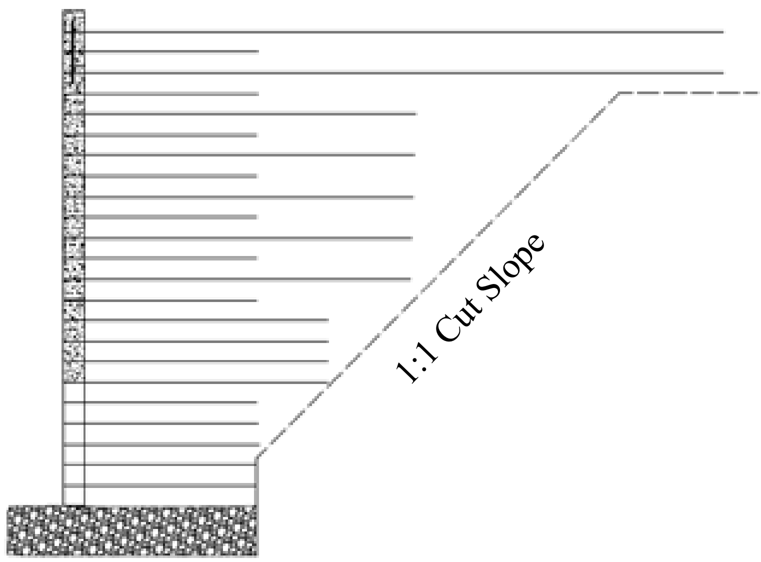
While $T_{@\varepsilon = 2\%}$ can theoretically vary along the height of the GRS abutment, it is again recommended that only one type of reinforcement be used throughout the entire abutment. This simplifies the construction process and avoids placement errors for the reinforcement. As a reminder, $T_{@\varepsilon = 2\%}$ is not equivalent to the load on the reinforcement under service conditions; the actual load will vary depending on the final strength and stiffness of the reinforcement specified. This check will ensure that the actual load on the reinforcement is less than what is calculated to limit lateral strain.

4.3.7.3.3 Depth of Bearing Bed Reinforcement

$T_{req,f}$ is found at each primary spacing layer for both the strength and service limit states. If $T_{req,f}$ is greater than $T_{f,f}$ or if T_{req} is greater than $T_{@\varepsilon = 2\%}$, then the reinforcement spacing must be reduced (typically in half) to the necessary depth. This depth is termed the “bearing bed reinforcement.” The minimum required depth is three courses of block, although traditionally five layers have been specified as a default for the majority of GRS abutments. To verify that reduced spacing for the bearing bed reinforcement is adequate, the required reinforcement strength for S_v at the top should be calculated again.

4.3.8 Step 8—Implement Design Details

Figure 30 shows a typical cross section of a GRS wing wall. Additional design details are discussed in this section.



Source: FHWA.

Figure 30. Illustration. Typical cross section of a GRS wing wall.

In the case of an abutment, the design layout for ease of construction, drainage, and other considerations that might affect the performance, serviceability, or efficiency of design should be finalized. The following GRS design implications and related details should be considered:

- A core of native soil in the center of the abutment face and two adjacent wing walls should be considered to minimize excavation (figure 31). The wing walls can be truncated like the abutment. They should be extended sufficiently into the cut slope to prevent erosion caused by undermining or piping. This should be a minimum of two facing block lengths.

Source: FHWA.

Figure 31. Photo. GRS abutment and wing walls built around a core of native soil.

- Engineers shall determine whether to build wing walls with either a full face or a stepped face that leads into the cut slope. The decision depends on several factors related to the height of the abutment, grade of fill slope (which is usually at a ratio of 2:1), time, and materials. For abutments less than 12 ft in height, a full face is probably most efficient, as it is the easiest to construct. However, for abutments greater than 12 ft, it might be more efficient to design a stepped-face wall that leads into the cut slope. Stepped walls use less material but require additional labor in building the second foundation to support the extended stepped wall. In either case, all facing blocks should be supported on well-compacted structural fill.

- GRS-IBS has been used for bridges with skew, superelevation, and grade without problems or serviceability issues. The maximum values for each on an in-service bridge are 30 degrees, 8 degrees, and 6 percent, respectively.

- For a skewed bridge, it is important to maintain the minimum bearing area of 2.5 ft along the length of the abutment face wall. Considerations for the wing wall corner details of skewed abutments should also be laid out depending on the type of facing (see chapter 7). The most efficient design is to have wing walls at 90 degrees from the abutment face if this can be accommodated at the site.

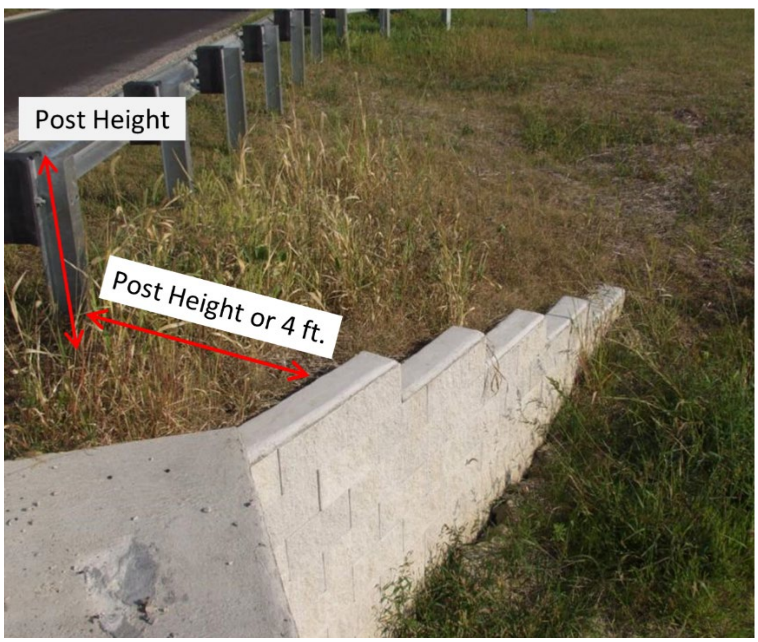
- For a bridge with superelevation, it is important to ensure that the minimum number of bearing bed reinforcement layers beneath the beam seat (calculated in section 4.3.7.3.3) are installed across the length of the abutment face.

- At the time this manual was written, there were no special considerations for opposing GRS abutments that support a bridge on a grade.

- The design needs to include provision for surface and subsurface drainage along the fill slope adjacent to the wing walls. Channel drains along the wing walls should be included to facilitate runoff. The drain path should not be located directly against the wall face. The drain path should be armored with a strip of geotextile beneath a layer of channel rock. Compacted native soil should be graded against the wing walls with a slope leading to the drainage path.

- As the height of the integrated approach increases (i.e., the span length of the bridge increases), the quantities of geotextile needed for the project will increase, particularly in the case where the cut slope is not very steep and the abutment is high. Weaker geotextiles could be specified for the integrated approach to save in project costs; however, it is recommended that one strength of reinforcement be specified for an IBS project to simplify the construction process and avoid placement errors.

- The face of the abutment (which includes the parapet) must be wide enough to accommodate the installation of guardrails (figure 32). The additional width should be enough to allow the guardrail to lay down in the event of impact (this lay-down length is approximately 4 ft). Nondisplacement steel rail posts are optimal because they can be driven directly through the GRS composite without damage. Wooden posts are nearly impossible to drive into GRS; however, it is possible to predrill through GRS with an auger to assist in the installation, if necessary.



Source: FHWA.

Figure 32. Photo. Guardrail lay-down distance.

- The GRS integrated approach fill should be contained by wrapping the geotextile layers adjacent to the beam ends to prevent lateral spreading (see chapter 7). The reinforcement layers should be extended at the approach back onto the road. Any abrupt transition of soil type from the roadway to the bridge should be avoided. Engineers should plan ahead to avoid trenching and account for the possible installation of utilities. Additionally, they should locate and plan to accommodate existing and potential future utilities.

4.3.9 Step 9—Finalize Material Quantities and Layout

To develop the reinforcement schedule, a reinforcement length should be chosen that makes use of the entire roll of the reinforcement material. Reinforcement material is usually 12–18 ft wide. For example, the width of a PP geosynthetic roll is 12 ft, and the base of a GRS wall is a minimum of 6 ft, including the width of the wall face. The roll can be cut in half by a chainsaw, and a 6-ft-wide roll can be used to build the base of the wall. The remaining 6-ft-wide rolls can be used for secondary or intermediate layers of reinforcement in the walls.

The layout should be drawn to scale to avoid errors in the calculation of quantities, and 10 percent should be added to the estimate of all materials. When using CMU, the exact dimensions of $7^5/_8$ by $7^5/_8$ by $15^5/_8$ inches should be used, and both corner and face blocks should be bought.

CHAPTER 5. DESIGN CONSIDERATIONS FOR EXTREME EVENTS

5.1 INTRODUCTION

This chapter describes how extreme events, such as seismicity, may alter the design of GRS-IBS, as presented in chapter 4. Scour is another event that must be evaluated if the bridge is crossing a waterway; the most recent FHWA hydraulics research and proposed design recommendations for bridges supported by shallow foundations (such as GRS) are presented in appendix D.

5.2 SEISMIC DESIGN

General AASHTO guidelines for seismic design of ordinary bridges allows for damage to occur when the structure is subjected to a 1,000-year return event, but the level of acceleration should not permit structure collapse and loss of life.(29) Bridges therefore may suffer slight damage that requires repair or partial replacement; however, GRS walls and abutments have exhibited excellent performance under moderate to high shaking. (See references 41–45.) The behavior of GRS bridge abutments has been characterized through large-scale shake table experiments, which evaluated their response to horizontal shaking of various earthquake frequencies.(44,45) Monitoring of maximum stresses within the geosynthetic reinforcement layers, vertical and lateral strains, and internal acceleration transfer during shaking revealed satisfactory performance of the GRS abutments tested, even when subjected to horizontal accelerations as high as 1g. The experimental results have indicated that the measured GRS abutment deformation caused by various applied ground accelerations were either negligible or very small. Overall, the GRS abutment remained stable with some minor cracking observed in the modular blocks. Test results have also been validated by analytical methods to develop generalized seismic design and construction guidance for GRS abutments in seismically active zones. (45)

Based on experiments and parametric analysis conducted by the aforementioned research projects, it can generally be concluded that GRS with close reinforcement spacing and modular block facing, even when designed using standard static loading methods, can also withstand earthquakes shaking with moderate horizontal ground acceleration (up to 0.4g) with almost no damage or appreciable deformation. The results also indicated that properly designed bearing pads on the bridge sill with a natural frequency below the ground dominant frequency are effective in isolating the superstructure inertia force from the GRS abutment. By isolating this motion, it will greatly reduce the potential for bridge sill sliding on the GRS. Note that the superstructure is integrated within an IBS whereby it is embedded within the integrated approach and side walls, which help limit sliding.

5.2.1 General Seismic Considerations

External stability for seismic design will need to be checked for GRS-IBS just like with any other gravity structure. Since GRS-IBSs are mostly single-span structures, the seismic design required is the same as for conventional shallow foundations. Design considerations for external stability and seismicity include increasing the base width of the wall and increasing the length of the reinforcement at the top of the wall. Additional bearing resistance and overall external stability is generally improved by increasing the base width of the wall. Stability is also created by increasing the length of the reinforcement at the top of the wall or abutment. This integrated approach has also been shown to be beneficial because it keys the structure into the existing terrain, preventing the development of a failure plane along the cut slope, which can lead to progressive failure. No seismic design requirements are necessary for the internal stability of GRS-IBS.

To determine the seismic performance zone for the bridge site, a seismic hazard analysis is required to determine the acceleration coefficient for a 1,000 year- return period event. Ground motions caused by earthquakes are influenced not only by the distance from active faults but also by the geology and subsurface materials found at the site. Subsurface profiles, with soil layers overlaying bedrock that significantly differ in stiffness and density, will have amplified ground motions and a resonant period governed by the soil layer thickness and weighted average shear wave velocity of the subsurface materials within 100 ft of the footing. The values for the ground acceleration coefficient and spectral acceleration coefficients for probabilistic design with a return period of 1,000 years can be determined directly from U.S. Geological Survey design maps.(46) The earthquake load factor used during the LRFD design of the bridge structure is listed in table 6 in chapter 4.(29)

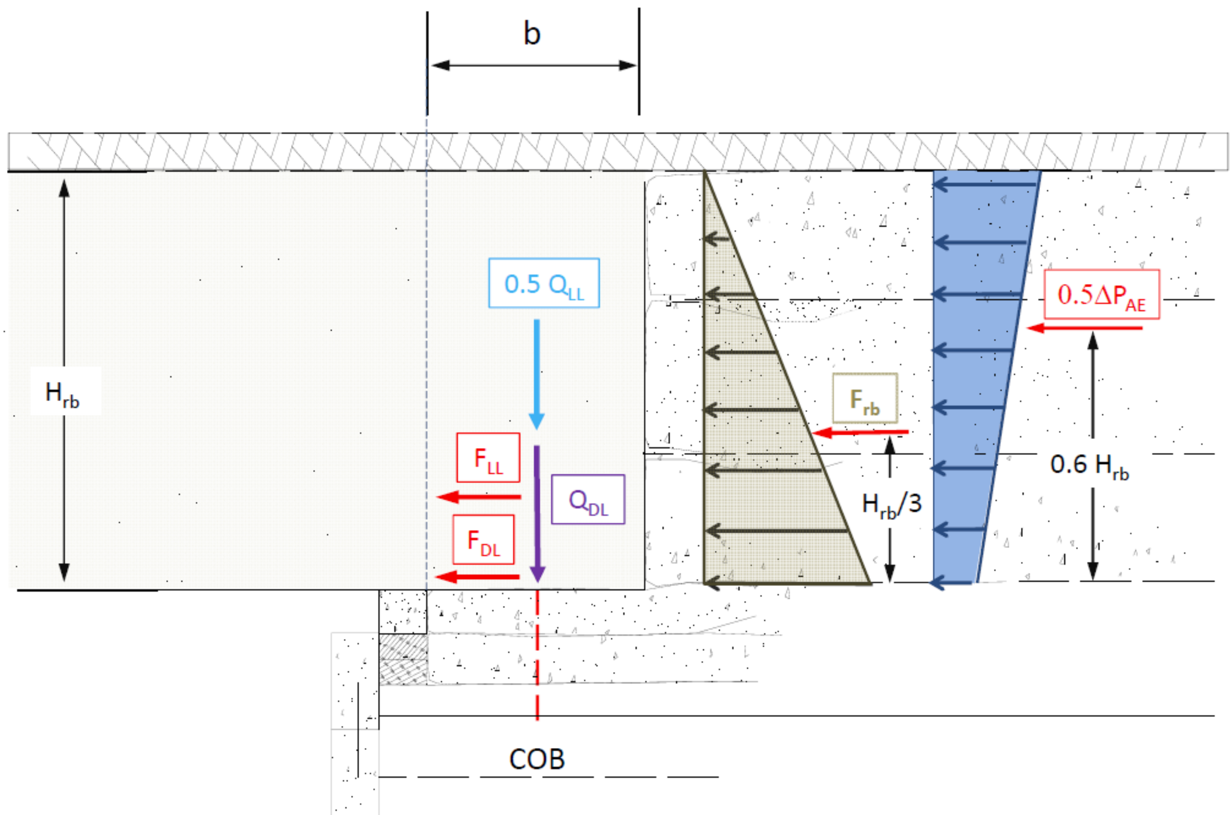
Regardless of the seismic zone, the superstructure for a single-span bridge will only require a design for the minimum beam seat length and connection force equivalent to the product of the acceleration coefficient and the bridge tributary weight.(29) Half the DL (0.5 DL) should be applied for determining the bridge tributary weight because it is embedded within the integrated approach on all sides. Therefore to best comply with *AASHTO LRFD Bridge Design Specifications* for bridges in areas where strong earthquake motions are expected, a conventional concrete sill may be necessary to connect the superstructure to the substructure.(29) If no concrete footing is installed, the passive earth pressure will be considered for resisting the dynamic forces.

5.2.2 Seismic Design Process

AASHTO does not provide specific guidance for the design of GRS abutments; therefore, the following recommended guidance should be considered for the GRS abutment design in an Extreme Event I limit state. When it is determined that the site may be subject to ground motion from relatively large earthquakes, the GRS abutments will be designed for axial and lateral loads, including the seismic loads imposed by the seismic ground acceleration coefficient modified by the site factor.

The center of bearing on each abutment should be placed so the overlap of the girder and seat satisfies the minimum seat length requirements for seismic loading if movement is permitted and to compensate for the eccentricity created by pseudostatic seismic loads from the bridge deck and approach backfill combined with the thickness of the bearing sill. The friction contact area should be designed to transfer bridge seismic loading from the bearing sill to the GRS abutment without failure. The longitudinal seismic forces will be resisted by the passive resistance provided by the integrated approach against the back of the bearing sill or precast slabs at the far end of the bridge. The transverse seismic loading is resisted through both the resistance provided by the back of the superstructure and, if needed, through shear keys installed beneath the bridge sill. Direct sliding will be calculated for using a basal GRS fill friction angle depending on the selected aggregate used in the GRS abutment.

The design checks at the Extreme Event I limit state loads using the appropriate acceleration coefficient for the site should be performed for the static and seismic external stability at the base of the bridge footing and at the base of the GRS abutment. The lateral seismic displacement is checked by considering the GRS abutment as a rigid block and computing the shear deformation of the GRS composite. LLs(due to the traffic on the superstructure as well as from the traffic on the roadway should not be considered in the seismic design calculations. Figure 33 and figure 34 are free-body diagrams of the static and dynamic forces and their relative locations acting on the bridge sill and on the GRS composite.

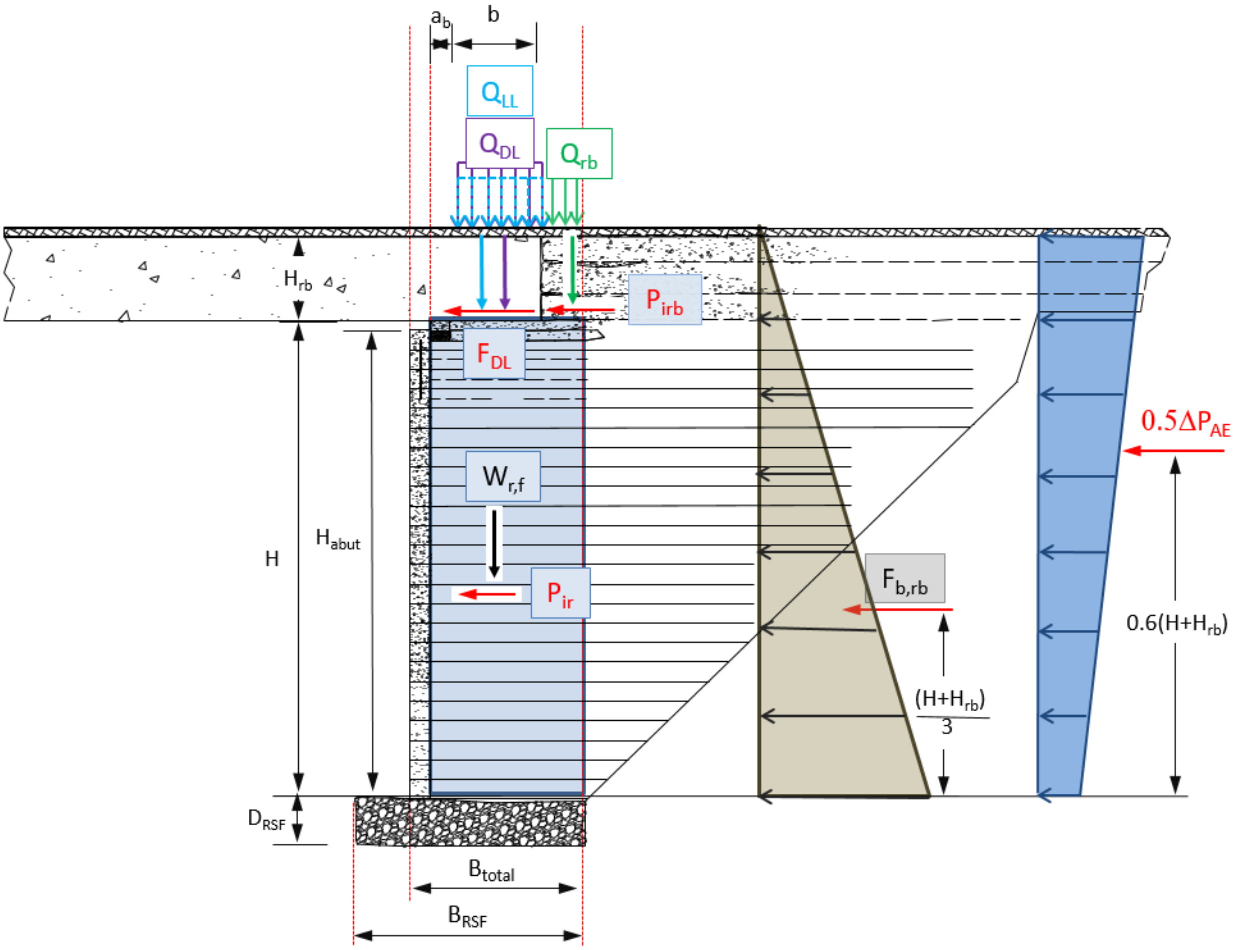


Source: FHWA.

Figure 33. Illustration. Static and dynamic forces acting on the bridge footing.

Where:

b = bearing width for the bridge/beam seat.
H_{rb} = height of the road base.
F_{LL} = horizontal inertia of the LL.
F_{DL} = horizontal inertia of the DL.
Q_{LL} = LL due to the bridge superstructure on each abutment.
Q_{DL} = DL due to the bridge superstructure on each abutment.
F_{rb} = nominal lateral force behind the GRS abutment due to the road base surcharge.
ΔP_{AE} = dynamic (pseudostatic) force.
COB = center of bearing.



Source: FHWA.

Figure 34. Illustration. Static and dynamic forces acting on the GRS abutment.

Where:

Q_{LL} = LL due to the bridge superstructure on each abutment.
Q_{DL} = DL due to the bridge superstructure on each abutment.
Q_{rb} = DL due to the road base on each abutment.
P_{ir} = inertia force of the reinforced fill beneath the abutment.
$P_{ir,b}$ = inertia force of the integrated approach road base behind the superstructure.
$f_{b,rb}$ = inertia force of the retained backfill and road base.

5.2.3 Liquefaction and Lateral Spread Potential in Seismically Active Areas

Liquefaction and lateral spread potential often governs foundation design in seismically active areas with saturated loose non-cohesive sediments as the foundation soil. During an earthquake, seismic shaking may cause an increase in pore water pressure within loose saturated silty sands and non-plastic silty soils. Depending on soil composition, liquefaction may occur even if earthquakes do not produce exceptionally strong ground shaking.

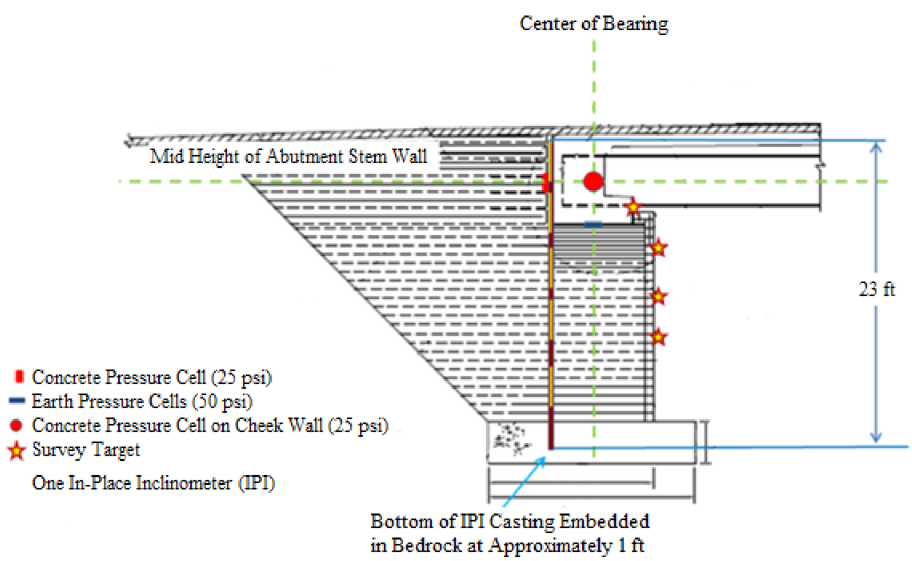
Liquefaction may have a detrimental effect on the GRS-IBS structure, including foundation settlement and large differential displacement. Liquefaction potential should therefore be assessed in the event of seismic shaking caused by the determined site design earthquake magnitude and peak ground acceleration. Similar to shallow concrete foundations, the risks associated with liquefaction and seismic settlement, lateral spreading, and lateral flow must be considered when designing GRS-IBS in seismic areas.

In general, GRS-IBSs have similar characteristics to spread footings and may not be effective where soil liquefaction may occur at or below the footing level unless the liquefiable soil is confined, not very thick, and well below the footing level. Occasionally, GRS footings may be cost effective if inexpensive soil improvement techniques, such as overexcavation of unsuitable soils, deep dynamic compaction, and stone columns, are feasible. Because the Service I limit state often controls the feasibility of GRS abutments, they may still be appropriate and cost effective if the bridge that is being supported can be designed to tolerate the settlement.

CHAPTER 6. IN-SERVICE PERFORMANCE MONITORING

6.1 INTRODUCTION

In the early stages of deployment, several GRS-IBS projects were instrumented and monitored to validate performance expectations and gain further insights into the behavior of the IBS with different superstructure details and geometries. (See references 3 and 47–50.) Figure 35 provides an example of an instrumentation layout to monitor deformations and substructure superstructure interaction. A distinctive feature of the GRS-IBS is that it works with settlement instead of resisting it to create a compatible connection between the bridge, approach, and the roadway, providing a long-term solution to mitigate the bump at the end of the bridge. Reducing the bump at the end of the bridge improves its overall performance and serviceability. This bump not only creates a chronic maintenance issue but also induces an amplification of LL on the superstructure, creating fatigue on bridge elements. In addition, the GRS-IBS works with lateral movement due to thermal contraction and expansion of the superstructure. By wrapping the integrated approach behind the backwall of the superstructure, bridge contraction does not result in infilling with aggregate and thus avoids high point loads on the subsequent expansion. In addition, the GRS abutment tends to move with the superstructure leading to an IBS. This chapter presents methods of evaluating the performance of GRS-IBS, including deformations, thermal movements, and scour monitoring.



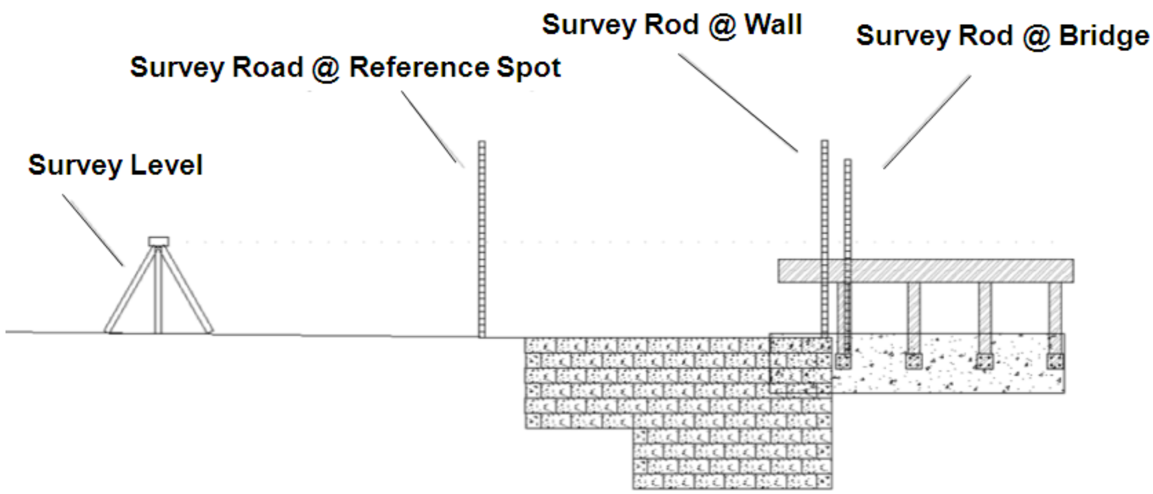
Source: FHWA.

Figure 35. Illustration. Example instrumentation layout.

6.2 DEFORMATIONS

Deformations primarily include vertical and lateral movement of the GRS abutment, foundation soils, wing walls, and superstructure as well as the differential movement causing the bump at the end of the bridge. The vertical and lateral deformations of the GRS abutment due to bridge loads are recorded using either a standard survey level and rod system or an electronic distance measurement (EDM) survey referenced off a permanent survey pole and benchmarks. The precision of all survey measurements (both the survey level method and the EDM system) is typically ±0.005 ft. By using either of these settlement measurement techniques, settlement can be recorded for both the abutment face wall and the superstructure. The difference between the settlement measured on the abutment face wall and the superstructure is the vertical deformation within the GRS abutment alone due to the bridge load. To forecast long-term settlement of either the GRS compression or the total settlement including the foundations soils, a plot of settlement versus log time can be prepared if monitored for several years.

For surveys where a survey level is used, bridge settlement should be measured at four locations (i.e., at each corner of the bridge) to check for angular distortion and differential settlement. Wall settlement is recorded by the rod off the top of the CMU facing block adjacent to the superstructure, and superstructure settlement is measured with the rod off a guardrail hanger bolt (figure 36). The disadvantage of the survey level method is that lateral deformations of the wall face are not measured.



Source: FHWA.

Figure 36. Illustration. Survey level method for superstructure and wall settlement.

For surveys with an EDM system, the total station is referenced off of a permanent pole embedded beneath the frost line and within accessible sight to both abutment face walls. The permanent pole should be installed prior to placement of the steel girders on the footings (figure 37). Targets placed on the abutment wall face and the bridge beam or footing are then used to measure both vertical and lateral movement relative to the permanent pole. Figure 38 shows an example of this for the Tiffin River Bridge in Defiance County, OH. In this figure, lateral and vertical movements of the abutment wall face were measured using custom reflective survey targets, which are shown in the black circles, while movement of the GRS abutment was measured using targets placed on the concrete footing itself, which are shown in the red circles. The difference between the two readings (i.e., movement of the abutment and movement of the wall face) provides the compression of the GRS abutment alone, not including foundation settlement. Inclinometers can also be used to measure vertical and lateral deformations.

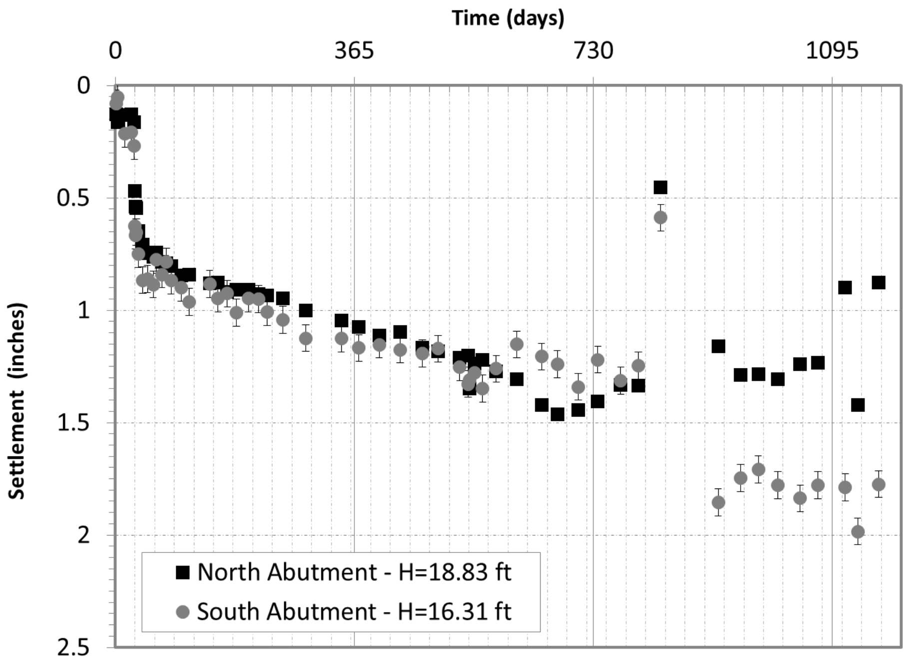
Source: FHWA.

Figure 37. Photo. Location of total station reference pole.

Source: FHWA.

Figure 38. Photo. Location of survey targets on Tiffin River Bridge in Defiance County, OH.

Figure 39 shows the settlement record for the Tiffin River Bridge measured over approximately 3.5 years. As shown in the settlement history, the survey reference pole moved at approximately 833 days, with the subsequent shift in the settlement for each abutment. If this happens, the data could be corrected assuming no other evidence indicates additional settlement or heave of the abutments.



Source: FHWA.

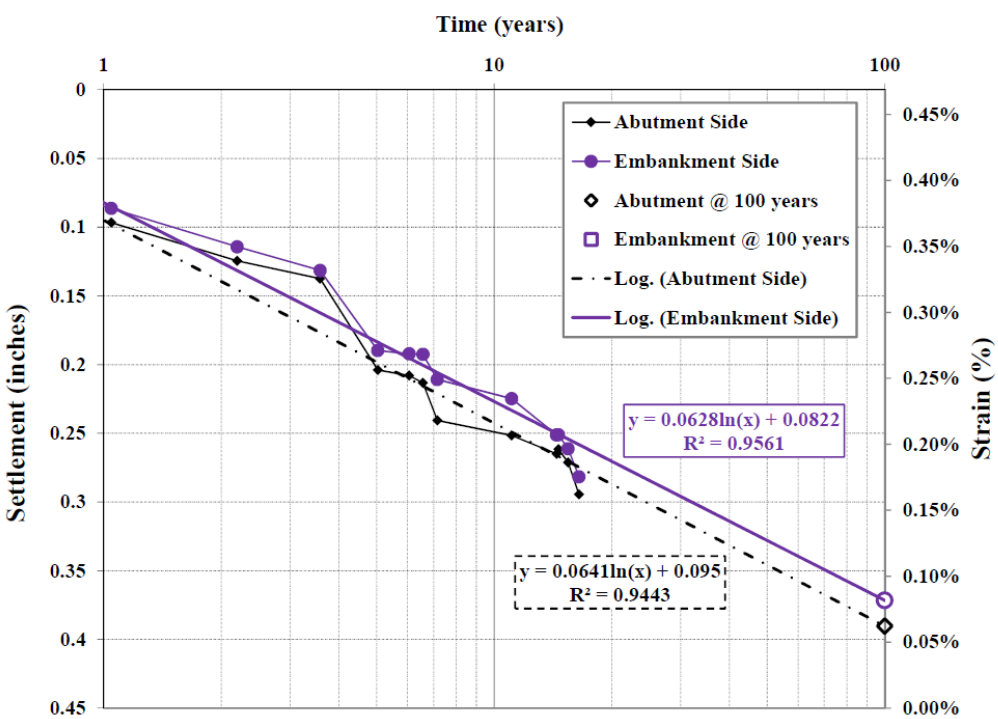
Figure 39. Graph. Tiffin River Bridge settlement record with the survey reference pole shifted at 833 days.

One of the longest known monitoring programs for GRS is being conducted through the measurement of settlement above a tunnel, built into the prototype FHWA IBS at TFHRC in 1999 to illustrate the versatility of the technology (figure 40). The height of the tunnel walls is 8 ft, the reinforcement spacing is 6 inches, and the vertical stress is 3,900 lb/ft^2 due to overburden surcharge.

Figure 41 shows the tunnel settlement history collected over the past 17 years to illustrate the predictive deformation behavior of GRS. To estimate long-term settlement over the 100-year design life, time is plotted on a semi-log scale. The data indicate that the total estimated settlement in 100 years is about 0.4 inch, which equates to 0.40 percent strain for this particular composite.

Source: FHWA.

Figure 40. Photo. Tunnel built into FHWA prototype IBS.



Source: FHWA.

Figure 41. Graph. FHWA prototype IBS abutment tunnel settlement history.

To measure the differential settlement between the approach and the bridge, it is suggested to use a high-speed inertial profiler that can provide the roadway profile throughout the transition. The profiler is a common tool typically used by transportation departments to measure the roughness of pavements; however, it can also be used to quantify the bump at the end of the bridge.

6.3 THERMAL INTERACTION

Thermal cycles occur on every bridge structure due to sustained temperature variations. The severity of the expansion and contraction depends on the coefficient of thermal expansion of the bridge. Observations of GRS-IBS projects built in various climate zones indicate considerable compatibility between the superstructure and the integrated approach, resulting in a smooth transition. During the past several years, many IBSs were instrumented to monitor in-service performance and substructure/superstructure interaction.[49–51] From in-service measurements, it has been determined that the bridge behaved more similar to an unrestrained system in a GRS-IBS due to the elastic properties of the integrated approach. Measurements also indicate that the integrated approach remained engaged against the backwall during the thermal cycles.[49] Results also show that both the abutment face and beams moved together during the thermal cycle to illustrate the compatibility of the IBS.[51]

GRS-IBS accommodates lateral thermal movement through the integrated approach behind the beam ends. The road base backfill in this zone is wrapped with geotextile and then compacted directly against the beam end. This process is described in detail in chapter 7. The wrapped face confines the soil and allows the beam to contract without the fill behind the beam ends sloughing off to fill the void, which can lead to excessive pressures when the bridge expands again.

To measure the in-service performance due to thermal interactions, pressure cells, inclinometers, and strain gauges can be used. The pressure cells should be installed behind the beam ends (figure 42). This will give an indication of the passive pressures resulting from thermal expansion of the superstructure. IPIs can also be placed at the interface between the superstructure and integrated approach to evaluate the movement. As an example, figure 43 and figure 44 show the center line position of inclinometer casing between the abutment backwall and the integrated approach to monitor thermal movement of superstructure against the integrated approach. Strain gauges on the superstructure can be used to measure thermal expansion and contraction of the bridge.

Source: FHWA.

Figure 42. Photo. Lateral earth pressure cell behind the GRS abutment backwall.

Source: FHWA.

Figure 43. Photo. Casing for a vertical IPI at the interface between the backwall and the integrated approach.

Source: FHWA.

Figure 44. Photo. Close-up view of an IPI used to measure thermally induced movement.

6.4 SCOUR MONITORING

The scour countermeasure (e.g., riprap protection) should be monitored during each bridge inspection or after an extreme flood. An indicator of scour on an abutment face or wing wall can be achieved by using colored blocks on the bottom five to eight rows of the abutment (see chapter 7). Any movement of riprap or other countermeasures should be noted and repaired to prevent scour from progressing and undermining the RSF or the abutment. In addition, any resulting debris should be removed. In all current installations of GRS abutments, no problems have been reported. It should be noted that these installations have been built in low-scour potential environments and with appropriate countermeasures, as recommended in current practice.[3,14]

CHAPTER 7. CONSTRUCTION

7.1 INTRODUCTION

GRS construction uses basic earthwork methods, primarily for excavation and compaction, along with sound general construction practices. The materials needed for GRS construction are readily available, which is a benefit of the generic nature of the system. This chapter provides guidance on most field-related scenarios, particularly with respect to a concrete modular block facing; however, this guidance can also be adapted to other GRS structures built with different facing types. All methods that are presented have been field-tested and applied during the construction of GRS-IBS. The techniques outlined can be applied to efficiently construct the layered system and have been proven to quickly construct the GRS-IBS. The contractor will ultimately choose the methods most efficient for the site, crew, and equipment on hand.

GRS construction has two principal components: (1) logistics and (2) aspects associated with actual construction. Logistics occur after the final design and before construction, outlining a plan for implementation and control of the construction process. Even though building a GRS abutment is for the most part as simple as repeatedly placing a row of facing block, a layer of well-compacted granular fill, and a sheet of reinforcement, the process is hampered without adequate planning to ensure optimum flow and placement of material throughout the course of the project.

Design plans should be made to easily provide information on the abutment layout, the reinforcement schedule, and the facing block schedule. The plans should also contain information on the limits of excavation and details about assembly of the GRS-IBS. It is important to lay out the abutments to scale, with accurate dimensions of the materials used to meet the planned elevations and limits of the abutment with respect to the superstructure and integrated approach. Additionally, an accurate illustration allows for a more precise estimate of material quantities that can be detailed on the plan with construction notes.

This chapter conveys the importance of the following details to ensure rapid GRS construction:

- **Careful attention to the first row of blocks**: Since all other courses of block are built off the first row, it is essential to ensure that the bottom row is level and even for fast construction.

- **Optimization of crew size and equipment for enhanced productivity**: Too many laborers or excess onsite equipment can cause confusion and slow down the construction process.

- **Allowance of time for a labor crew to adjust to the construction of the GRS-IBS**: Having each crew member do his or her part in the three basic steps of GRS construction (i.e., laying a course of facing block, compacting a layer of granular backfill, and placing a layer of reinforcement) dramatically improves productivity.

- **Establishment of a central position of the excavator**: Typically, it is best to limit movement of the excavator by locating it toward the back of the abutment where it can both reach and place material without moving.

7.2 LABOR, TOOLS, AND EQUIPMENT

The labor and equipment needs are minimal for GRS abutments and IBSs and do not require much specialized training or mobilization. The following subsections provide additional detail on the labor, tools, and equipment needs.

7.2.1 Labor

In many situations, a typical labor crew on GRS-IBS projects consists of five workers: four laborers and one equipment operator (figure 45). The equipment operator is central to the project and provides support to the labor crew. He or she is responsible for shaping the excavation to facilitate construction of the RSF and the GRS abutment in addition to placing fill material and moving facing units into the work area. Typically, one member of the labor crew has the role of foreman and is responsible for the layout of excavation limits and grades, alignment of wall face, placement of facing blocks, compaction of fill, and placement of geosynthetic reinforcement, as well as other activities to streamline production and the flow of material to the job site.

Source: FHWA.

Figure 45. Photo. Typical labor crew with centrally located track hoe.

7.2.2 Tools and Equipment

For most construction projects, specialized equipment is not required to construct GRS-IBS. Simple tools that are readily available and relatively inexpensive can be used. These include hand tools, measuring devices, and heavy equipment. The contractor may modify the following lists of tools and equipment depending on the site, crew, and size of the IBS.

Typical hand tools include the following:

- Gravel rake (concrete spreader).
- Shovels (flat blade and spade).
- Heavy rakes.
- Broom to sweep top of blocks.
- Whisk broom.
- A 2- to 3-lb sledgehammer and wood two-by-fours to align blocks.
- Heavy rubber mallet.
- Spade trowel.
- Razor knives or utility knives to cut reinforcement.
- Hand tamper with metal base plate.
- Chainsaw to cut reinforcement roll.
- Concrete saw.
- 5-gal bucket.
- Block lifter.
- Standard concrete mixing and finishing tools.

Typical measuring devices include the following:

- Survey equipment.
- Laser level.
- String line to align blocks.
- A 4-ft carpenter's level.
- Plum bob to check wall batter.
- Measuring tapes.
- Chalk line.

Typical heavy equipment includes the following:

- Walk-behind vibratory plate tampers (200 lb and 18 inches wide or larger).
- Track hoe excavator.
- Riding smooth drum vibratory roller (compacting 3.28 ft from wall face).
- Pallet forks for the excavator (for moving CMU block in and out of work area).
- Trash pump and hose for dewatering the foundation excavation.
- Backhoe (as needed for material staging).

7.3 SITE PREPARATION

GRS is built from the bottom up and generally from within the footprint of the structure. Staging and delivery of materials to the site should allow for continuous GRS construction and effective use of the space. Delivered material should be easily accessible to the excavator, which is the central piece of equipment. As shown in figure 46, the excavator is positioned inside the wall area for easy placement of fill, block, and other materials. Labor should be organized to assemble construction materials as needed on the work platform.

Source: FHWA.

Figure 46. Photo. Cut slope of retained soil.

7.3.1 Site Layout

Site preparation begins with a survey of the bridge site to stake limits for the excavation. Reference stakes should be located in an area where they will remain undisturbed during construction of the base of the wall, which is usually about 5 ft from the excavation.

The base of the GRS abutment and wing walls should be constructed to within 1 inch of the staked elevations. The external GRS abutment and wing walls should be constructed to within ±0.5 inch of the surveyed staked dimensions.

7.3.2 Excavation

All excavations should comply with Occupational Safety and Health Administration requirements.[52] Excavation of the site involves shaping the slope for temporary slope stability, safety, and constructability. The temporary cut in the retained soil should be designed to accommodate movement of labor. The design of a temporary excavation needs to consider the loading imposed by heavy equipment and the reach limits of the excavator. Figure 47 shows a typical cut slope in stiff clay. The excavation should include provisions for drainage with a sloped cut to facilitate the movement of water. Any open excavations that form a pit should be backfilled with crushed aggregate and compacted. Excavation also includes the clearing and grubbing of vegetation. In situations where the retained fill is stable, the volume of excavation can be limited to reduce the size of the GRS composite. In the case of an abutment application, this would form a horseshoe-shaped excavation, as shown in figure 45 and figure 47.

Source: FHWA.

Figure 47. Photo. Horseshoe-shaped excavation with native soil still intact in middle.

Building in a flooded excavation can be addressed through a variety of methods ranging from using dewatering pumps (figure 48), building a coffer dam with sheeting (figure 49), or quickly compacting the structural backfill to create the stable working platform. The selection will depend on the influx of water at the site.

Figure 48. Photo. Dewatering during excavation of the RSF.

Copyright: King County, WA.

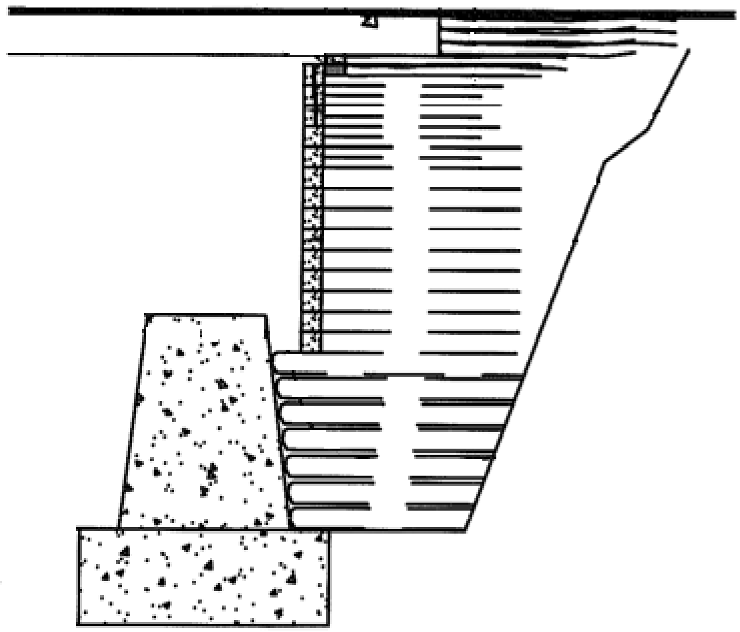
Figure 49. Photo. Sheet-pile supported excavation.

7.3.3 Placement of Abutment Behind Existing Substructure

In some situations, it may be beneficial to build the GRS-IBS behind an existing substructure. Project feasibility, environmental considerations, and other factors need to be assessed before selecting this type of project layout. Building the bridge behind an existing substructure often requires the removal of the top part of the existing abutment walls to provide additional space for the width of the new GRS-IBS. Figure 50 through figure 52 illustrate this technique. Note that the design of the GRS-IBS is the same whether it is built behind an existing abutment or not.

Copyright: St. Lawrence County, NY.

Figure 50. Photo. GRS-IBS built behind an existing concrete abutment.



Copyright: St. Lawrence County, NY.

Figure 51. Illustration. Cross section of a GRS-IBS built behind an existing concrete abutment.

Source: FHWA.

Figure 52. Photo. Building the RSF behind an existing abutment.

7.4 RSF

The depth and footprint of the excavation for the RSF should be based on external stability and, if necessary, the hydraulic analysis. The base of the RSF should be cut smooth. It should be excavated to uniform depth, and all loose, unstable material should be removed from the site (figure 53). If the base of the excavation is left open, it should be graded to one end to facilitate the removal of any intrusion of water with a pump. If flooded, all water should be removed along with soft, saturated soils. The excavation should be backfilled as soon as possible to provide a suitable foundation and avoid adverse weather delays. The construction of the RSF can typically be completed in less than 1 day but is dependent on the size and depth of excavation, type of materials, equipment, and experience.

Figure 53. Photo. RSF excavation below the stream level.

The base of the excavation should be compacted before construction of the RSF. This may require proof rolling, and any soft spots or voids should be backfilled with compacted fill material. Figure 54 shows the preparation of the RSF cut.

Source: FHWA.

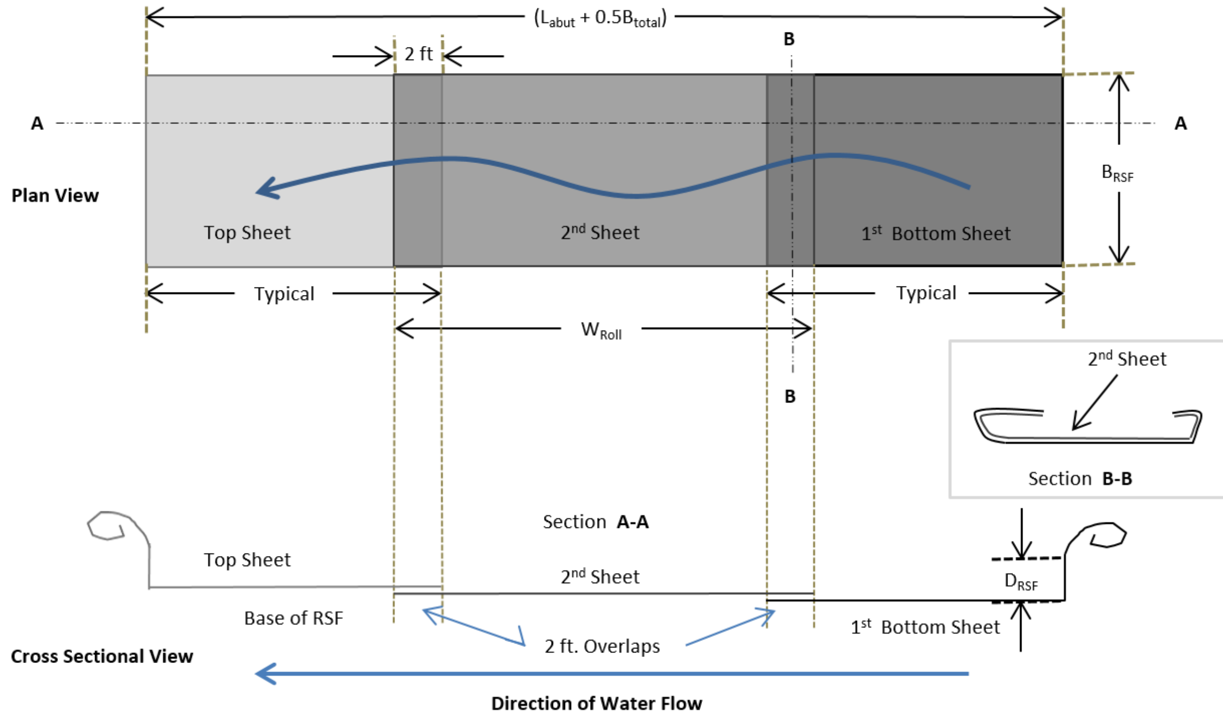
Figure 54. Photo. RSF cut preparation.

The RSF shall be encapsulated in geotextile reinforcement placed perpendicular to the abutment face to protect it from possible erosion due to scour (figure 55). The reinforcement sheets should be measured and sized to fully enclose the RSF on three sides: the face and the two wing wall sides.

Source: FHWA.

Figure 55. Photo. Encapsulation of fill in RSF.

If the GRS abutment is built on an RSF, particularly for water crossings, and if more than one sheet of reinforcement is needed to encapsulate the excavation, the first reinforcement sheet placed in the excavation shall be on the upstream side of the RSF with the subsequent sheet(s) placed on top with a 2-ft minimum overlap (figure 56). All overlapped sections of reinforcement in the area of the RSF should be oriented to prevent running water or surface runoff from penetrating the layers of reinforcement. The first layer of reinforcement should be placed on the upstream side of the abutment with subsequent layers (if needed) overlapped a minimum of 2 ft on the downstream side. This prevents water from infiltrating the RSF. The wrapped corners of the RSF need to be tight and without exposed soil within the RSF to complete the encapsulation.



Source: FHWA.

Figure 56. Illustration. RSF geotextile layout with respect to water flow direction.

Where:

L_{abut} = abutment length.
B_{total} = total base width of the GRS abutment including the width of the facing.
B_{RSF} = base width of the RSF.
W_{roll} = width of the reinforcement roll.
D_{RSF} = depth of the RSF.

Note that in water crossings and some soil conditions, generic concrete bin blocks (2 by 2 by 6 ft) have been used to form the perimeter of the RSF to facilitate construction (figure 57 and figure 58). Alternatively, welded wire baskets have also been used to form the perimeter of the RSF.

Figure 57. Photo. Construction of the RSF with large concrete bin blocks.

Figure 58. Photo. Completed RSF constructed within perimeter of concrete bin blocks.

Typical reinforcement spacing in the RSF is 12 inches. The reinforcement should be pulled taut to remove all wrinkles prior to placing and compacting the structural backfill. Fill should be placed from the face to the back to roll folds or wrinkles to the free end of the reinforcement layer.

The RSF should be constructed with structural fill, as specified in chapter 3. The structural fill is to be compacted in accordance with section 7.5 in compacted lifts not to exceed 8 inches. The first course of wall block sits directly on the RSF, as shown in figure 59, so it is important that the fill material is graded and level before encapsulating the RSF.

Source: FHWA.

Figure 59. Photo. Placement of wall block on wrapped RSF.

After this, a channel rock geotextile apron can be fixed to the abutment to stabilize and prevent the filtration or loss of material beneath the riprap for GRS-IBS construction for water crossings,. The geotextile apron can be placed beneath the first course of the facing blocks; however, if greater than 0.5 inch of material is used to level the first course of the facing block on the RSF, it is suggested to place the channel rock geotextile apron between the first and second courses to protect the leveling material from erosion.

The use of solid block at the base of the abutment should be considered to protect against vehicle impacts or any damage due to placement of channel rock that extends above the solid block zone. For water crossings, riprap protection should be placed in a manner to prevent damage to the wall face. Impact of large rock or concrete fragments during placement can crack the CMU block. Larger rocks should be uniformly distributed and placed firmly in contact with each other, with smaller rocks and fragments filling the voids between the larger rocks. This procedure often requires hand placement of smaller rocks to fill the voids. Chapter 9 provides repair procedures in the event that any CMU block is damaged.

7.5 COMPACTION

Compaction of the backfill should be to at least 95 percent of maximum dry density according to AASHTO T 99 for a well-graded aggregate and a method specification (e.g., three passes of the compactor) for an open-graded aggregate.[53] Backfill material containing fines should be compacted at a moisture content close to optimum (±2 percent). Lifts of 8 inches should be compacted using vibratory roller compaction equipment. The facing blocks provide a form for each lift of fill. Other stiffness-based compaction control methods can be used. For open-graded fills, compact to non-movement or no appreciable displacement and both the compaction of the aggregate and movement of the facing block should be visually assessed as outlined in section 7.5.1 and section 7.7.3.

Since the facing elements are not rigidly connected to the reinforcement, hand-operated compaction equipment (e.g., a lightweight mechanical tamper, plate, or roller) is recommended within 3 ft of the front of the wall face. It is very important for adequate GRS performance that the backfill is properly compacted. The backfill in the bearing bed reinforcement zone should be compacted to 100 percent of the maximum density according to AASHTO T 99 for a well-graded backfill or according to a method specification if the backfill is open-graded.(53)

Onsite compaction equipment should be selected to achieve the required density of the fill materials. Considering that compaction is critical to the success of the project, compaction equipment should be in good operating order for efficient use. In addition, backup equipment should be available to provide quality construction throughout the project and to avoid construction delays.

7.5.1 Compaction Procedure

Once fill is placed at the required thickness and graded, all areas behind the modular block should be compacted to the required density. Any depression behind the facing block should be filled level to the top of the modular block prior to compaction.

Compaction directly behind the modular block should be performed in a manner that maintains wall alignment while improving the density of fill behind the block. This can be achieved in the following ways:

- Placing a fill lift directly behind the modular block face and rodding or foot tamping along the row of block while exerting downward pressure on the block to prevent lateral movement. For multiple lifts, the top lift height is slightly higher than the block to compensate for compression of the fill during compaction.

- Using a lightweight vibratory plate compactor directly behind the modular block while exerting downward pressure on the block to prevent lateral movement.

- Using larger vibratory compactors for the remainder of the fill area 3 ft from the face of the GRS wall. Outward block movement should be checked for and adjusted accordingly.

The most common compaction QC tool is the nuclear density gauge. Other instruments are also available for compaction control, such as the Clegg hammer, the soil stiffness gauge, or the falling weight deflectometer. These devices are typically used by correlating their measurements to soil density and moisture content. Method-based compaction specifications can also be used. For open-graded fills, compact to non-movement or no appreciable displacement, and the fills should be visually assessed.

7.6 REINFORCEMENT

Generally, the length of the reinforcement layers will follow the cut slope, as shown in figure 20. While the reinforcement layers in the GRS abutment can be any geosynthetic, the RSF and integrated approach should be constructed and encapsulated with a geotextile to confine the compacted granular fill. The geosynthetic should be placed so that the strongest direction is perpendicular to the abutment face, as shown in figure 60 for a geotextile. Where the roll ends, the next roll should begin. Overlapping between sheets is not required. The geosynthetic reinforcement should extend between layers of CMU blocks to provide a frictional connection. The geosynthetic reinforcement should cover a minimum of 85 percent of the top surface of the CMU blocks; any excess can be removed by either burning it with a propane torch or cutting it with a razor knife.

Source: FHWA.

Figure 60. Photo. Geotextile reinforcement rolled out parallel to the wall face (strong direction perpendicular to the wall face).

After the geosynthetic is rolled out, it should be laid so that it is taut, free of wrinkles, and flat. The geosynthetic can be held in place with the fill. Placement of fill should be from the wall face backward to remove and prevent the formation of wrinkles in the geosynthetic. A conscious effort should be taken during placement of fill to prevent the development of wrinkles.

Splices of reinforcement can occur without overlap. Splice seams should be staggered to avoid a continuous break in the reinforcement throughout the GRS structure. Following this procedure, all splice seams can run either perpendicular or parallel to the wall face.

Overlaps of adjacent geosynthetic should be trimmed where they are in contact with the surface of the facing block to avoid varying geosynthetic thicknesses between the CMU block. Any seams in the geosynthetic should be staggered with each successive layer of the GRS abutment. All seams between adjacent sheets of geosynthetic located in the area beneath the footprint of the beam seat should be perpendicular to the abutment wall face.

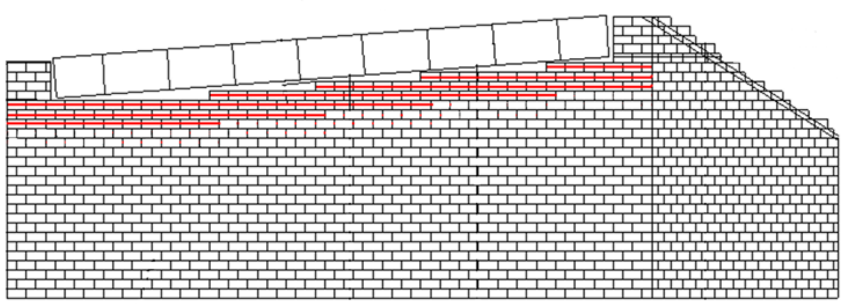
7.6.1 Operating Equipment on Geosynthetic Reinforcement

Driving is not allowed directly on the geosynthetic reinforcement. A minimum 6-inch layer of granular fill should be placed prior to operating any vehicles or equipment over the geosynthetic reinforcement. In the bearing reinforcement zone, hand-operated compaction equipment should be used over the 4-inch lifts to prevent excessive installation damage of the geosynthetic reinforcement. Rubber-tired equipment may pass over the geosynthetic reinforcement at speeds less than 5 mi/h. Skid steers and tracked vehicles can cause considerable damage to the geosynthetic reinforcement. For example, a track hoe once operating on a GRS structure turned and pulled the fabric, causing deformation to the wall face. For this reason, it is recommended to restrict the use of these vehicles on GRS structures. If absolutely necessary, use may be permitted provided no sudden braking or sharp turning occur and a minimum 6-inch cover is placed.

7.6.2 Bearing Reinforcement Bed

The bearing reinforcement bed provides additional strength in the upper GRS wall layers directly beneath the bearing area of the superstructure. These reinforcement layers are not sandwiched between two consecutive rows of block but are placed behind the facing block at 4-inch spacing. This 4-inch reinforcement spacing is generally placed in the top three to five layers of the GRS abutment or as determined by the design (see chapter 4).

Bearing bed reinforcement spacing in superelevated abutment walls requires additional planning. The 4-inch reinforcement spacing needs to be placed in the top three or more courses of block at each elevation across the length of the abutment wall (see the red reinforcement lines in figure 61). The reinforcement schedule will guide field personnel in the proper placement of the geosynthetic along a wall block course.



Source: FHWA.

Figure 61. Illustration. Superelevation reinforcement schedule.

7.6.3 Superelevation

The reinforcement layers become stair-stepped in the upper wall layers as the superelevation of the abutment is constructed (figure 62). The reinforcement terminates along the angle surface of the superelevation. The GRS wall reinforcement schedule should show the termination of each layer of reinforcement across the abutment wall from low to high elevation (figure 61).

Copyright: Defiance County, OH.

Figure 62. Photo. Superelevation reinforcement layers.

7.7 WALL FACE

This manual focuses on the use of concrete modular blocks for the wall facing; however, since GRS is internally stable, any facing elements can be used in construction. For simplicity, CMUs are used throughout this section to refer to the facing. For flexible facings other than the CMU block (including different types of concrete modular blocks, wrapped, timber, natural rock, or welded wire basket facing), alternative construction guidelines may need to be followed and/or developed. These other facing systems are described by Wu et al.[40] The general design guidelines for GRS-IBS, however, remain the same as those in this manual.

7.7.1 Leveling Course

Setting the first course of facing block level and grading it is critical in maintaining wall alignment for the entire height of the abutment. Typically, the first course is placed on top of the RSF directly on the geotextile; however, due to the large aggregate size of the RSF fill material, a thin leveling layer of fine aggregate can help set the facing blocks to grade and prevent them from rocking. The leveling layer should be kept to a minimum thickness (i.e., no more than 0.5 inch). If the leveling layer exceeds this thickness and there is the potential for water to erode and undermine the aggregate, mortar or grout should be placed in the gap between the RSF and the first course.

7.7.2 Setting the CMU Block

CMU block wall construction should begin at the lowest portion of the excavation, with each layer placed horizontally as shown in the project plans. Each layer should be constructed entirely before beginning the next layer. A stretcher or running bond should be maintained between courses of block so that the joints between the blocks are offset with each row.

Since the CMU blocks are dry stacked without mortar, it is important to avoid cracking the blocks and to maintain a horizontal uniform elevation by sweeping the top surface of the blocks clean of debris and fill material prior to the placement of the next layer of geosynthetic and CMU blocks. Gravel material between layers of blocks creates point loads that can cause cracks. Also, gravel material between the blocks causes them to rock, making it difficult to secure a good fit.

When setting a course of blocks, each block should be placed tightly against the adjoining block, preventing gaps from which fill material can escape. Before proceeding to the next layer, it is often useful to walk along the top of the blocks to easily identify a poorly seated block.

To avoid cutting a block when the CMU block schedule shows the wall terminating with half a block, a full CMU block can be turned 90 degrees, placing the 8-inches width toward the face. This typically occurs at the termination of a wing wall. The end block that forms the termination does not have to be a corner CMU block (with two finished sides) because the ends of most wing walls are embedded into the fill slope.

7.7.3 Wall Face Alignment

When placing and compacting fill behind the CMU blocks, it is sometimes necessary to set the blocks back about 0.5 inch to allow for lateral outward movement of the CMU blocks during compaction. It should be noted that each combination of wall facing and backfill reacts differently during the compaction process, and adjustment of the setback distance between block courses should be performed as needed to maintain the necessary batter.

Alignment of the GRS abutment wall should be checked for plumbness at least every other layer, and any deviations greater than 0.5 inch should be corrected. Wall face verticality or batter should be maintained to conform to the limits and shape of the abutments to avoid potential as-built changes in the setback distance and clear space. While there are some cases of GRS abutments being built with poor face alignment, without exhibiting instability, wall appearance is a serviceability issue because questions may arise on whether the wall was built with poor alignment (e.g., a bulge) or if it experienced post-construction deformations. Before placing the backfill, every other row of block alignment should be checked with a string line referenced off the back of the facing block from wall corner to corner (figure 63).

Figure 63. Photo. Checking block alignment with string line reference from the back of the block.

If CMU blocks become displaced during construction, they can often be hammered back into position using a 3-lb sledgehammer and a block of wood as protection. If the CMU blocks are excessively out of alignment, the fill material needs to be excavated, the CMU blocks need to be repositioned, and the fill material needs to be replaced and recompacted.

7.7.4 Block Alignment for Battered Walls

Block alignment for battered walls is similar to that for vertical walls. In abutment situations where the face wall turns to form the wing wall, however, it is necessary to trim blocks on either end to account for the reduced wall length. All cuts should be performed to maintain the standard running or stretcher bond between the rows of dry-stacked blocks, with the vertical joints of each course midway between those of adjoining courses.

In special situations, negative battered walls (not abutments) have been constructed when the top area needs to be greater than the bottom, as in the case of road widening shown in figure 64. The negative batter can be created by offsetting the CMU block by a measured amount in consecutive wall layers and then filling and compacting as specified. Again, this practice is typically limited to walls and has not been used for GRS abutments, but it helps highlight the stability of closely spaced GRS.

Figure 64. Photo. Negative batter wall face.

7.7.5 Superelevation

When the plans shows a superelevation for the bridge, the top courses of CMU blocks beneath the superstructure should be trimmed to match the elevation difference and clear space across the abutment (figure 65). This will produce a sloped face wall and aid in construction of the beam seat. One method is to snap a chalk line along the back of the block at the superelevation slope. A carpenter's angle finder can also be used to mark the cut.

Source: FHWA.

Figure 65. Photo. Blocks trimmed to match superelevation.

7.7.6 Wall Corners and Curves

Right-angle wall corners, as shown in figure 66, are constructed with CMU corner blocks that have architectural detail on two sides, providing an aesthetic finish. Facing wall and wing wall courses should be staggered to form a tight, interlocking, stable corner.

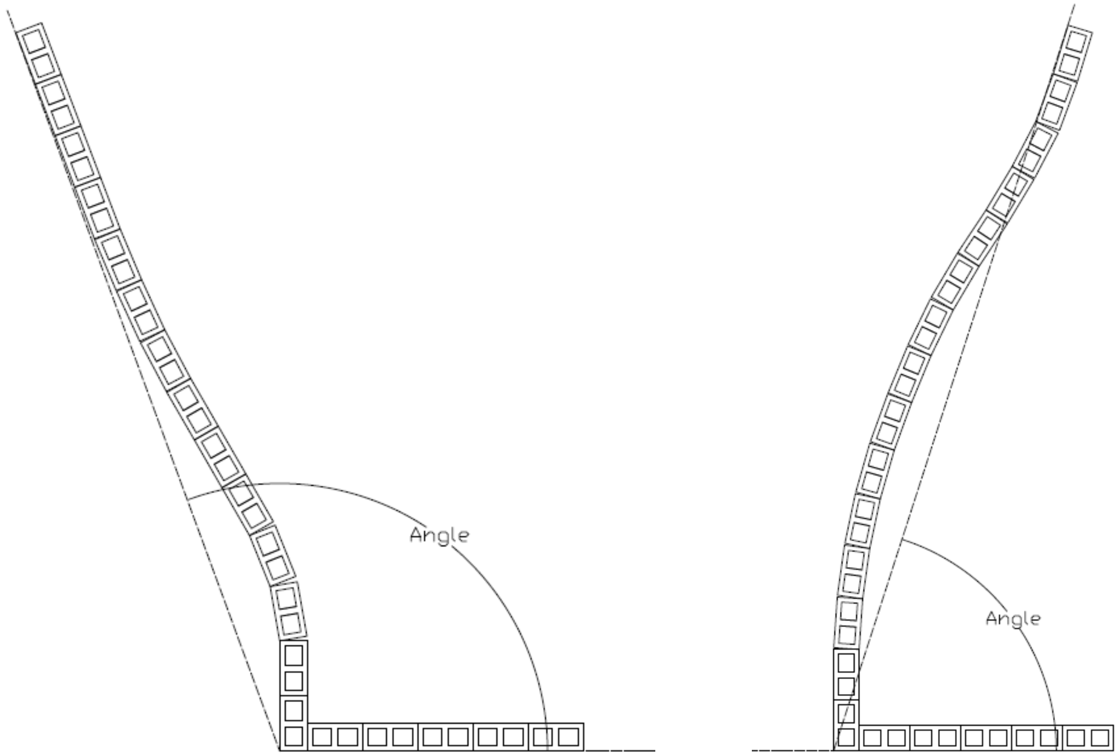
Source: FHWA.

Figure 66. Photo. Right-angle wall corner.

Walls with angles larger or smaller than 90 degrees require additional effort. The corner blocks need to be cut to form the angled face. As a result, a vertical seam or joint is formed at the corner (figure 67). Corners with vertical seams may have open block joints, making it prudent to fill the corner blocks with a concrete mix and install bent rebar to close and connect the seam at each course of block, as shown in figure 68. This procedure secures the two faces and prevents compaction-induced separation during construction of subsequent GRS layers. It may also be used wherever added strength at the wall corner is desired. Alternatively, the wing walls can be built without cutting the block by gradually turning the blocks to avoid the need for cutting blocks and the subsequent vertical joint at the junction between the wing wall and abutment faces (e.g., see figure 69 through figure 71).

Figure 67. Photo. Vertical seam in the wing wall.

Figure 68. Photo. Rebar installed in the vertical seam prior to grout.



A. Obtuse wall corner. B. Acute wall corner.

Source of subfigure images: FHWA.

Figure 69. Illustrations. Examples of alternative obtuse and acute wall corner details with rectangular blocks.

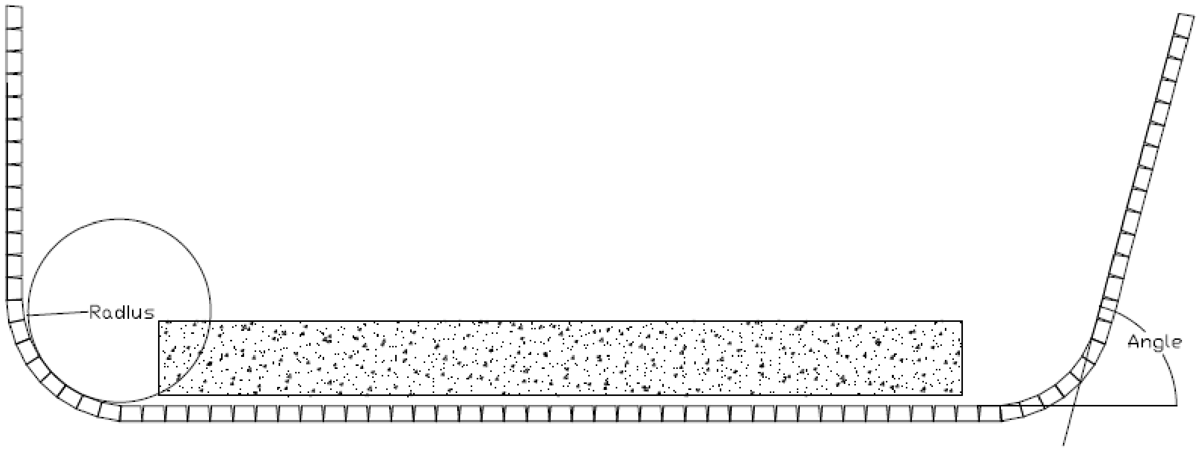
Source: FHWA.

Figure 70. Photo. Alternative wing wall obtuse angle corner detail with CMU blocks.

Source: FHWA.

Figure 71. Photo. Alternative wing wall acute angle corner detail with CMU blocks.

Curves can also be constructed in lieu of a sharp corner when using SRW blocks. Figure 72 illustrates how the layout of a GRS abutment can be formed in a curved shape from the transition into the wing walls. The size of the radius to create the convex curved wing wall is dictated by the tapered shape of the SRW blocks. These layouts are aesthetic and may offer some advantages in some hydraulic conditions depending on the project requirements. Note that these details can create a larger footprint area of the abutment, leading to an increased volume of fill material needed. Regardless, the layout of the block schedule should include details of how the parapets link to the sides of the superstructure as discussed in section 7.9.3 found later in this chapter. An example of a curved corner on a GRS-IBS under construction is shown in figure 73.



Source: FHWA.

Figure 72. Illustration. Abutment layout with curved wing walls.

Copyright: Hamilton County, IN.

Figure 73. Photo. GRS-IBS with curved corner details under construction.

7.7.7 Top of Wall Facing

The top three courses of CMU blocks in the abutment are susceptible to movement simply from not having the weight of successive layers holding them in place. To prevent displacement, the hollow cores of the top three courses of CMU blocks are filled with a concrete wall fill and pinned together with No. 4 rebar, preferably epoxy-coated, and embedded with a minimum 2-inch cover (figure 74).

Copyright: Defiance County, OH.

Figure 74. Photo. Connecting the top courses of blocks.

To grout and pin the top of the wall, the reinforcement between the top two courses of CMU blocks needs to be removed to open the core for placement of concrete wall fill and a 20-inch-long No. 4 rebar dowel, preferably epoxy-coated with 2-inch cover (see chapter 3). This can be accomplished either by cutting the reinforcement with a razor knife or by burning the geosynthetic reinforcement.

The concrete wall fill is placed in two steps. After the block void is filled with concrete to the top of the block and the steel rebar is inserted, a thin layer of the same concrete mix is placed on top of the block to form the coping cap, as shown in figure 75 and figure 76. The coping is then hand-troweled either square or round and sloped to drain. A wet-cast cap is more durable than a dry-cast cap and eliminates the need to furnish and install a separate cap unit.

Source: FHWA.

Figure 75. Photo. Square coping cap.

Copyright: Defiance County, OH.

Figure 76. Photo. Rounded coping cap.

Once the top of the wall has been tied together, care should be taken to avoid any construction activity that may pull on the top layer of the reinforcement. The frictional connection between the block is strong, and when courses are pinned together, the entire grouted wall face can be pulled out of alignment.

If another type of concrete modular block is used for the abutment face, the designer will need to develop a suitable method of connection. Many proprietary SRW systems have pre-engineered methods of connection, which may or may not be compatible with the wall face layout or pinning and grouting as previously discussed. An alternative method may include the use of concrete adhesives. Regardless, copping and connecting the top wall face is important.

7.8 BEAM SEAT

The beam seat is constructed directly above the bearing bed reinforcement zone. The superstructure is then positioned on top of the beam seat, as shown in figure 77 and figure 78. The purpose of the beam seat is to ensure that the superstructure bears on the GRS abutment and not the wall facing block and to provide the necessary clear space between the superstructure and the wall face. Typically, the clear space is 3 inches, or 2 percent of the abutment height, depending on the required design (see chapter 4).

Source: FHWA.

Figure 77. Photo. Box beam placed on the beam seat.

Copyright: Defiance County, OH.

Figure 78. Photo. Detailed view of a box beam placed on a beam seat.

In general, the thickness of the beam seat is approximately 8 inches and consists of two 4-inch lifts of wrapped-face GRS. Before construction of the beam seat, the cores of the CMU blocks on the abutment wall face must be pinned with No. 4 rebar and filled with concrete wall mix (figure 79).

Source: FHWA.

Figure 79. Photo. Bearing area block grouted prior to beam placement.

7.8.1 Beam Seat Procedure

Once the block elevation beneath the bearing area is established and the hollow cores are filled with grout, the beam seat is ready for construction. The following steps should be used to construct the beam seat:

1. Place precut 4-inch-thick foam board on the top of the bearing bed reinforcement. Sometimes, a thin layer of backfill may be necessary beneath the foam board for grading purposes and to ensure the proper clear space height and drainage (crown in bridge) (figure 80). The foam board should butt against the back face of the CMU block. The exposed edge of the foam board helps form the nose of the reinforcement wrap across the length of the bearing area. The stiffness of the foam board should allow it to compress as the beam settles (see chapter 3).

Source: FHWA.

Figure 80. Photo. Foam board and 4-inch-thick block assembly to form a beam seat.

2. Set the 4-inch-thick solid concrete blocks on top of the foam board across the entire length of the bearing area (figure 81). The back edge of the top CMU face block holds the

4-inch-thick concrete block in place during compaction. Note that the distance between the top of the grouted CMU block and the top of the beam seat (the clear space) is the distance the beams can settle before bearing on the facing blocks.

Source: FHWA.

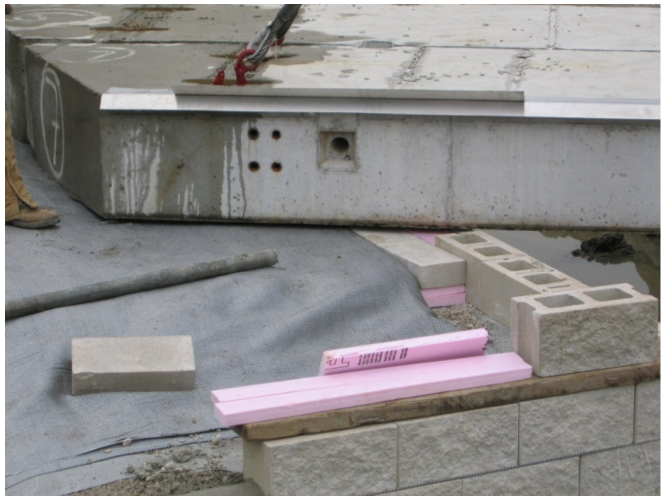
Figure 81. Photo. A 4-inch-thick concrete block on top of a foam board against the top of the CMU face block.

3. Use the first 4-inch wrapped layer of compacted fill as the thickness to the top of the foam board (figure 82).

Source: FHWA.

Figure 82. Photo. First 4-inch wrap butted against the foam board.

4. Place the second 4-inch wrapped layer of compacted fill to the top of the 4-inch-thick solid block, creating the clear space as shown in figure 83. The top of this layer controls the beam elevation and should therefore be carefully compacted and graded.



Source: FHWA.

Figure 83. Photo. Top 4-inch wrap butted against a 4-inch solid block.

5. Grade the surface aggregate of the beam seat (as necessary) to about 0.5 inch to aid in seating the superstructure and to maximize contact with the bearing area before folding the final wrap.

   For temporary GRS abutments, it may be possible to add an additional layer of reinforcement placed between the beam seat and concrete or steel beams to provide additional protection of the beam seat (figure 84). The additional layer of reinforcement may decrease the sliding resistance between the superstructure and the beam seat.

Source: FHWA.

Figure 84. Photo. Additional reinforcement under the beam.

7.8.2 Setback

The setback is the distance between the back of the facing block and the front of the beam seat. This distance can be established during construction of the beam seat and placement of the block and foam board used to form the beam seat wrap. The setback distance is usually 8 inches but can be greater.

7.8.3 Drip Edge

The optional drip edge (e.g., aluminum flashing) is installed prior to setting the bridge beams and is placed in between the bottom of the beams and the foam board. The flashing is held in place by the pressure of the beams on the compressible foam board (figure 85). The length of the flashing should extend beyond the outside edge of the bridge beams and be trimmed to fit against the parapets.

Source: FHWA.

Figure 85. Photo. Aluminum flashing (drip edge) between the beams and the top of the CMU facing.

7.8.4 CIP or Precast Footing

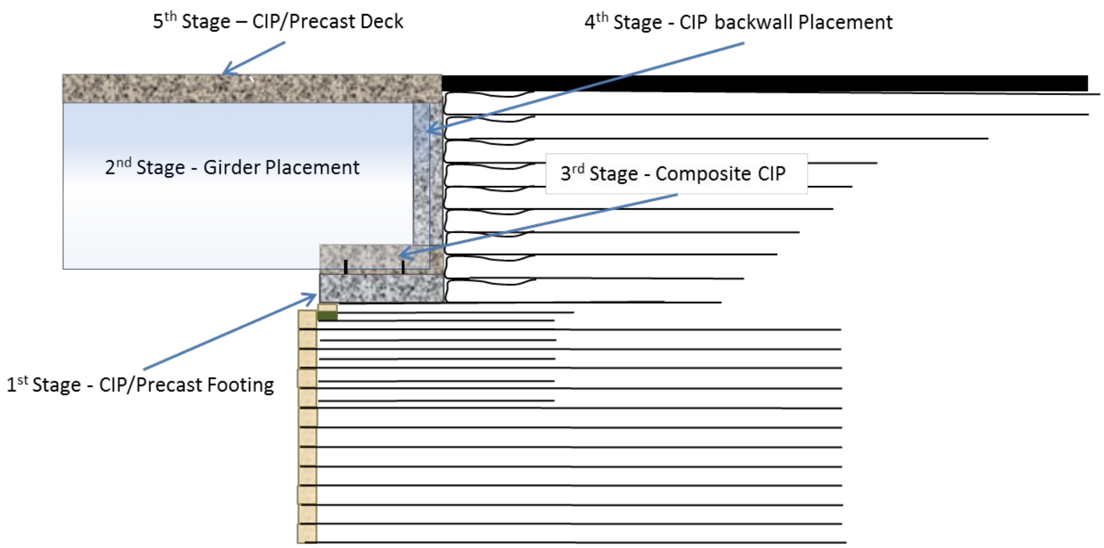
For GRS-IBS built without adjacent concrete beams, a CIP or precast footing may be necessary, as with steel beams or spread girders (figure 86 and figure 87). Figure 88 illustrates a simple method to create a composite bridge superstructure with multiple steel girders. The result forms a semi-integral type abutment. The final stage in the illustration (stage 5) is the placement of the deck to complete the composite bridge.

Source: FHWA.

Figure 86. Photo. Steel girder on CIP footing.

Source: FHWA.

Figure 87. Photo. CIP footings for steel girders.



Source: FHWA.

Figure 88. Illustration. Details to cast steel girders and backwall on CIP footing.

7.9 PLACEMENT OF SUPERSTRUCTURE

Prepare the beam seat as described in section 7.8.1. The grade of the beam seat will control the final elevation of the bridge.

7.9.1 Crane Position on GRS Mass

The crane used for placement of the superstructure can be positioned on the GRS abutment provided the outrigger pads are sized less than the factored bearing resistance of the GRS mass. The outrigger pads should be sized for 4,000 lb/ft^2 near the face of the abutment wall, with greater loads able to be supported with increasing distance from the abutment face (figure 89).

Source: FHWA.

Figure 89. Photo. Outrigger pads near the wall face.

7.9.2 Superstructure Placement on the Beam Seat (Without CIP Footing)

Since the bearing surface is aggregate under a layer of geosynthetic reinforcement, it is important to set beams square and level. They should never be dragged over the beam seat surface, which could create the potential for an uneven bearing area or a void under the beam, producing uneven bearing stresses between bridge elements.

7.9.3 Wing Walls and Parapets

Wing walls and parapets are constructed after the superstructure is set. The CMU block in the parapet wall should be trimmed or saw cut for a custom fit against the beam edge of the superstructure to prevent the loss of fill material. Figure 90 and figure 91 show the construction of the parapet against the superstructure. If the gap between the superstructure and the facing block is difficult to fill using thin slices of cut facing block, a mortar mix or other material should be used to close the gap.

Source: FHWA.

Figure 90. Photo. First view of parapet and wing wall construction.

Source: FHWA.

Figure 91. Photo. Second view of parapet and wing wall construction.

7.10 APPROACH INTEGRATION

A properly constructed integrated approach that transitions the road to the bridge is essential for minimizing settlement in front of the bridge beams and mitigating the bump at the end of the bridge. This is accomplished by compacting and reinforcing the approach fill with wrapped geotextile layers. The material for the integrated approach zone should be well-graded, as outlined in chapter 3.

Once the superstructure is in place, the approach to the bridge can be constructed using the following steps:

1. Trim a geotextile reinforcement sheet to provide the planned length after it is wrapped, and place it behind the beam end (figure 92). The width of the sheet should allow for wrapping of the sides after the fill layer is placed and compacted. Wrapping of the sides prevents lateral migration of the fill.

Source: FHWA.

Figure 92. Photo. Reinforcement placement.

2. Place a 6-inch-thick lift of fill and compact per compaction specifications for road base (figure 93). Add a secondary layer of reinforcement on top of the 6-inch-thick lift and then place another 6-inch-thick lift of fill and compact (figure 94). Fold back the reinforcement sheet to wrap the compacted fill layer and smooth wrinkles (figure 95).

Source: FHWA.

Figure 93. Photo. First 6-inch-thick fill lift.

Source: FHWA.

Figure 94. Photo. Secondary reinforcement sheet.

Source: FHWA.

Figure 95. Photo. Completed wrapped approach layer.

3. Repeat these steps until the integrated approach is approximately 2 inches from the top of the beam grade, as shown in figure 96.

Source: FHWA.

Figure 96. Photo. Second 6-inch-thick fill lift.

Multiple sheets can be used along the width of the approach as long as all seams are kept perpendicular to the beam ends. The typical wrap reinforcement spacing is 12 inches, with intermediate layers spaced at 6 inches and compacted in 6-inch-thick lifts. However, in the case of beams with a reduced depth, the spacing of the wrapped layers may need to be reduced, and the intermediate layers may need to be eliminated. At a minimum, the top two reinforcement layers of the integrated approach should extend 3 ft over the cut slope to blend the roadway on to the GRS abutment. The top wrap fold should increase in length with each successive wrapped layer until the fill is 2 inches below the bridge grade. It is important to ensure that the backfill used is specified to limit the amount of fines in the integrated approach to prevent frost heave.

7.10.1 Wrapped Reinforcement Layers on Sides

If lateral spreading of the fill in the integrated approach will be an issue (e.g., wing walls are not sufficient to confine the fill at the sides), the reinforcement sheets comprising the wrapped layers should be folded over along the sides and perpendicular to the bridge (figure 97).

Source: FHWA.

Figure 97. Photo. Completed approach fill.

7.10.2 Preloading

In some situations, it might be beneficial to preload the abutment before paving to minimize post-construction deformation or settlement within the GRS abutment. A simple method of preloading can be achieved by parking fully loaded trucks on the bridge for several days before placing the asphalt pavement.

7.10.3 Paving

The top layer of reinforcement should be kept approximately 2 inches below the beam grade. This will allow a layer of aggregate cover to be placed to protect the reinforcement from contact with hot mix asphalt.

When IBS is finished with asphalt mix overlay, a layer of paving fabric or waterproof membrane should be extended over the beams onto the approach way (see figure 15). Extending the paving fabric 3 ft over the beam approach interface is recommended to bridge the gap and provide an interface to accommodate thermal movement, minimize surface water infiltration, and prevent cracks in the road. Note that paving fabric is already used on top of the beams as a barrier to water infiltration and to absorb stresses to minimize reflective and fatigue cracking of the new asphalt surface layer. When the superstructure has a nonasphalt wearing surface, a control joint should be detailed to tie the bridge surface with the approach roadway material (figure 98).

Source: FHWA.

Figure 98. Photo. Control joint between the concrete deck and asphalt pavement.

7.10.4 Guardrail Posts

Nondisplacement steel H posts are recommended for any railing that is driven through the reinforcement (figure 99). It is also possible to drill through the GRS with an auger to set other types of posts; both methods are acceptable. Depending on the jurisdiction, some guardrail post installation occurs after paving by augering through the asphalt and into the reinforced fill. After the posts are set, the holes are filled and recompacted, and an asphalt patch is placed in the area around the post.

Source: FHWA.

Figure 99. Photo. Guardrail posts.

7.11 SITE DRAINAGE

The GRS-IBS construction area should be protected from surface runoff during the project. Critical areas are behind the abutment wall at the interface between the GRS abutment and the retained fill, at the base of the abutment, and at any location where a fill slope meets the wall face. The design needs to include provisions for surface drainage along the fill slope adjacent to the wing walls. Provisions for drainage should also be included at the boundary of the wing walls and the fill slope. Long walls built along variable elevation or the abutment wing walls are often stepped to reduce excavation. In these situations, the termination of wall steps should be sufficiently embedded to prevent problems with erosion. The drainage swell or channel should be separated from the wall to avoid flow directly against the wall face.

Site preparation for drainage should include the following:

- **Grading**: The site should be graded to drain away from the GRS every night in anticipation of precipitation to avoid saturation of soil.

- **Diversion trenches**: An alternative to grading is placing diversion trenches around the perimeter to divert water.

- **Compaction of loose soil:** Any loose soil placed to construct GRS should be graded and compacted before stoppage of work for the day. Also, onsite stockpiles of fill material containing fines should be protected from excess precipitation.

7.12 UTILITIES

All utilities that pass through a GRS abutment should follow local, State, and Federal utility codes. With GRS, utilities can be placed in the reinforced zone, passing either perpendicular or parallel through the GRS fill (figure 100). Reinforcement can be trimmed to accommodate pipes and casing, and extra reinforcement sheets can be added to replace cut out sections. Waterlines should be installed with a sleeve pipe in the abutment to prevent any erosion or loss of material should there be a break (figure 101).

Source: FHWA.

Figure 100. Photo. Utilities through a GRS abutment.

Copyright: Anderson County, SC.

Figure 101. Photo. Waterline through a GRS abutment.

Some items to consider related to utility construction include the following:

- **Wall stability**: Waterlines within a GRS abutment should be contained in a sleeve pipe (see figure 101) so that in the event the waterline breaks with the abutment, the unleached water exits the wall without saturating the wall face.

- **Utility ports**: Pass-through portals should be detailed and constructed for fit against the wall face to prevent the loss of backfill material. Utility ports should also be designed to accommodate any differential movement.

- **Repair access**: Utilities passing through an abutment should be laid out for somewhat easy access in the event of repair or maintenance. This consideration should include not only the abutment but also traffic.

- **Attachments and connections to the wall face**: Hanging utilities on an abutment wall face are permitted, provided the connections are compatible with the facing type. Additionally, connections should be designed to accommodate lateral and vertical movement associated with substructure–superstructure interaction.

CHAPTER 8. QC AND QA

8.1 INTRODUCTION

Quality is the responsibility of anybody who is involved in the project. Builders must ensure QC by overseeing the implementation, measurement, and enforcement of sound construction practices and field inspection procedures to ensure construction quality, as outlined in this manual. QC also involves the selection of quality materials. The successful completion of a project is dependent on a proper monitoring program with necessary adjustments at each stage of construction. In addition to QC, QA can either be the responsibility of the owner agency or a third-party agency. QA is necessary to ensure that the finished product meets specifications through inspection, testing, and final acceptance. The process involves constant evaluation of the project activities related to planning, design, development of plans and specifications, and construction as well as all interactions associated with these fundamental activities.

8.2 ROLE OF THE CONTRACTOR

Since GRS is a nonproprietary generic wall system, the contractor building the wall may be responsible for developing and maintaining a QA/QC plan for project quality. Prequalification based on the procedures outlined in this manual should be a necessary requirement for this type of construction.

8.3 QC TESTING

QC testing performed during construction mainly applies to onsite field testing of backfill material and associated laboratory tests.

8.3.1 Laboratory Testing

Gradation and moisture-density tests (e.g., Proctor compaction test) are required for field monitoring of well-graded backfill material. The classification tests and moisture-density tests should follow appropriate standards for aggregate sampling and testing.

Large-scale direct shear tests are the most efficient method for determining the friction angle of the coarse-grained backfill aggregates specified in GRS-IBS construction. These methods of testing are preferred over the standard direct shear test or smaller diameter triaxial tests that are performed on the minus No. 10 material.

8.3.2 Field Testing

Fill placement and compaction is the predominant construction activity that needs to be monitored in a GRS-IBS project. Field density tests should be performed on each layer. The field test method should be applicable to the aggregate type that is used for the backfill material.

Well-graded backfill can be tested with a nuclear gauge. State transportation departments' procedures for density testing should be used. Moisture content should be monitored and controlled prior to fill placement for an effective compaction process.

A procedural (method-based) specification is preferable for the compaction of open-graded fill material, which exhibits a high percentage of void space. Open-graded aggregates are not conducive to in-place nuclear density testing procedures, as the direct transmission nuclear gauge procedure is difficult to perform (the transmission hole will typically not stay open), and nuclear backscatter testing is not effective due to poor soil/gauge contact.

In lieu of density testing for open-graded gravels, maximum density can be achieved with a recommended method or procedural specification. The procedure can specify three to five passes with a walk-behind vibratory plate compactor near the wall face. Larger ride-on vibratory rollers with greater frequency and efficiency can be used in the core of the GRS abutment with fewer passes 3 ft from the wall face.

8.4 CONSTRUCTION INSPECTION

Thorough inspection before and during construction will ensure the GRS structure is built in accordance to the plans and guidelines. Inspection requires an understanding of GRS design and methodology. Familiarity and understanding of the design plans is necessary. It is important to have firsthand knowledge of the GRS construction processes. A properly implemented field inspection program provides an opportunity to take corrective action during the construction process.

A critical component of construction is compaction behind the facing element followed by placement of the geosynthetic reinforcement. Those responsible for performing these construction activities are best suited for maintaining the quality of each GRS wall layer. Note that in the RSF and the integrated approach, a geotextile must be used to prevent migration of fill material and erosion.

8.4.1 Materials

Once materials are delivered to the site, they should be inspected for compliance with the guidelines and project specifications. Materials should be visually inspected for quality, damage, and defects as follows:

- **Backfill**: In addition to the quarry material certificate showing the gradation of the aggregate, a visual inspection should be performed to verify maximum grain size, amount of fines, grain shape (angular or rounded), excess fines, moisture content, and durability.

- **Geosynthetic reinforcement**: It should be verified that the specified type and strength of geosynthetic is correct along with the required roll dimensions. Chapter 3 provides detailed tests that should be documented for each roll of reinforcement.

- **Facing block**: As outlined in chapter 3, the facing block should be inspected for integrity, consistency, and dimension tolerances. It should be confirmed that sufficient quantities and proper block type (e.g., solid block, corners, and face block) are present onsite and ready for use.

- **Riprap (when needed)**: Without comprehensive construction acceptance testing, there is little assurance that the riprap will perform as intended. Consequently, when a riprap countermeasure is used, rock quality, acceptance criteria, and testing frequency requirements must be developed and included in the construction contract specifications to properly control the manufacture, placement, and acceptance of the riprap. In addition, the size and gradation test methods to be used for accepting the riprap mass must be included in, or referenced by, the contract. Including such provisions in the contract will reduce the chances of premature riprap failure. The FHWA Office of Federal Lands Highways *Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects* provides sampling, testing, and acceptance requirements along with material requirements for rock riprap.(17)

After construction, riprap countermeasure condition and channel instability should be assessed during each regular bridge inspection and after large flood events. Any significant countermeasure failure or change in channel stability should be noted and scheduled for repair or stabilization. Without proper inspection and maintenance, a scour countermeasure may fail or a channel may become unstable, which can lead to abutment failure.

8.4.2 Equipment

Compaction of the backfill in a GRS wall or abutment is a critical construction activity. It should be confirmed that the compaction equipment onsite is compatible with the selected backfill material. It should also be verified that the required hand tools are onsite for spreading and grading aggregate, maintaining the facing alignment, and sweeping the top of the CMU facing block.

8.4.3 Project Layout

It should be verified that all layout reference points are established, with particular emphasis on the location of the following areas:

- Center line of superstructure.
- RSF area within lines and grade of working drawings.
- Bearing area of the bridge beams.
- Wing wall width and length.
- Clear space and setback.
- Span length.
- Center of bearing to center of bearing.
- Elevations.
- Grades.

8.4.4 Construction Activities

GRS is built from the bottom to the top. Those responsible for inspection need to make certain that each layer is constructed and tested in accordance with the contract drawings and specifications before proceeding with subsequent layers. Inspection and QC/QA activities include the following:

- **Working bench**: Before excavation, the working bench/platform needs to be inspected for stability with consideration for drainage. Any movement should be controlled.

- **Foundation excavation**: The foundation should be cut, as outlined in the project plan, and inspected for any soft areas before compaction and proof rolling.

- **Geotextile-wrapped RSF foundation**: For encapsulation of the RSF, it should be confirmed that the open edge of any overlap is facing downstream and that the three sides (i.e., two wing walls and one abutment face wall) are contained by a layer of geotextile to prevent erosion.

- **Leveling course**: To set the first course of facing block level and plumb, the top elevation of the RSF should be as close to grade as possible. Often, a thin (0.5-inch) leveling layer of sand is placed under the first course. Inspection of this leveling layer should be performed to determine its thickness or the need to replace it with a low-slump wet concrete/grout mix.

- **Compaction of backfill:** Inspection of backfill operations should verify compliance with the construction guidelines outlined in chapter 7. Compaction behind the wall face and within the bearing area is important. Inspection should confirm that each lift never exceeds the specified thickness.

  Compaction control should be maintained through field density tests or other soil stiffness-based methods. For backfill material containing fines (minus No. 200), the moisture content should be within the specified range (±2 percent). This improves the compaction process. Compaction of open-graded aggregate should be observed to ensure nonmovement of material under the compaction equipment; this observation is an indication of compaction or stiffness and is dictated by the number of passes.

- **Reinforcement installation**: The installation of each reinforcement layer should be inspected to ensure it is properly placed, has adequate facing element coverage for the frictional connection, and is free of wrinkles. The location and placement of the bearing bed reinforcement layers, particularly in situations when the bridge is superelevated, should be anticipated.

- **Facing block placement**: Prior to placement of the reinforcement layer over the facing block, the block should be inspected to verify a clean surface. This is essential in maintaining wall alignment and avoiding block cracking due to point loads. The inspection process should ensure that there is no rocking motion when setting the block, which can be indicative of point load bearing.

- **Wall alignment**: Visual inspection should be performed at regular intervals during construction. This will help ensure that the wall is within vertical and horizontal tolerances for alignment. QC should be performed on the block alignment using a string line on at least every third course. Vertical alignment can be checked with a plumb bob.

- **Wall termination**: All wing wall terminations should be sufficiently embedded to prevent undermining from erosion. A terminated course needs to be founded on a stable compacted layer of granular fill material, as outlined in chapter 7, or on an excavated cut into native soils.

- **Fill slope side**: The fill slope at the wing walls is usually built with native soil fill following embankment construction standards, with fill placed in compacted lifts not exceeding 12 inches in thickness, as the GRS wall advances upward. The fill slope should be constructed with a drain path that leads away from the wall face. Surface runoff should be diverted to prevent saturation of the soil fill slope. Temporary drainage may need to be installed to preserve the integrity of the cut slope.

- **Site drainage**: The working platform should be compacted and graded to drain surface water away from the working area. Any pit excavation should be sloped to drain to a location that can be pumped.

- **Heavy equipment operation**: It is beneficial to have construction equipment centrally located in the work area and materials strategically stocked near the equipment for efficient transfer to the labor crew. Equipment operators should take caution when working near large layers of exposed geosynthetic to avoid any actions that may tear or pull on the reinforcement.

- **Beam seat**: Construction of the beam seat should be inspected to confirm the use of methods described in chapter 7. It is important to verify that the beam seat is constructed at the correct elevation and grade to provide the specified clear space and setback.

- **CMU core grouting**: The core of the top three courses of CMU blocks should be filled with concrete wall mix. Rebar dowels should be cut to length (20 inches) and inserted into the core of the top three courses. The concrete mix should be rodded with a rebar dowel before insertion to eliminate voids. Sufficient concrete should be available to form the coping cap during the same pour.

- **Wrapped integrated approach**: Prior to placement of the geotextile reinforcement, it should be verified that the length of the reinforcement is adequate to wrap the fill and extend back toward the road as shown in the one-sheet plan. Sufficient reinforcement width should also be available to laterally confine the approach fill if necessary.

  The thickness for each lift should be checked to ensure that it does not exceed the maximum thickness and that secondary reinforcement is placed within fill layers that are greater than 6 inches. If a granular road base is used in the wrapped approach, it should be verified that its compaction conforms to density requirements for the road as well as the GRS.

  At the top of the integrated approach, it should be verified that a 1- to 2-inch layer of aggregate is placed on the top reinforcement layer for protection from hot mix asphalt. It should be verified that the paving fabric, if used, bridges the interface from the deck to the approach as described in chapter 7.

8.5 DOCUMENTATION

The following subsections describe common documentation for construction of GRS abutments and IBSs.

8.5.1 Compliance Documentation

Field test results should be carefully measured and archived as a permanent part of the job record. This information can also be used to modify field (construction and inspection) practices.

The main field measurement—moisture density tests–should be documented during construction. Other documentation should include construction modifications, field changes, and daily construction reports.

8.5.2 Record Drawings

As-built plans should be prepared and provided to the owner upon completion of the project.

8.6 CONTRACTING METHODS

Of the two types of contracting methods commonly used for specialty construction (i.e., the procedural method and performance method), the preferred approach for the GRS-IBS is the procedural method. GRS performance-based methods can be developed when the technique becomes more widespread. The generic nature of GRS walls and abutments fits well with the performance-based method, which can advance the technology by creating an opportunity to develop new techniques, details, and equipment.

8.6.1 Performance Method

In a performance-based contract, the contractor can choose a GRS system based on its performance and constructability. The contractor should verify that the GRS-IBS is constructible and performs as outlined in the requirements. Careful attention should be placed on the compatibility between the backfill material, the wall facing, and the reinforcement to ensure that the wall meets the necessary requirements.

Under this method, design and performance criteria should be based on the data provided in chapters 4 and 6. Material and construction specifications can be based on the information in chapters 3 and 7. This contract method requires that the reviewers have considerable knowledge in GRS technology to accept design submittals.

8.6.2 Procedural Method

In this contract method, the agency or owner should provide a detailed set of design plans and construction specifications in the bid document. QA begins with an initial plan, design, and review of construction materials. Approval should be dependent on someone experienced in the design and construction of the GRS system. Also, the completed project should comply with local agency building codes and regulations.

Fully detailed plans and items requiring review prior to initiating a GRS project should consist of the following:

- Design calculations.
- Loads.
- Stability analysis.
- Hydraulic analysis.
- Project drawings.
- Plan drawings.
- Cross sectional drawings of all abutment wall faces and wing walls.
- Elevation drawings.
- Horizontal and vertical curve details.
- Construction details addressing guardrails, parapets, beam seat, wing wall configurations, etc.
- General notes.
- Fabric schedule.
- Block schedule.
- One-sheet plan for quick reference.
- Geotechnical report.
- Plan view of testing.
- Subsurface profile (if necessary).
- Test boring logs (if necessary).
- Laboratory test data.
- Engineering properties of foundation soil, retained soil, and settlement analysis.
- Allowable/ultimate bearing pressure of foundation soil.
- Ground water and free water conditions.

- Existing abutment conditions (if replacement bridge).
- Historical flood events.
- Hydraulic report.
- Appropriate flows and velocities.
- Stable particle size analysis for scour potential.
- Land uses that could impact flood levels.

In addition, verification of experience (prequalification) for the contractor should be certified or prequalified in GRS construction methods in accordance with the procedures presented in this manual. As an alternative, a contractor should be able to verify demonstrated knowledge in constructing GRS structures. For GRS-IBS projects, the contractor should provide information on the successful construction of many GRS walls and abutments. It is important that the contractor understand the compatible relationship between wall facings and backfill material.

A QC/QA plan should also be developed by the agency or contractor performing the work, and it should be followed by the contractor. The plan should detail types of measurements and documentation that will be maintained during construction to ensure compliance with GRS guidelines and standards.

8.6.3 Contractor Submittals

Materials used to construct GRS-IBS are readily available from a number of sources. The only requirement is that they meet the standards provided in this manual. The main materials that should be reviewed prior to construction are as follows (see chapter 3 for more details):

- CMU block specifications or other type of facing element.
- Backfill gradation, type, and source. The backfill submittal should include aggregates used for the RSF, the abutment wall, and the integrated approach.
- Geosynthetic reinforcement.

8.7 PROJECT DRAWINGS AND DOCUMENTS

Typical GRS-IBS working drawings include a one-page plan sheet, estimated quantities, general notes, project plans, profiles, GRS abutment details, and a site plan.[1,2]

CHAPTER 9. IN-SERVICE INSPECTION, MAINTENANCE, AND REPAIR

9.1 INTRODUCTION

A key feature of a GRS-IBS is that it has fewer parts than conventional bridges and abutments and should therefore need less maintenance. Like other bridges, the main components are the superstructure and the substructure. The superstructure is the same as a conventional bridge and should have the same protocol for inspection, rating, maintenance, and repair. IBS is also somewhat similar to an integral abutment in how the ends are embedded in the approach. A difference is that the IBS is embedded in compacted reinforced aggregate, whereas an integral abutment is encased in concrete. Both bridges are designed without a joint to limit the effect of water at the beam ends for improved durability.

At the time that this manual was written, over 300 GRS-IBS bridges have been built for local or off-system service, with the oldest built in 2005.(2) None of the bridges show any signs of distress. All indications are that GRS-IBS works well, suggesting that the long-term performance of this system is adequate. In addition, IBS has fewer components and is designed for a smooth transition, thereby reducing impact loads (a major contributor to fatigue of the superstructure). This promise of improved performance, however, does not mean that the bridge system is immune to the common problems of conventional systems. This chapter focuses on potential requirements unique to this IBS and other components associated with the integrated approach.

9.2 IN-SERVICE INSPECTION

Both superstructure and substructure elements should be included as part of the visual inspection process. As previously indicated, the superstructure is similar to a conventional bridge and therefore has a similar procedure for inspection. The following elements should be included as part of the inspection of the IBS:

- **Pavement**: If the bridge has asphalt pavement, check for a transverse crack, shoving, or separation at the approach end wall interface.

- **Approach**: Check the approaches for vehicle rideability, smoothness, and pavement cracks.

- **Parapet walls**: Check the interface between the beams and the parapet wall for separation or shifting.

- **Beam ends**: Check embedded beam ends for corrosion (i.e., rust stains) at the beam bases.

- **Scour**: Monitor GRS abutments built adjacent to a water channel for scour. Riprap or other appropriate countermeasures should be monitored at each bridge inspection or after an extreme flood event. Any movement of rock should be noted and repaired to prevent scour from progressing and endangering the RSF or the abutment. No problems have been noted in installations of properly designed GRS abutments even after sequential flooding events. An indicator of scour on an abutment face or wing wall can be achieved by using colored blocks on the bottom five to eight rows. Solid blocks are recommended at the bottom, as they are more likely to resist any impact of moving riprap, ice, or other abrasion associated with the normal water elevation. The colored solid blocks are also covered from view by the initial riprap, and any exposure of colored block during inspection serves as a visual check for movement or undermining of the riprap, indicating a need for remediation or repair to protect the RSF and abutment from scour.

- **Drainage**: All GRS structures should include consideration for surface drainage. Check the critical drainage paths where the fill slope meets the wing walls leading to the base of the wall. It is imperative that the wing walls have sufficient embedment to prevent erosion due to roadway runoff.

- **Wall face cap**: Inspect the coping for cracks.

- **Modular blocks**: Check for the following for GRS walls built with modular facing blocks:

  - Cracked blocks.

  - Separated blocks.

  - Block durability problems. (More information about block durability due to freeze thaw, spalling, and efflorescence is available.)[54]

- **Guardrail**: Inspect traffic barriers for damage.

- **Wall face**: Inspect wall faces for excessive lateral movement or settlement as follows:

  - Lateral deformation can be checked visually or with a plumb bob referenced from known points from the top of the wall to the bottom.

  - Visual inspection for wall settlement can be achieved by checking for distortion between the horizontal courses of block.

- **Clear space**: Inspect, measure, and record the distance from the top of the wall face to the base of the superstructure beam for any settlement within the GRS abutment mass. A clear space must be maintained throughout the life of the bridge to prevent loading on the facing elements.

- **Drip edge**: Inspect any drip edge detail at the top of the wall beneath the beam for water diversion (if installed).

- **Burrows**: Inspect and remove any animal burrows adjacent to the walls.

- **Utility lines**: Check alignment and hanging brackets of utilities to ensure compatibility and serviceability of both the utility and the GRS abutment.

9.3 MAINTENANCE

If properly designed and constructed, GRS-IBS should need minimal maintenance because it has fewer parts than a traditional bridge (e.g., no approach slab, sleeper slab, CIP parapet walls, bridge bearings, or joint details). Since the bridge superstructure is built with common materials, general maintenance should be similar to that of a conventional bridge system. Maintenance duties might include the following:

- Sealing of a pavement crack(s), particularly one(s) forming at the beam approach interface.
- Stabilizing drainage ditches to prevent erosion along the wing wall.
- Removing vegetation growth from the wall face unless it is part of the design.
- Sealing any gaps in the facing large enough to allow for a loss of fill.
- Repairing lateral spreading of approach fill (figure 102).

Source: FHWA.

Figure 102. Photo. Lateral spreading and erosion of approach fill.

There are various methods for sealing a pavement crack. The method that is selected should be based on the width of the crack. The technique typically involves routing or sawing the crack, cleaning and drying it, and applying a sealer or filler to it. Figure 103 illustrates an example of repair with the removal and replacement of the section of damaged asphalt in the interface zone. After replacement of the asphalt, a V-notch is cut in the asphalt interface between the end of the beams and the integrated approach. The cut is cleaned and filled with suitable filler.

Copyright: St. Lawrence County Department of Highways.

Figure 103. Photo. Approach repair by asphalt removal and filling.

9.4 WALL FACE REPAIR

This section includes tips and suggested repair methods in the event of damage to the GRS abutment wall face. Damage can occur as a result of impact or poor wall face durability. Since a GRS abutment is internally supported, the face is not considered a structural element; however, its integrity is important to ensuring long-term performance of the GRS abutment.

The following list provides repair procedures for potential problems:

- **Damage to a few hollow-core CMU blocks within the face of the wall**: The face of the damaged block(s) should be chipped out and replaced with the face of another block. The face piece should be cut slightly smaller and secured with mortar. As an alternative to replacing the concrete face, repairs can be made by forming and placing concrete in the damaged area. Concrete or cement grout can be used along with rebar for improved strength. This type of repair can be done in stages if multiple block layers are damaged. Figure 104 shows the repair of chipped facing block with matched colored mortar. Figure 105 shows a colored mortar patch used to fill the gap between the parapet wall and the bridge beams.

Source: FHWA.

Figure 104. Photo. Repair of damaged block with colored mortar.

Source: FHWA.

Figure 105. Photo. Colored mortar patch used to fill the gap between the parapet wall and bridge beams.

- **Deteriorating facing blocks or scour damage:** Although there is no case history for this, shotcrete can be used to repair the face of a modular block wall. Figure 106 shows a GRS wall built with CMU blocks being used to repair a failed MSE wall, and figure 107 shows the same CMU covered with shotcrete. Note that drains were installed at the base of the wall to facilitate the flow of water from the GRS abutment. In some situations, it might be necessary to install vertical strip drains in the face of the GRS wall before applying the shotcrete.

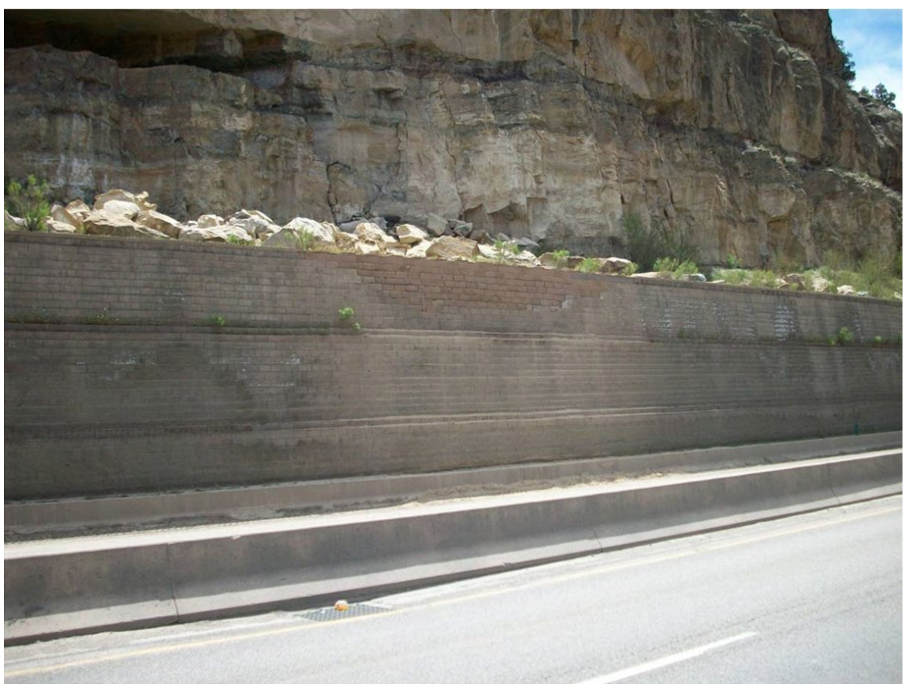
Figure 106. Photo. Use of GRS wall to repair damaged MSE wall.

Figure 107. Photo. CMU GRS wall with a shotcrete face.

- Figure 108 and figure 109 show a GRS wall before and after the repair of a rock-fall impact. The repaired section is set slightly back from the original wall alignment. To repair this wall, the boulder was removed, and each soil layer within the damaged zone was excavated. To access the fill, the fabric layers were cut perpendicular to the face and peeled back enough to access all the reinforcement layers within the damaged zone. This process was repeated until the damaged zone was exposed. The exposed zone was rebuilt using the methods explained in chapter 7 one layer at a time from the bottom up. In areas where the reinforcement was excessively damaged, new reinforcement was spliced in to reestablish the frictional connection. The top courses were then pinned and grouted.

Copyright: Colorado Department of Transportation.

Figure 108. Photo. GRS wall damaged by a large sandstone boulder.



Copyright: Colorado Department of Transportation.

Figure 109. Photo. Repair of a GRS wall after damage caused by rock fall.

- **Excessive settlement of the beam seat**: While this has never been observed, it is possible that the superstructure could experience excessive movement due to compression of either the GRS abutment or foundation soils or due to external instability. If the clear space is lost and the superstructure is causing distress to the wall, it is possible to saw a new gap to relieve the pressure on the facing. An alternative method would be to pressure grout and elevate the superstructure back to its original grade, which may also require repair to the approach pavement.

APPENDIX A. IN-SERVICE GRADATIONS

The counties and agencies that are building GRS-IBS have often selected locally available structural backfills for their projects. The gradations of various backfill types used for in-service projects are shown in table 10 through table 16.

Table 10. AASHTO No. 89 clean, crushed limestone.(12)

U.S. Sieve Size	Percent Passing
0.5 inch	100
0.375 inch	90–100
No. 4	20–55
No. 8	5–30
No. 16	0–10
No. 50	0–5

Table 11. AASHTO No. 67 clean, crushed rock.(12)

U.S. Sieve Size	Percent Passing
1 inch	100
0.75 inch	90–100
0.375 inch	20–55
No. 4	0–10
No. 8	0–5

Table 12. Washington State Department of Transportation 1.25-inch minus gravel with clean round rock and sand mixture-pit run.(55)

U.S. Sieve Size	Percent Passing
1.25 inch	100
1 inch	90–100
No. 4	50–80
No. 40	0–30
No. 200	0–7

Table 13. New York State Department of Transportation No. 1 clean, crushed rock.(56)

U.S. Sieve Size	Percent Passing
1 inch	100
0.5 inch	90–100
0.25 inch	0–15
No. 200	0–1

Table 14. Illinois Department of Transportation CA6 road base subrounded gravel with sand mix.[57]

U.S. Sieve Size	Percent Passing
1.25 inch	100
1 inch	90–100
0.5 inch	60–90
No. 4	30–56
No. 16	10–40
No. 200	4–12

Table 15. Virginia Department of Transportation 21A crushed diabase.[58]

U.S. Sieve Size	Percent Passing
2 inches	100
1 inch	94–100
0.375 inch	63–72
No. 10	32–41
No. 40	14–24
No. 200	6–12

Table 16. AASHTO No. 8 clean, crushed rock.[12]

U.S. Sieve Size	Percent Passing
0.5 inch	100
0.375 inch	85–100
No. 4	10–30
No. 8	0–10
No. 16	0–5

APPENDIX B. ASD DESIGN PROCEDURE

B.1 INTRODUCTION

There are nine basic steps in the design of GRS-IBS (see figure 16 in chapter 4). Note that the design philosophy illustrated in this appendix is ASD. It is FHWA policy that design for all Federal-aid funded projects be conducted using the AASHTO LRFD methodology.(59) Guidelines to design GRS-IBS in an LRFD format are presented in chapter 4. Similar results will be produced between the two design methods.

B.2 GRS-IBS DESIGN GUIDELINES

The nine basic design steps for GRS-IBS are the same whether using ASD or LRFD. The following sections provide additional information on each design step in the ASD format:

1. Establish project requirements (section 4.3.1).

2. Perform a site evaluation (section 4.3.2).

3. Evaluate project feasibility (section 4.3.3).

4. Determine GRS-IBS layout (section 4.3.4).

5. Calculate loads (section 4.3.5).

6. Conduct external stability analysis (section B.2.1).

7. Conduct internal stability analysis (section B.2.2).

8. Implement design details (section 4.3.8).

9. Finalize GRS-IBS design (section 4.3.9).

As noted, the differences between the two design platforms largely lie in the external and internal stability requirements (steps 6 and 7, respectively). In this appendix, only the differences in steps 6 and 7 that result from conversion to the ASD format are presented.

B.2.1 Step 6—Conduct an External Stability Analysis

The external stability of a GRS-IBS is evaluated by looking at the following potential external failure mechanisms:

- Direct sliding (figure 25).
- Bearing capacity (figure 26).
- Global stability (figure 27).

B.2.1.1 Direct Sliding

Lateral translation, or direct sliding, must be resisted for stability. For an IBS, direct sliding shall be evaluated at both the interface between the GRS abutment and RSF and between the RSF and the foundation soils. For a standalone GRS abutment, direct sliding should be evaluated at the interface between the GRS abutment and the foundation soils.

B.2.1.1.1 Direct Sliding at the Base of the GRS Abutment

The roadway LL surcharge due to traffic (q_t) on the approach pavement is assumed to act only over the retained backfill and not the reinforced soil mass; the contribution of q_t (and superstructure LL pressure (q_{LL})) on the abutment is ignored because the loads are transient and cannot be counted on as stabilizing surcharges. The bridge DL pressure (q_{DL}), however, is permanent and has a stabilization effect against direct sliding when considering an abutment. Because the integrated approach extends over the GRS abutment and the retained backfill, it acts to both stabilize and drive direct sliding. Regardless of whether an integrated approach is selected as part of the IBS or a traditional approach is selected with a standalone GRS abutment, contributions to both the driving force and to the resisting force from the approach road base must be taken into account because it is a permanent load.

The total nominal driving force behind the GRS abutment for direct sliding (F_n) for ASD is calculated in much the same way as in LRFD (i.e., factored driving force for direct sliding at the base of the GRS soil abutment (F_R)) except load factors are eliminated from each component of the thrust force (equation 55).

$$F_n = F_b + F_{rb} + F_t \tag{55}$$

Where the resultant forces behind the GRS abutment from the retained backfill (F_b), the road base (F_{rb}), and the roadway LL surcharge (F_t) are calculated using equation 9 through equation 11.

The nominal resisting force for direct sliding at the base of the GRS abutment (R_n) is calculated using equation 56. This equation is the LRFD modification of equation 13 that includes a sliding resistance factor (Φ_τ) where Φ_τ equals 1.0.[29]

$$R_n = W_t\mu \tag{56}$$

Where:

W_t = total weight (weight of the GRS plus the weight of the bridge beam plus the weight of the road base over the GRS mass only), which is determined using equation 57 (which is similar to equation 14 in the LRFD methodology except modified to remove the load factors).
μ = friction factor (see section 4.3.6.1.1).

$$W_t = W + q_{DL}b + W_{face} + q_{rb}b_{rb,t} \tag{57}$$

Where:

W = weight of the GRS abutment backfill (see equation 15).
q_{DL} = superstructure DL pressure.
b = bearing width of the bridge.
W_{face} = weight of the facing elements (see equation 8).
q_{rb} = surcharge due to structural backfill of the integrated approach (i.e., road base).
$b_{rb,t}$ = width of the traffic and road base surcharges over the GRS abutment (see figure 22).

For ASD, the factor of safety against direct sliding at the base of the GRS abutment ($FS_{slide,GRS}$) is computed according to equation 58. $FS_{slide,GRS}$ must be greater than or equal to 1.5. If not, lengthening the reinforcement at the base should be considered. Alternatively, a more complex analysis, including the full weight of the GRS abutment up to the cut slope, can be performed.

$$FS_{slide,GRS} = \frac{R_n}{F_n} \geq 1.5 \quad (58)$$

B.2.1.1.2 Direct Sliding at the Base of the RSF

Direct sliding should also be checked at the interface between the RSF and the foundation soil. The check is similar to that previously performed to evaluate sliding at the base of the GRS abutment; however, the weight of the RSF (W_{RSF}) is also included as a resisting force. The resultant driving forces behind the GRS abutment and RSF from the retained backfill ($F_{b,RSF}$), the road base ($F_{rb,RSF}$), and the roadway LL surcharge ($F_{t,RSF}$) are determined along the height of the GRS abutment and depth of the RSF (see equation 17 through equation 19). The total nominal driving force on the RSF in ASD ($F_{n,RSF}$) is calculated according to equation 59. On the other side, the resisting force for direct sliding at the base of the RSF ($R_{n,RSF}$) is calculated according to equation 60.

$$F_{n,RSF} = F_{b,RSF} + F_{rb,RSF} + F_{t,RSF} \quad (59)$$

$$R_{n,RSF} = (W_t + W_{RSF})\mu_{RSF} \quad (60)$$

Where:

W_t = Total weight on the RSF (see equation 57).
W_{RSF} = weight of the RSF (see equation 23).
μ_{RSF} = friction factor between the base of the RSF and the foundation soil (see section 4.3.6.1.2 in chapter 4).

For ASD, the factor of safety against direct sliding at the base of the RSF ($FS_{slide,RSF}$) is computed according to equation 61. $FS_{slide,RSF}$ must be greater than or equal to 1.5. If not, widening the RSF should be considered. Alternatively, a more complex analysis including the full weight of the GRS abutment up to the cut slope can be performed. Note that the passive pressures due to any material in front of the RSF is not included as a conservative measure; however, the designer may elect to calculate this resistance assuming the material will remain in place throughout the life of the GRS-IBS.

$$FS_{slide,RSF} = \frac{R_{n,RSF}}{F_{n,RSF}} \geq 1.5 \tag{61}$$

B.2.1.2 Bearing Capacity

To prevent bearing failure, the vertical pressure at the base of the RSF (or abutment, if standalone) must not exceed the allowable bearing capacity of the underlying soil foundation. In an IBS, the vertical pressure is a result of the weight of the GRS abutment, the weight of the RSF, the bridge DL, the road base load from the integrated approach, the LL on the superstructure, and the LL on the approach pavement. The nominal vertical pressure at the base of the GRS mass ($\sigma_{v,base,n}$) is calculated according to a Meyerhof-type distribution shown in equation 62.

$$\sigma_{v,base,n} = \frac{\sum V}{B_{RSF} - 2e_{B,n}} \tag{62}$$

Where:

ΣV = total vertical load (equation 63).
B_{RSF} = base width of the RSF.
$e_{B,n}$ = nominal eccentricity for bearing resistance (equation 64).

$$\sum V = W + W_{RSF} + W_{face} + q_t b_{rb,t} + q_{rb} b_{rb,t} + q_{DL} b + q_{LL} b \tag{63}$$

Where:

W = weight of the GRS abutment backfill (equation 15).
W_{RSF} = weight of the RSF (equation 23).
W_{face} = weight of the facing elements (equation 8).
q_t = roadway LL surcharge due to traffic.
q_{rb} = surcharge due to structural backfill of the integrated approach (i.e., road base).
$b_{rb,t}$ = width of the traffic and road base surcharges over the GRS abutment.
q_{DL} = superstructure DL pressure.
b = bearing width of the beam seat.
q_{LL} = bridge LL pressure.

$$e_{B,n} = \frac{\sum M_{\mathrm{D}} - \sum M_R}{\sum V} \tag{64}$$

Where:

ΣM_D = total driving moment (equation 65).
ΣM_R = total resisting moment (equation 66).

The moments are calculated about the bottom center of the width of the RSF. If $e_{B,n}$ is negative, $e_{B,n}$ should be taken equal to zero.

$$\sum M_D = F_{b,RSF}\left(\frac{H + D_{RSF}}{3}\right) + F_{t,RSF}\left(\frac{H + D_{RSF}}{2}\right) + F_{rb,RSF}\left(\frac{H + D_{RSF}}{2}\right) \tag{65}$$

Where:

$F_{b,RSF}$ = nominal lateral force behind the GRS abutment and RSF due to the retained backfill (see equation 17).
$F_{rb,RSF}$ = nominal lateral force behind the GRS abutment and RSF due to the road base surcharge (see equation 18).
$F_{t,RSF}$ = lateral force behind the GRS abutment and RSF due to LL on the roadway
(see equation 19).
H = height of the GRS abutment including the clear space.
D_{RSF} = depth of the RSF.

$$\sum M_R = \left(q_{DL}b + q_{LL}b\right)\left[\left(\frac{b}{2} + a_b\right) - \left(\frac{B_{RSF}}{2} - x_{RSF} - b_{block}\right)\right] + \left(q_t b_{rb,t} + q_{rb} b_{rb,t}\right)$$
$$\left(\frac{B_{RSF}}{2} - \frac{b_{rb}}{2}\right) + W\left(\frac{B_{RSF}}{2} - \frac{B}{2}\right) + W_{face}\left(B + \frac{b_{block}}{2} - \frac{B_{RSF}}{2}\right) \tag{66}$$

Where:

q_{DL} = superstructure DL pressure.
b = bearing width of the beam seat.
q_{LL} = LL on the superstructure.
a_b = setback distance between the back of the face and the beam seat.
B_{RSF} = width of the RSF.
x_{RSF} = length of the RSF in front of the wall face.
b_{block} = Width of the facing block, and a_b is the setback distance.
q_t = roadway LL surcharge.
$b_{rb,t}$ = width of the traffic and road base surcharges over the GRS abutment.
q_{rb} = surcharge due to structural backfill of the integrated approach (i.e., road base).
B = width of the GRS abutment.
W_{face} = weight of the facing elements.
b_{block} = width of the facing element.

The bearing capacity of the foundation (q_n) can be found using equation 67.

$$q_n = c'_f N_c + \frac{1}{2} B' \gamma'_f N_\gamma + \gamma'_f D_f N_q \tag{67}$$

Where:

c'_f = effectivecohesion of the foundation soil.
N_c, N_γ, and N_q = bearing capacity coefficients (dimensionless) (table 9).
γ'_f = effective unit weight of the foundation soil.
B' = effective foundation width (equal to $B_{RSF} - 2e_{B,R}$).
D_f = depth of the embedment.

The friction angle in table 9 should be taken as the foundation's effective friction angle (ϕ'_f). If groundwater is present, modifications to equation 67 may be necessary and are provided by AASHTO.[(29)]

The factor of safety against bearing failure ($FS_{bearing}$) is computed according to equation 68. $FS_{bearing}$ must be greater than or equal to 2.5. If not, options include increasing the width of the GRS abutment and RSF by increasing the length of the reinforcement layers, replacing the foundation soil with a more competent soil, or adding embedment depth.

$$FS_{bearing} = \frac{q_n}{\sigma_{v,base,n}} \geq 2.5 \tag{68}$$

B.2.1.3 Global Stability

Global stability is evaluated according to classical slope stability theory using either a rotational or wedge analysis. To facilitate the global stability check, it is prudent to collect accurate soil property information. Standard slope stability computer programs can then be used to assess the global and compound stability of a GRS structure. The factor of safety for global stability should equal at least 1.5.

B.2.2 Step 7—Conduct Internal Stability Analysis

Internal stability for GRS includes ensuring adequate internal vertical capacity, tolerable deformations, and required reinforcement strength.

B.2.2.1 Vertical Capacity

The vertical capacity of a GRS abutment is determined either empirically through a GRS performance test or analytically through a semi-empirical equation.

B.2.2.1.1 Empirical Method

Empirically, the results of an applicable performance test using the same geosynthetic reinforcement and compacted granular backfill as planned for the site should be used. The nominal bearing resistance of the geosynthetic reinforced soil abutment using the empirical method ($q_{n,emp}$) in this case is defined as the stress at which the GRS composite fails (i.e., cannot sustain any more loading). An example of a performance test result is shown in figure 28, and more discussion is provided in section 4.3.7.1.1.

The applied pressure on top of the GRS mass ($V_{applied}$) is equal to the sum of the vertical pressures on the bridge bearing area (equation 69). The vertical pressures of interest include the bridge DL (q_{DL}) and bridge LL (q_{LL}). The DL surcharge due to the road base (q_{rb}) and the LL surcharge (q_t) due to the approach pavement are located behind the bearing area and are therefore not included in the capacity related to the bridge superstructure.

$$V_{applied} = q_{DL} + q_{LL} \tag{69}$$

The factor of safety for internal vertical capacity using the empirical method ($FS_{capacity,emp}$) is then calculated using equation 70. It must be at least 3.5. If not, then the applied pressures must be reduced to an allowable level, or a different GRS composite can be made (i.e., by changing to a stronger backfill, a larger maximum aggregate size that still meets the material specifications, or a different reinforcement strength or layout).

$$FS_{capacity,emp} = \frac{q_{n,emp}}{V_{applied}} \geq 3.5 \tag{70}$$

B.2.2.1.2 Analytical Method

As an alternative, the load-carrying capacity of a GRS abutment can be evaluated using an analytical formula, referred to as the “soil–geosynthetic composite capacity equation” (see equation 34).[37] The factor of safety for vertical capacity using the analytical method ($FS_{capacity,an}$) is calculated using equation 71. Because the analytical method is based on the results of performance tests (i.e., the empirical method), $FS_{capacity,an}$ is the same and must be greater than or equal to 3.5. If not, then the applied pressures must be reduced to an allowable level or a different GRS composite can be made (i.e., by changing to a stronger backfill, a larger maximum aggregate size that still meets the material specifications, or a different reinforcement strength or layout).

$$FS_{capacity,an} = \frac{q_{n,an}}{V_{applied}} \geq 3.5 \tag{71}$$

B.2.2.2 Deformations

The method to estimate both vertical and lateral deformations is not dependent on the design code chosen (ASD or LRFD). Therefore, refer to section 4.3.7.2 for estimating deformations.

B.2.2.3 Required Reinforcement Strength

The required reinforcement strength must be sufficient to prevent failure of the GRS abutment and to maintain deformations within tolerable limits. To prevent failure, the required reinforcement strength (T_{req}) in the direction perpendicular to the abutment wall face can be determined analytically by equation 50. T_{req} should be calculated at each layer of reinforcement to ensure adequate strength throughout the GRS abutment; however, it is recommended that only one type of reinforcement be specified to simplify the design, avoid construction placement issues, and limit costs.

The allowable reinforcement strength (T_{allow}) is found assuming a factor of safety for the required reinforcement strength (FS_{reinf}) of 3.5 (equation 72). The specified ultimate reinforcement strength (T_f) must be equal to or greater than T_{allow}.

$$T_{allow} \leq \frac{T_{req}}{FS_{reinf}} \leq \frac{T_{req}}{3.5} \tag{72}$$

Because geosynthetic reinforcements of similar ultimate strength can have rather different load–deformation relationships depending on their material, it is important that a reinforcement strength at 2 percent strain ($T_{@\varepsilon = 2\%}$) also be specified (equation 73).

$$T_{@\varepsilon = 2\%} \leq T_{req} \tag{73}$$

If T_{req} is greater than $T_{@\varepsilon = 2\%}$ for the strong direction, a stiffer geosynthetic must be chosen, or the reinforcement spacing must be decreased. Because bridges are often in a plane strain condition, the bridge loads will not be shed to the wing walls; therefore, $T_{@\varepsilon = 2\%}$ only needs to be specified in the direction perpendicular to the abutment wall face. As a reminder, $T_{@\varepsilon = 2\%}$ is not equivalent to the load on the reinforcement under service conditions; the actual load will vary depending on the final strength and stiffness of the reinforcement specified. This check ensures that the actual load on the reinforcement is less than what is calculated to limit lateral strain.

APPENDIX C. DESIGN EXAMPLES

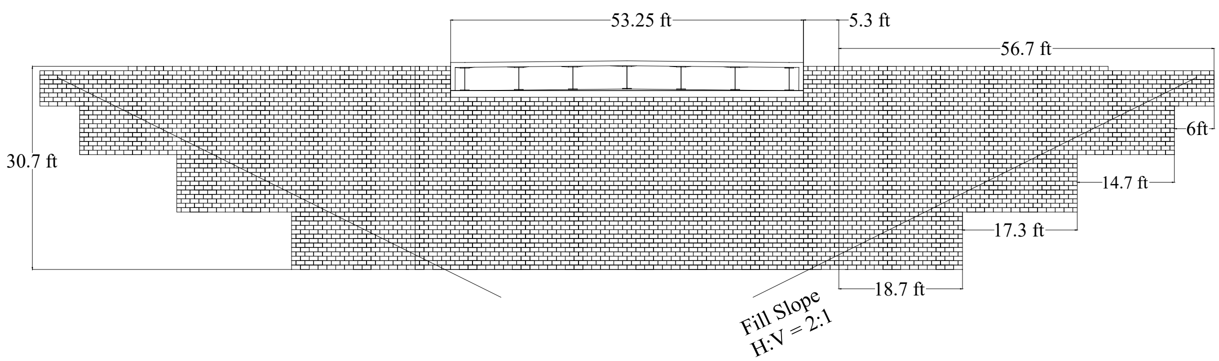
Two design examples are presented in this appendix that go step by step through the design methodology presented in chapter 4.

C.1 EXAMPLE 1—GRS-IBS RAILROAD CROSSING

Replacement of a three-span bridge that was constructed in 1936 is planned. The existing bridge is supported by two concrete abutments and two concrete piers on spread footings. The designer would like to use GRS-IBS with a steel superstructure and concrete bridge deck as a replacement. To determine the design and feasibility of the GRS-IBS, the nine steps outlined in chapter 4 were followed.

C.1.1 Step 1—Establish Project Requirements

The GRS-IBS will support a steel superstructure approximately 105 ft in total length. The bridge will have two lanes and sidewalks, with a total width of 53.3 ft. To ensure appropriate clearance for the trains passing underneath, the total height between the base of the abutment and the girders was determined to be 27 ft. Schematics of the proposed abutments are shown in figure 110. Additional project requirements are described the following the figure.



Source: FHWA.

Figure 110. Illustration. Abutment and wing wall.

Geometry criteria include the following:

- **Height of the GRS abutment (H_{abut})**: 26 ft.
- **Abutment length (L_{abut})**: 64 ft.
- **Batter angle**: 0 degrees.
- **Wall placement with respect to ground conditions**: No back slope or toe slope.
- **Skew angle**: 0 degrees.
- **Grade**: 0 percent.
- **Superelevation angle**: 0 degrees.

Performance criteria for tolerable movements include the following:

- **Vertical settlement (of the GRS abutment)**: Limited to 1 percent of the height or about 3 inches.
- **Lateral displacement**: Limited to 1.5 inch.
- **Differential settlement**: 1/200.
- **Angular distortion between abutments**: 1/200.
- **Design life:** 100 years.

Constraints include the following:

- **Environmental**: None.
- **Construction**: The abutments must not extend into a railroad's right of way.
- **Scour and stream stability**: Not applicable.

C.1.2 Step 2—Perform a Site Evaluation

The 1936 construction drawings indicate that the existing spread footings were founded on bedrock. To verify and determine the soil parameters, a site visit was conducted, where standard penetration tests (SPTs) were performed, and samples of the native soil behind the existing abutment were collected. The borings indicate that the top 28 ft are a dense sand with bedrock below. Groundwater is approximately 2 ft below the planned RSF. The results of tests conducted on the soil samples are shown in table 17 and table 18.

Table 17. Design example 1—foundation soil properties.

Parameter	Symbol	Value
Unit weight of the foundation soil (lb/ft^3)	γ_f	140
Effective unit weight of the foundation soil (lb/ft^3)	γ'_f	77.6
Friction angle of the foundation soil (degrees)	ϕ_f	38
Cohesion of the foundation soil (lb/ft^2)	c'_f	0

Table 18. Design example 1—retained backfill soil properties.

Parameter	Symbol	Value
Unit weight of the retained backfill (lb/ft^3)	γ_b	120
Friction angle of the retained backfill (degrees)	ϕ_b	34
Cohesion of the retained backfill (lb/ft^2)	c_b	0

The road base to be used in the integrated approach is a granular fill material that is brought to the site (see table 19). The select granular reinforced fill for the GRS abutment and RSF is specified as an AASHTO No. 8 stone is a select granular fill.[12] Large-scale direct shear tests were performed on a sample taken from the quarry that will supply the material. Parameters for the AASHTO No. 8 that will be used in design are shown in table 20.

Table 19. Design example 1—road base soil properties.

Parameter	Symbol	Value
Unit weight of the road base material (lb/ft^3)	γ_{rb}	120
Friction angle of the road base material (degrees)	ϕ_{rb}	40
Cohesion of the road base material (lb/ft^2)	c_{rb}	0

Table 20. Design example 1—reinforced fill properties.

Parameter	Symbol	Value
Unit weight of the reinforced backfill (lb/ft^3)	γ_r	110
Maximum grain size (inches)	d_{max}	0.5
Friction angle of the reinforced backfill (degrees)	ϕ_r	50
Cohesion of the reinforced backfill (lb/ft^2)	c_r	0

C.1.3 Step 3—Evaluate Project Feasibility

The foundation soil conditions at the site are competent with relatively shallow bedrock; therefore, shallow foundations are a good option to support the bridge. A rough cost analysis shows that the use of GRS-IBS would save approximately 50 percent compared to the traditional alternative of concrete abutments on concrete footings. Because the site is not a water crossing, scour is not a concern. The project is therefore feasible for this site.

C.1.4 Step 4—Determine Layout of GRS-IBS

Steps to determine the layout of the GRS-IBS are as follows:

1. **Define the geometry of the abutment face wall and wing walls**: The height of the abutments is 26 ft. The distance between the two abutment faces is 95 ft. The wing walls extend back 57.6 ft from the face of the abutment (see figure 110).

2. **Lay out the abutment with respect to the superstructure, including any skew, superelevation, or grade requirements**: Because the span length is greater than 25 ft, the minimum bearing width for the superstructure would be 2.5 ft. For this project, the target service pressure is 4,000 lb/ft^2 for the combined LL and DL of the bridge. Based on values submitted by the bridge engineer, the total DL of the bridge is 1,003,365 lb, and the total bridge LL (based on HL-93 loading) is estimated at 650,000 lb. Each abutment must therefore support a service load of about 826,682 lb (half of the combined load). Given the estimated footing width of 53.3 ft, the minimum required bearing width for this project is 3.9 ft; however, for constructability and anticipated long-term use of the superstructure under higher traffic loads, b = 5 ft is selected. The superstructure consists of steel girders; therefore, a footing must be designed to support the girders and evenly distribute the pressure. As mentioned previously, the footing dimensions are 5 by 53.3 ft. The skew is 0 degrees with no superelevation or grade.

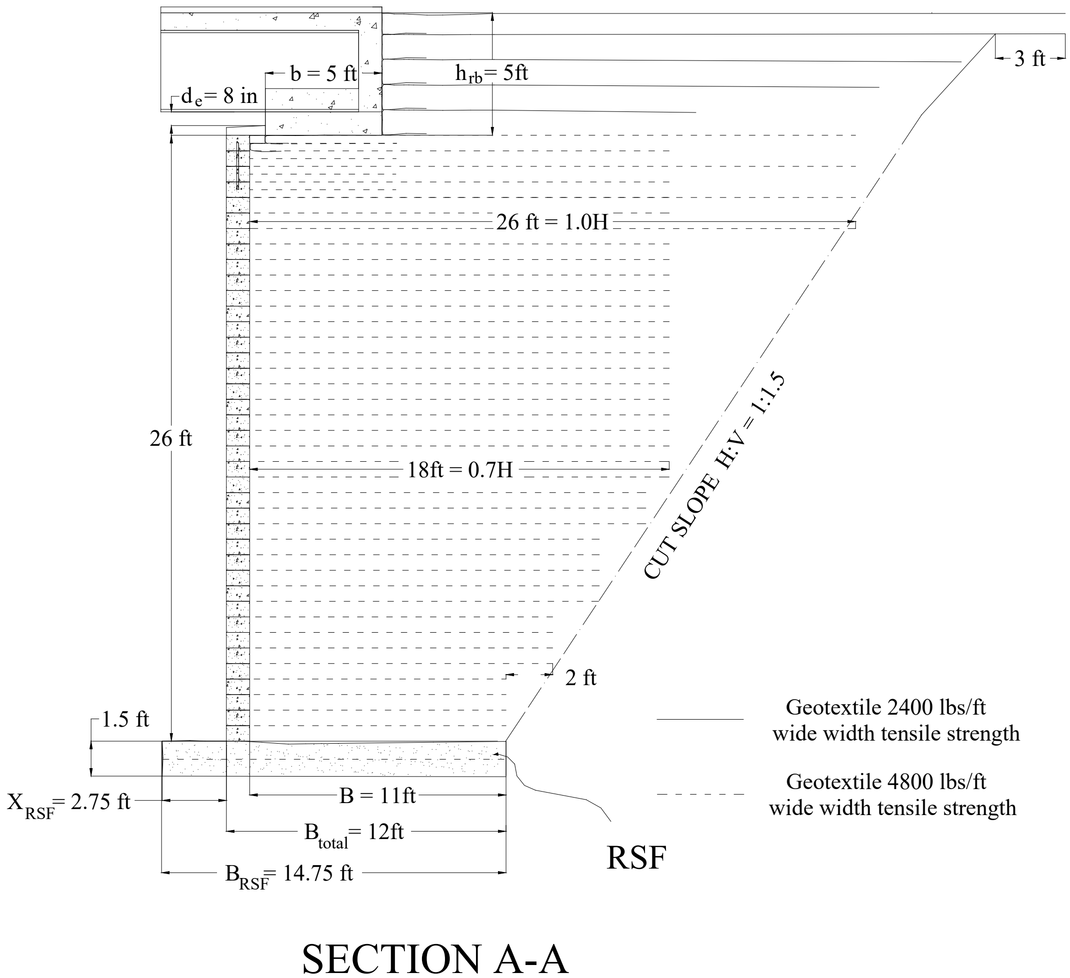
3. **Account for setback and clear space to calculate the elevation of the abutment face wall and the span length of the bridge**: The setback distance (α_b) is selected as 8 inches. The clear space distance is set to 8 inches (greater than the 2 percent minimum) because of the thickness of the required concrete footing.

4. **Determine the depth and volume of excavation necessary for construction**: For a B/H of 0.3, the minimum base length of the reinforcement not including the wall face (B) would be 7.8 ft; however, after the first iteration of design, this reinforcement embedment did not meet external stability requirements. B was therefore adjusted to 11 ft to satisfy all requirements. An SRW facing was selected for the project with a width of 12 inches so the total base width of the GRS abutment including the width of the facing (B_{total}) equals 12 ft for this project. The excavation extends out beyond the length of the RSF in front of the abutment wall face (x_{RSF}) by one-quarter of B_{total}, or 3.0 ft. The depth of the RSF (D_{RSF}) is equal to the minimum of 18 inches. Additional excavation is not necessary considering the dense foundation soils and bedrock underneath, as determined during the site investigation.

5. **Select the length of reinforcement throughout the height of the abutment**: As mentioned in step 4, B_{total} is 12 ft, which equates to a B/H of approximately 0.42. The reinforcement follows the cut slope up in zones until the distance from the cut slope is 2 ft, up to 0.7H, or about 18 ft (figure 111). The cut slope is at an angle of 1.0:1.5 (horizontal:vertical). Reinforcement lengths in the abutment do not extend beyond 26 ft limited by a B/H of 1.0 (figure 111).

6. **Add a bearing reinforcement zone underneath the bridge seat**: A minimum of three layers of bearing bed reinforcement at a vertical spacing of 4 inches is planned unless the subsequent design steps require additional layers. The length of the bearing bed reinforcement is 6.3 ft ($b + 2\alpha_b$).

7. **Place a beam seat above the bearing bed reinforcement zone to support the superstructure**: The total thickness of the beam seat consists of two lifts of 4 inches of wrapped-face GRS at the established setback distance of 8 inches. The reinforcement layers of the integrated approach behind the backwall of the superstructure will extend to the cut slope. The bridge engineer provided the depth of the backwall for the superstructure (i.e., 5 ft). This will require five wrapped layers in the integrated approach at a primary vertical spacing of 12 inches. The top two layers will extend beyond the cut slope by 3 ft.



Source: FHWA.

Figure 111. Illustration. Design example 1—reinforcement schedule and RSF dimensions.

C.1.5 Step 5—Calculate Applicable Loads and Pressures

The applied vertical pressures associated with the structure include the surcharges due to the bridge, traffic, and road base, along with the weight of the GRS backfill, RSF, and facing (see table 21). Lateral loads resulting from these applied pressures are calculated separately during the external stability calculations (see step 6 in section C.1.4).

Table 21. Applied vertical loads and pressures for Bowman Road.

Loads and Pressures	Value	Notes
Bridge DL (q_{DL}) (lb/ft^2)	1,882	$q_{DL} = \frac{Q_{DL}}{bL}$ Where: Q_{DL} = 501,683 lb (per abutment). b = 5 ft. L = 53.3 ft.
Bridge LL (q_{LL}) (lb/ft^2)	1,220	$q_{LL} = \frac{Q_{LL}}{bL}$ Where: Q_{LL} = 325,000 lb (per abutment).
Traffic surcharge (q_t) (lb/ft^2)	240	$q_t = h_{eq}\gamma_b$ Where: h_{eq} = 2 ft.(see 29)
Road base surcharge (q_{rb}) (lb/ft^2)	600	$q_{rb} = H_{rb}\gamma_{rb}$ Where: H_{rb} = 5 ft. γ_{rb} = 120 lb/ft^3.
Weight of GRS abutment (W) (lb/ft)	31,460	$W = BH\gamma_r$
Weight of RSF (W_{RSF}) (lb/ft)	2,475	$W_{RSF} = B_{RSF}D_{RSF}\gamma_r$
Weight of facing (W_{face}) (lb/ft)	2,340	$W_{face} = N_{block}\frac{W_{block}}{L_{block}}$ Where: N_{block} = 39. h_{block} = 8 inches. W_{block} = 80 lb. L_{block} = 16 inches.

C.1.6 Step 6—Conduct an External Stability Analysis

The external stability of a GRS-IBS is evaluated by looking at the following potential external failure mechanisms:

- Direct sliding (figure 25).
- Bearing capacity (figure 26).
- Global stability (figure 27).

C.1.6.1 Direct Sliding

Lateral translation, or direct sliding, must be resisted for stability. For an IBS, direct sliding shall be evaluated at both the interface between the GRS abutment and RSF and between the RSF and the foundation soils. For a standalone GRS abutment, direct sliding should be evaluated at the interface between the GRS abutment and the foundation soils.

C.1.6.1.1 Direct Sliding at the Base of the GRS Abutment

The driving forces on the GRS abutment are composed of the lateral forces due to the retained backfill, the road base, and the traffic surcharge behind the abutment.

The nominal resultant force due to the retained backfill is calculated in equation 74 using the previously presented formula in equation 9 as follows:

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2 = \frac{1}{2}\left(120\,\frac{\text{lb}}{\text{ft}^3}\right)(0.28)(26\text{ ft})^2 = 11{,}357\ \frac{\text{lb}}{\text{ft}} \tag{74}$$

The lateral force due to the road base and traffic surcharges are calculated in equation 75 and equation 76 using the previously provided formulae presented in equation 10 and equation 11, respectively.

$$F_{rb} = q_{rb} K_{ab} H = \left(600\,\frac{\text{lb}}{\text{ft}^2}\right)(0.28)(26\text{ ft}) = 4{,}368\ \frac{\text{lb}}{\text{ft}} \tag{75}$$

$$F_t = q_t K_{ab} H = \left(240\,\frac{\text{lb}}{\text{ft}^2}\right)(0.28)(26\text{ ft}) = 1{,}747\ \frac{\text{lb}}{\text{ft}} \tag{76}$$

The total factored driving force (F_R) is then calculated in equation 77 using the previously provided formula presented in equation 12.

$$F_R = \gamma_{EH\,MAX}(F_b + F_{rb}) + \gamma_{LS} F_t = 1.5\left(11{,}357\frac{\text{lb}}{\text{ft}} + 4{,}368\frac{\text{lb}}{\text{ft}}\right) + 1.75\left(1{,}747\frac{\text{lb}}{\text{ft}}\right) = 26{,}645\frac{\text{lb}}{\text{ft}} \tag{77}$$

The factored resisting force (R_R) is a product of the sliding resistance factor (equal to 1.0), the total factored resisting weight, and the friction factor between the base of the abutment and the RSF. The total factored resisting weight ($W_{T,R}$) is calculated in equation 78 using the previously provided formula in equation 14.

$$W_{T,R} = \gamma_{EV\,MIN} W + \gamma_{DC\,MIN}(q_{DL} b) + \gamma_{DC\,MIN}(W_{face}) + \gamma_{EV\,MIN}(q_{rb} b_{rb,t}) =$$

$$1.0(31{,}460\frac{\text{lb}}{\text{ft}}) + 0.9\left(1{,}882\frac{\text{lb}}{\text{ft}^2}\right)(5\text{ ft}) + 0.9\left(2{,}340\frac{\text{lb}}{\text{ft}}\right) + 1.0\left(600\frac{\text{lb}}{\text{ft}^2}\right)(5.3\text{ ft}) = 45{,}215\ \frac{\text{lb}}{\text{ft}} \tag{78}$$

The friction force (μ) is equal to tanϕ_{crit}. There was no interface shear testing performed, so the friction factor is assumed to equal two-thirds times the tangent of the reinforced backfill friction angle (ϕ_r). Therefore, R_R is then calculated in equation 79 using the previously provided formula in equation 13.

$$R_R = \Phi_\tau\left(W_{T,R}\mu\right) = 1.0\left[\left(45{,}215\frac{\text{lb}}{\text{ft}}\right)\left(\frac{2}{3}\right)\tan(50\text{ deg})\right] = 35{,}923\ \frac{\text{lb}}{\text{ft}} \tag{79}$$

R_R is greater than the factored driving force at the interface between the GRS abutment and the RSF (F_R) (equation 80), so the design satisfies this requirement.

$$\frac{R_R}{F_R} = \frac{35{,}923\frac{\text{lb}}{\text{ft}}}{26{,}645\frac{\text{lb}}{\text{ft}}} = 1.3 \geq 1.0 \tag{80}$$

C.1.6.1.2 Direct Sliding at the Base of the RSF

The driving forces on the GRS abutment (and RSF) comprise the lateral forces due to the retained backfill, the road base, and the traffic surcharge. The nominal resultant driving forces behind the GRS abutment and RSF from the retained backfill ($F_{b,RSF}$), the road base ($F_{rb,RSF}$), and the roadway LL surcharge ($F_{t,RSF}$) are determined along the height of the GRS abutment and depth of the RSF in equation 81 through equation 83 using the previously provided formulae in equation 17 through equation 19.

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2 = \frac{1}{2}\left(120\frac{\text{lb}}{\text{ft}^3}\right)(0.28)(26\text{ ft} + 1.5\text{ ft})^2 = 12{,}705\ \frac{\text{lb}}{\text{ft}} \tag{81}$$

$$F_{rb,RSF} = q_{rb}K_{ab}(H + D_{RSF}) = \left(600\frac{\text{lb}}{\text{ft}^2}\right)(0.28)(26\text{ ft} + 1.5\text{ ft}) = 4{,}620\ \frac{\text{lb}}{\text{ft}} \tag{82}$$

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF}) = \left(240\frac{\text{lb}}{\text{ft}^2}\right)(0.28)(26\text{ft} + 1.5\text{ft}) = 1{,}848\frac{\text{lb}}{\text{ft}} \tag{83}$$

The total factored driving force at the base of the RSF ($F_{R,RSF}$) is then calculated in equation 84 using the previously presented formula in equation 20.

$$\begin{gathered} F_{R,RSF} = \gamma_{EH\,MAX}\left(F_{b,RSF} + F_{rb,RSF}\right) + \gamma_{LS}F_{t.RSF} = \\ 1.5\left(12{,}705\frac{\text{lb}}{\text{ft}} + 4{,}620\frac{\text{lb}}{\text{ft}}\right) + 1.75\left(1{,}848\ \frac{\text{lb}}{\text{ft}}\right) = 29{,}222\ \frac{\text{lb}}{\text{ft}} \end{gathered} \tag{84}$$

The factored resisting force including the RSF ($R_{R,RSF}$) is a product of the sliding resistance factor (equal to 1.0), the total factored resisting weight and the friction factor between the base of the abutment and the RSF. The total factored resisting weight including the RSF ($W_{T,R,RSF}$) is calculated in equation 85 using the previously presented formula in equation 22.

$$W_{T,R,RSF} = W_{T,R} + \gamma_{EV\,MIN}W_{RSF} = 45{,}215\frac{\text{lb}}{\text{ft}} + 1.0\left(2{,}475\frac{\text{lb}}{\text{ft}}\right) = 47{,}690\frac{\text{lb}}{\text{ft}} \tag{85}$$

The friction force (μ) is equal to $\tan\phi_{crit}$. There was no interface shear testing performed, so the friction factor is assumed to equal the friction angle of the foundation soil (38 degrees) since it represents the weaker interface. Therefore, $R_{R,RSF}$ is then calculated in equation 86 using the previously presented formula in equation 21.

$$R_{R,RSF} = \Phi_\tau(W_{T,R,RSF}\mu_{RSF}) = 1.0\left[\left(47{,}690\,\frac{\text{lb}}{\text{ft}}\right)\tan(38\text{ degrees})\right] = 37{,}260\ \frac{\text{lb}}{\text{ft}} \tag{86}$$

$R_{R,RSF}$ is greater than the factored driving force at the interface between the RSF and the underlying foundation ($F_{R,RSF}$) (equation 87), so the design satisfies this requirement.

$$\frac{R_{R,RSF}}{F_{R,RSF}} = \frac{37{,}260\,\frac{\text{lb}}{\text{ft}}}{29{,}222\,\frac{\text{lb}}{\text{ft}}} = 1.3 \geq 1.0 \tag{87}$$

C.1.6.2 External Bearing Resistance

Before calculating the applied vertical bearing pressure ($\sigma_{v,base,R}$), the total factored vertical load (ΣV_R) must be calculated (equation 88) using the formula previously provided in equation 26.

$$\sum V_R = \gamma_{EV\,MAX}(W) + \gamma_{EV\,MAX}(W_{RSF}) + \gamma_{DC\,MAX}\left(W_{face}\right) + \gamma_{LS}\left(q_t b_{rb.t}\right) + \gamma_{EV\,MAX}\left(q_{rb} b_{rb,t}\right) + \gamma_{DC\,MAX}\left(q_{DL} b\right) + \gamma_{LS}\left(q_{LL} b\right) = 1.35\left(31{,}460\,\frac{\text{lb}}{\text{ft}}\right) + 1.35\left(2{,}475\,\frac{\text{lb}}{\text{ft}}\right) + 1.25\left(2{,}340\,\frac{\text{lb}}{\text{ft}}\right) + 1.75\left(240\,\frac{\text{lb}}{\text{ft}^2}\right)(5.3\text{ ft}) + 1.35\left(600\,\frac{\text{lb}}{\text{ft}^2}\right)(5.3\text{ ft}) + 1.25\left(1{,}882\,\frac{\text{lb}}{\text{ft}^2}\right)(5\text{ ft}) + 1.75\left(1{,}220\,\frac{\text{lb}}{\text{ft}^2}\right)(5\text{ ft}) = 77{,}694\ \frac{\text{lb}}{\text{ft}} \tag{88}$$

In addition, the eccentricity of the resulting force at the center base of the wall ($e_{B,R}$) or RSF must be calculated. To do this, the factored driving ($\Sigma M_{D,R}$) and resisting moments ($M_{R,R}$) must be found in equation 89 and equation 90, respectively, using the formulae previously presented in equation 28 and equation 29, respectively.

$$\sum M_{D,R} = \gamma_{EH\,MAX} F_{b,RSF}\left(\frac{H + D_{RSF}}{3}\right) + \gamma_{LS} F_{t,RSF}\left(\frac{H + D_{RSF}}{2}\right) + \gamma_{EH\,MAX} F_{rb,RSF}\left(\frac{H + D_{RSF}}{2}\right) = 1.50\left(12{,}705\ \frac{\text{lb}}{\text{ft}}\right)\left(\frac{26\text{ ft} + 1.5\text{ ft}}{3}\right) + 1.75\left(1{,}848\,\frac{\text{lb}}{\text{ft}}\right)\left(\frac{26\text{ ft} + 1.5\text{ ft}}{2}\right) + 1.50\left(4{,}620\ \frac{\text{lb}}{\text{ft}}\right)\left(\frac{26\text{ ft} + 1.5\text{ ft}}{2}\right) = 314{,}449\ \frac{\text{ft}\cdot\text{lb}}{\text{ft}} \tag{89}$$

$$\sum M_{R,R} = (\gamma_{DC\,MAX} q_{DL} b + \gamma_{LS} q_{LL} b) \left[\left(\frac{b}{2} + a_b\right) - \left(\frac{B_{RSF}}{2} - x_{RSF} - b_{block}\right)\right]$$
$$+ \left(\gamma_{LS} q_t b_{rb,t} + \gamma_{EV\,MAX} q_{rb} b_{rb,t}\right)\left(\frac{B_{RSF}}{2} - \frac{b_{rb}}{2}\right) + \gamma_{EV\,MAX} W \left(\frac{B_{RSF}}{2} - \frac{B}{2}\right) + \gamma_{DC\,MAX} W_{face}$$
$$\left(B + \frac{b_{block}}{2} - \frac{B_{RSF}}{2}\right) = \left(1.25\left(1{,}882\,\frac{\text{lb}}{\text{ft}^2}\right)(5\text{ ft}) + 1.75\left(1{,}220\,\frac{\text{lb}}{\text{ft}^2}\right)(5\text{ft})\right)$$
$$\left[\left(\frac{5\text{ ft}}{2} + 0.67\text{ ft}\right) - \left(\frac{15\text{ ft}}{2} - 3\text{ ft} - 1\text{ ft}\right)\right] + \left(1.75\left(240\,\frac{\text{lb}}{\text{ft}^2}\right)(5.3\text{ ft}) + 1.5\left(600\,\frac{\text{lb}}{\text{ft}^2}\right)(5.3\text{ ft})\right)$$
$$\left(\frac{15\text{ ft}}{2} - \frac{5.3\text{ ft}}{2}\right) + 1.35\left(31{,}460\,\frac{\text{lb}}{\text{ft}}\right)\left(\frac{15\text{ ft}}{2} - \frac{11\text{ ft}}{2}\right) + 1.25\left(2{,}340\,\frac{\text{lb}}{\text{ft}}\right)$$
$$\left(11\text{ ft} + \frac{1\text{ ft}}{2} - \frac{15\text{ft}}{2}\right) = 123{,}168\text{ ft·lb/ft} \tag{90}$$

The eccentricity is then calculated in equation 91 using the equation previously provided in equation 27 as follows:

$$e_{B,R} = \frac{\sum M_{D,R} - \sum M_{R,R}}{\sum V_R} = \frac{314{,}449\text{ft·lb/ft} - (123{,}168\text{ ft·lb/ft})}{77{,}694\,\frac{\text{lb}}{\text{ft}}} = 2.5\text{ ft} \tag{91}$$

The factored vertical pressure at the base of the GRS mass ($\sigma_{v,base,R}$) is a result of the weight of the GRS abutment, RSF, and integrated approach, along with the bridge seat load and traffic surcharge. It is calculated in equation 92 using the previously provided formula in equation 25.

$$\sigma_{v,base,R} = \frac{\sum V_R}{B_{RSF} - 2e_{B,R}} = \frac{77{,}694\,\frac{\text{lb}}{\text{ft}}}{15\text{ ft} - 2(2.5\text{ ft})} = 7{,}769\,\frac{\text{lb}}{\text{ft}^2} \tag{92}$$

The factored bearing resistance (q_R) is then calculated in equation 93 using the previously provided formula presented in equation 30. The bearing capacity factors N_c, N_γ, and N_q were found using table 9 for the foundation friction angle of 38 degrees.

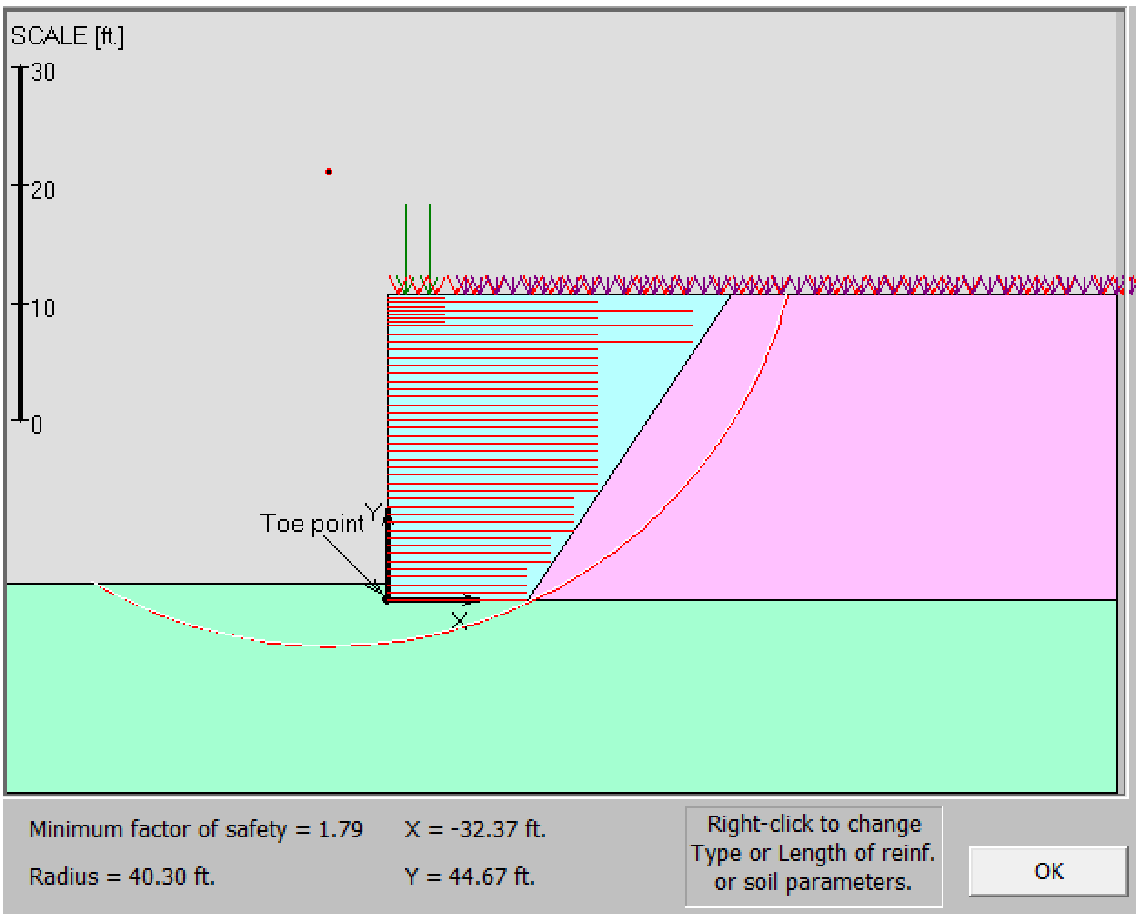
$$q_r = \Phi_{bc}\left(c_f' N_c + \frac{1}{2} B' \gamma_f' N_\gamma + \gamma_f' D_f N_q\right)$$
$$= 0.65\left[\left(0\,\frac{lb}{ft^2}\right)(61.40) + \frac{1}{2}\left(15\,ft - 2(2.5\,ft)\right)\left(77.6\,\frac{lb}{ft^3}\right)(78)\right.$$
$$\left. + 77.6\,\frac{lb}{ft^3}(1.5\,ft)(48.9) = 23{,}371\,\frac{lb}{ft^2}\right] \tag{93}$$

The ratio of q_R and $\sigma_{v,base,R}$ must be greater than or equal to 1.0, as shown in equation 94 (which it is).

$$\frac{q_R}{\sigma_{v,base,\ R}} = \frac{23{,}371\ \dfrac{\text{lb}}{\text{ft}^2}}{7{,}769\,\dfrac{\text{lb}}{\text{ft}^2}} = 3.0 \geq 1.0 \tag{94}$$

C.1.6.3 Global Stability

Global and compound stability were checked using the software program ReSSA; other similar programs could be used as well. A screenshot of the global stability failure mode is shown in figure 112. The factor of safety is found to equal 6.6, which is much greater than the minimum requirement of 1.5. As such, global and compound stability is not a problem.



Source: FHWA.

Figure 112. Screenshot. ReSSA results for global stability.

C.1.7 Step 7—Conduct Internal Stability Analysis

Internal stability for GRS includes ensuring adequate internal bearing resistance, tolerable deformations, and required reinforcement strength.

C.1.7.1 Internal Bearing Resistance

The bearing resistance of a GRS abutment can be determined using two different methods: empirical or analytical. These are described in more detail in the following subsections.

C.1.7.1.1 Empirical Method

The empirical method uses the load test results of a performance test on an identical (or very similar) GRS composite material to that used in the field. For this project, no performance tests were conducted.

C.1.7.1.2 Analytical Method

Alternatively, the internal bearing resistance was found analytically for a granular backfill, where the confining stress due to the facing (σ_c) is assumed to equal 0, S_v equals 8 inches, d_{max} equals 0.5 inch (table 20), T_f equals 4,800 lb/ft, and ϕ_r equals 50 degrees (table 20). Note that although the spacing under the bridge bearing area is 4 inches, 8 inches was chosen in this calculation for the entire abutment to be conservative.

The nominal bearing resistance of the GRS abutment using the analytical method ($q_{n,an}$) is solved for in equation 95 using the previously provided formula in equation 34. Additionally, the coefficient of passive earth pressure for the reinforced backfill (K_{pr}) is solved for in equation 96 using the previously provided formula in equation 35.

$$q_{n,an} = \left[0.7^{\left(\frac{S_v}{6d_{max}}\right)}\frac{T_f}{S_v}\right]K_{pr} = \left[0.7^{\left(\frac{8\text{ inches}}{6(0.5\text{ inch})}\right)}\frac{4{,}800\,\frac{\text{lb}}{\text{ft}}}{0.67\text{ ft}}\right]7.55 = 20{,}895\ \frac{\text{lb}}{\text{ft}^2} \tag{95}$$

$$K_{pr} = \frac{1+\sin\phi_r}{1-\sin\phi_r} = \frac{1+\sin(50\text{ degrees})}{1-\sin(50\text{ degrees})} = 7.55 \tag{96}$$

The applied factored vertical pressure on the abutment ($V_{applied,f}$) is defined in equation 97 using the previously presented formula in equation 32.

$$V_{applied,f} = \gamma_{DC\,MAX}q_{DL} + \gamma_{LL}q_{LL} = 1.25\left(1{,}882\frac{\text{lb}}{\text{ft}^2}\right) + 1.75\left(1{,}220\frac{\text{lb}}{\text{ft}^2}\right) = 4{,}488\ \frac{\text{lb}}{\text{ft}^2} \tag{97}$$

The factored applied pressure is less than the factored bearing resistance (equation 98), so the GRS abutment as designed can support the applied loads.

$$\frac{\Phi_{cap}(q_{n,an})}{V_{applied,f}} = \frac{0.45\left(20{,}895\ \frac{\text{lb}}{\text{ft}^2}\right)}{4{,}488\ \frac{\text{lb}}{\text{ft}^2}} = 2.1 \geq 1.0 \tag{98}$$

C.1.7.2 Deformations

C.1.7.2.1 Vertical

In the absence of a performance test, the vertical strain was limited to 1 percent of the abutment height (or about 3 inches) by imposing a prescribed service bearing pressure for DL, as solved for in equation 99 using the previously provided formula in equation 37.

$$q_{DL,allow\ @\varepsilon=1\%} = 0.2\left[0.7^{S_v/6d_{max}}\left(\frac{T_f}{S_v}\right)K_{pr}\right] = 0.2\left[0.7^{8\ inches/6(0.5\ inch)}\left(\frac{4{,}800\,\frac{lb}{ft}}{0.67\,ft}\right)\right]7.55$$
$$= 4{,}179\,\frac{lb}{ft^2} \tag{99}$$

The allowable DL (4,179 lb/ft^2) is considerably more than the service DL pressure (1,882 lb/ft^2), so deformations are expected to be less than 3 inches. Considering the allowable DL is about 10 percent of the bearing resistance, a more realistic estimate of deformation would be about 1.5 inch (vertical strain of less than 0.5 percent).

C.1.7.2.2 Lateral

The lateral strain and deformation are found in equation 100 and equation 101, respectively, assuming a maximum settlement of 3 inches (0.25 ft) using the previously presented formula in equation 39 and equation 38, respectively.

$$\varepsilon_L = 2\varepsilon_v = 2(1.0\text{ percent}) = 2.0\text{ percent} \tag{100}$$

$$D_L = \frac{2b_q D_v}{H} = \frac{2(5\text{ ft} + 0.67\text{ ft})(0.25\text{ ft})}{26\text{ ft}} = 0.11\text{ ft} = 1.3\text{ inch} \tag{101}$$

Assuming the more realistic 1.5 inches of vertical deformation for this project, lateral deformation would be estimated at 0.65 inch.

C.1.7.3 Required Reinforcement Strength

The ultimate strength of the reinforcement used in the project was 4,800 lb/ft. According to the manufacturer, $T_{@\varepsilon = 2\%}$ was equal to approximately 920 lb/ft.

C.1.7.3.1 Strength Limit

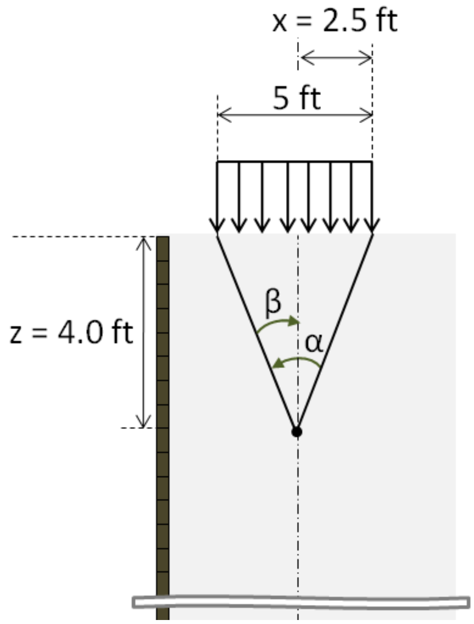
The factored maximum required reinforcement strength is found as a function of depth, reinforcement spacing, and maximum aggregate size (equation 40). The factored total lateral pressure ($\sigma_{h,f}$) is a combination of the factored lateral pressures due to the equivalent bridge load ($\sigma_{h,bridge,f}$), the road base surcharge ($\sigma_{h,rbf}$), the traffic surcharge ($\sigma_{h,tf}$), the LL due to trucks ($\sigma_{h,LLf}$), and the GRS reinforced soil ($\sigma_{h,Wf}$). The lateral stress is calculated for each depth of interest (each layer of reinforcement). All lateral stresses are calculated and shown in table 22.

Table 22. Design example 1—required reinforcement strength calculations.

	Equivalent Bridge Load				Road Base DL and Approach LL (lb/ft^2)				GRS Abutment Fill (lb/ft^2)		Total Lateral Pressure Service (lb/ft^2)		Strength Check		2 Percent Service Check	
z (ft)	α	β	$\sigma_{h,bridge}$ (lb/ft^2)	$\sigma_{h,bridge,f}$ (lb/ft^2)	$\sigma_{h,rb}$	$\sigma_{h,rb,f}$	$\sigma_{h,t}$	$\sigma_{h,t,f}$	$\sigma_{h,W}$	$\sigma_{h,W,f}$	$\sigma_{h,f}$	σ_h	$T_{req,f}$ (lb/ft)	$T_{req,f} > T_{f,f}$ (Yes/No)	T_{req} (lb/ft)	$T_{req} > T_{@\,\varepsilon=2\%}$ (Yes/No)
0.7	2.62	–1.31	298	417	79	119	32	55	10	15	606	418	1,046	No	722	No
1.3	2.16	–1.08	285	400	79	119	32	55	19	29	604	416	1,042	No	718	No
2.0	1.79	–0.90	264	370	79	119	32	55	29	44	588	404	1,015	No	698	No
2.7	1.51	–0.75	239	335	79	119	32	55	39	58	568	389	980	No	671	No
3.3	1.29	–0.64	214	300	79	119	32	55	49	73	548	374	946	No	646	No
4.0	1.12	–0.56	192	269	79	119	32	55	58	87	530	362	919	No	628	No
4.7	0.98	–0.49	173	243	79	119	32	55	68	102	519	353	897	No	608	No
5.3	0.88	–0.44	157	220	79	119	32	55	78	117	511	346	882	No	597	No
6.0	0.79	–0.39	143	200	79	119	32	55	87	131	506	342	874	No	590	No
6.7	0.72	–0.36	131	184	79	119	32	55	97	146	504	340	870	No	586	No
7.3	0.66	–0.33	121	169	79	119	32	55	107	160	505	339	871	No	585	No
8.0	0.61	–0.30	112	157	79	119	32	55	117	175	507	340	874	No	587	No
8.7	0.56	–0.28	104	146	79	119	32	55	126	189	510	342	881	No	590	No
9.3	0.52	–0.26	98	137	79	119	32	55	136	204	516	345	8	No	595	No
10.0	0.49	–0.24	92	128	79	119	32	55	146	219	522	349	900	No	602	No
10.7	0.46	–0.23	86	121	79	119	32	55	155	233	529	353	913	No	609	No
11.3	0.43	–0.22	82	114	79	119	32	55	165	248	537	358	926	No	618	No
12.0	0.41	–0.21	77	108	79	119	32	55	175	262	545	363	941	No	627	No
12.7	0.39	–0.19	73	103	79	119	32	55	185	277	555	369	957	No	637	No
13.3	0.37	–0.19	70	98	79	119	32	55	194	291	564	376	974	No	648	No
14.0	0.35	–0.18	67	93	79	119	32	55	204	306	574	382	991	No	659	No
14.7	0.34	–0.17	64	89	79	119	32	55	214	321	585	389	1,009	No	671	No
15.3	0.32	–0.16	61	86	79	119	32	55	223	335	596	396	1,028	No	683	No
16.0	0.31	–0.15	59	82	79	119	32	55	233	350	607	403	1,047	No	696	No
16.7	0.30	–0.15	56	79	79	119	32	55	243	364	618	411	1,067	No	709	No
17.3	0.29	–0.14	54	76	79	119	32	55	253	379	630	418	1,087	No	722	No
18.0	0.28	–0.14	52	73	79	119	32	55	262	393	642	426	1,107	No	735	No
18.7	0.27	–0.13	51	71	79	119	32	55	272	408	654	434	1,128	No	749	No
19.3	0.26	–0.13	49	68	79	119	32	55	282	423	666	442	1,149	No	762	No

z (ft)	Equivalent Bridge Load				Road Base DL and Approach LL (lb/ft^2)				GRS Abutment Fill (lb/ft^2)		Total Lateral Pressure Service (lb/ft^2)		Strength Check		2 Percent Service Check	
	α	β	$\sigma_{h,bridge}$ (lb/ft^2)	$\sigma_{h,bridge,f}$ (lb/ft^2)	$\sigma_{h,rb}$	$\sigma_{h,rb,f}$	$\sigma_{h,t}$	$\sigma_{h,t,f}$	$\sigma_{h,W}$	$\sigma_{h,W,f}$	$\sigma_{h,f}$	σ_h	$T_{req,f}$ (lb/ft)	$T_{req,f} > T_{f,f}$ (Yes/No)	T_{req} (lb/ft)	$T_{req} > T_{@\,\varepsilon=2\%}$ (Yes/No)
20.0	0.25	–0.12	47	66	79	119	32	55	291	437	678	450	1,170	No	776	No
20.7	0.24	–0.12	46	64	79	119	32	55	301	452	691	458	1,192	No	791	No
21.3	0.23	–0.12	44	62	79	119	32	55	311	466	703	466	1,214	No	805	No
22.0	0.23	–0.11	43	60	79	119	32	55	321	481	716	475	1,236	No	819	No
22.7	0.22	–0.11	42	58	79	119	32	55	330	495	729	483	1,258	No	834	No
23.3	0.21	–0.11	41	57	79	119	32	55	340	510	742	492	1,280	No	849	No
24.0	0.21	–0.10	39	55	79	119	32	55	350	525	755	500	1,302	No	864	No
24.7	0.20	–0.10	38	54	79	119	32	55	359	539	768	509	1,325	No	879	No
25.3	0.20	–0.10	37	52	79	119	32	55	369	554	781	518	1,348	No	894	No
26.0	0.19	–0.10	36	51	79	119	32	55	379	568	794	527	1,371	No	909	No

An example calculation for the required reinforcement strength at a depth (z) of 4 ft (or layer six of primary reinforcement from the top) is shown in figure 113. First, the lateral pressure must be found (equation 102, which uses the previously provided formula in equation 41). The location of interest is directly under the centerline of the bridge load (where $x = 0.5b = 0.5(5\text{ ft}) = 2.5\text{ ft}$).



Source: FHWA.

Figure 113. Illustration. Lateral pressure due to the design example bridge loads.

$$\sigma_{h,f} = \sigma_{h,W,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f} = 87\frac{\text{lb}}{\text{ft}^2} + 269\frac{\text{lb}}{\text{ft}^2} + 119\frac{\text{lb}}{\text{ft}^2} + 55\frac{\text{lb}}{\text{ft}^2} = 530\frac{\text{lb}}{\text{ft}^2} \quad (102)$$

The lateral pressure resulting from the weight of the GRS abutment is found in equation 103 using the previously provided formula in equation 42.

$$\sigma_{h,W,f} = \gamma_{EH\,MAX}(\gamma_r z K_{ar}) = 1.5\left[110\frac{\text{lb}}{\text{ft}^3}(4\text{ ft})\left(\frac{1-\sin(50\text{ degrees})}{1+\sin(50\text{ degrees})}\right)\right] = 87\frac{\text{lb}}{\text{ft}^2} \quad (103)$$

To simplify calculations, the pressure due to an equivalent bridge load is found by assuming the surcharges due to the road base and the traffic uniformly extend to the edge of the GRS abutment (and thus must be subtracted out), as shown in equation 104 using the previously presented formula in equation 43.

$$\sigma_{h,bridge,f} = \frac{(\gamma_{DC\,MAX}q_{DL} + \gamma_{LL}q_{LL}) - (\gamma_{EH\,MAX}q_{rb} + \gamma_{LS}q_t)}{\pi}\left[\alpha_b + \sin(\alpha_b)\cos(\alpha_b + 2\beta_b)\right]K_{ar}$$
$$= \frac{\left[1.25\left(1{,}882\frac{\text{lb}}{\text{ft}^2}\right) + 1.75\left(1{,}220\frac{\text{lb}}{\text{ft}^2}\right)\right] - \left[1.5\left(600\frac{\text{lb}}{\text{ft}^2}\right) + 1.75\left(240\frac{\text{lb}}{\text{ft}^2}\right)\right]}{\pi}$$
$$[1.12\text{ radians} + \sin(1.12\text{ radians})\cos(1.12\text{ radians} + 2(-0.56\text{ radians}))]\ (0.132) = 269\frac{\text{lb}}{\text{ft}^2} \quad (104)$$

The values for α and β are found in equation 105 and equation 106 using the previously presented formulae in equation 46 and equation 47, respectively.

$$\alpha_b = \tan^{-1}\left(\frac{b}{2z}\right) - \beta_b = \tan^{-1}\left(\frac{5\text{ ft}}{2(4\text{ ft})}\right) - (-32\text{ degrees}) = 64\text{ degrees} = 1.12\text{ radians} \tag{105}$$

$$\beta_b = \tan^{-1}\left(\frac{-b}{2z}\right) = \tan^{-1}\left(\frac{-5\text{ ft}}{2(4\text{ ft})}\right) = -32\text{ degrees} = -0.56\text{ radians} \tag{106}$$

The lateral pressures due to the road base and traffic surcharges are also computed, as shown in equation 107 and equation 108 using the previously presented formulae in equation 44 and equation 45, respectively.

$$\sigma_{h,rb,f} = \gamma_{EH\,MAX} q_{rb} K_{ar} = 1.5\left(600\frac{\text{lb}}{\text{ft}^2}\right)(0.132) = 119\frac{\text{lb}}{\text{ft}^2} \tag{107}$$

$$\sigma_{h,t,f} = \gamma_{LS} q_t K_{ar} = 1.75\left(240\frac{\text{lb}}{\text{ft}^2}\right)(0.132) = 55\frac{\text{lb}}{\text{ft}^2} \tag{108}$$

The factored maximum required reinforcement strength at this reinforcement level can then be found in equation 109 using the previously defined formula presented in equation 40.

$$T_{req,f} = \left[\frac{\sigma_{h,f}}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v = \left[\frac{530\frac{\text{lb}}{\text{ft}^2}}{0.7^{\left(\frac{8\text{ inches}}{6(0.5\text{ inch})}\right)}}\right] 0.67\text{ ft} = 919\ \frac{\text{lb}}{\text{ft}} \tag{109}$$

To determine the factored reinforcement strength ($T_{f,f}$), equation 110 is used accounting for the previously defined formula presented in equation 48.

$$T_{f,f} = \Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right) = 0.4T_f = 0.4\left(4{,}800\frac{\text{lb}}{\text{ft}}\right) = 1{,}920\frac{\text{lb}}{\text{ft}} \tag{110}$$

$T_{f,f}$ is then checked against the factored required reinforcement strength ($T_{req,f}$), as shown in equation 111, which satisfies the strength limit.

$$\frac{T_{f,f}}{T_{req,f}} = \frac{1{,}920\frac{\text{lb}}{\text{ft}}}{919\ \frac{\text{lb}}{\text{ft}}} = 2.1 \geq 1.0 \tag{111}$$

Note that a weaker geosynthetic could be selected (e.g., 2,400 lb/ft ultimate strength) based on the calculated values, but the service limit check would still need to be ensured.

C.1.7.3.2 Service Limit

The check at the service limit is identical to that at the strength limit except no load factors are applied. At the same depth shown in the previous example (z = 4 ft), the required reinforcement strength at the strength limit is shown in equation 112 using the previously presented formula in equation 50.

$$T_{req} = \left[\frac{\sigma_h}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v = \left[\frac{362\,\frac{\text{lb}}{\text{ft}^2}}{0.7^{\left(\frac{8\text{ inches}}{6(0.5\text{ inch})}\right)}}\right] 0.67\text{ ft} = 628\ \frac{\text{lb}}{\text{ft}} \tag{112}$$

The required reinforcement strength at this depth is less than the manufacturer's supplied $T_{@\varepsilon = 2\%}$ of 920 lb/ft.

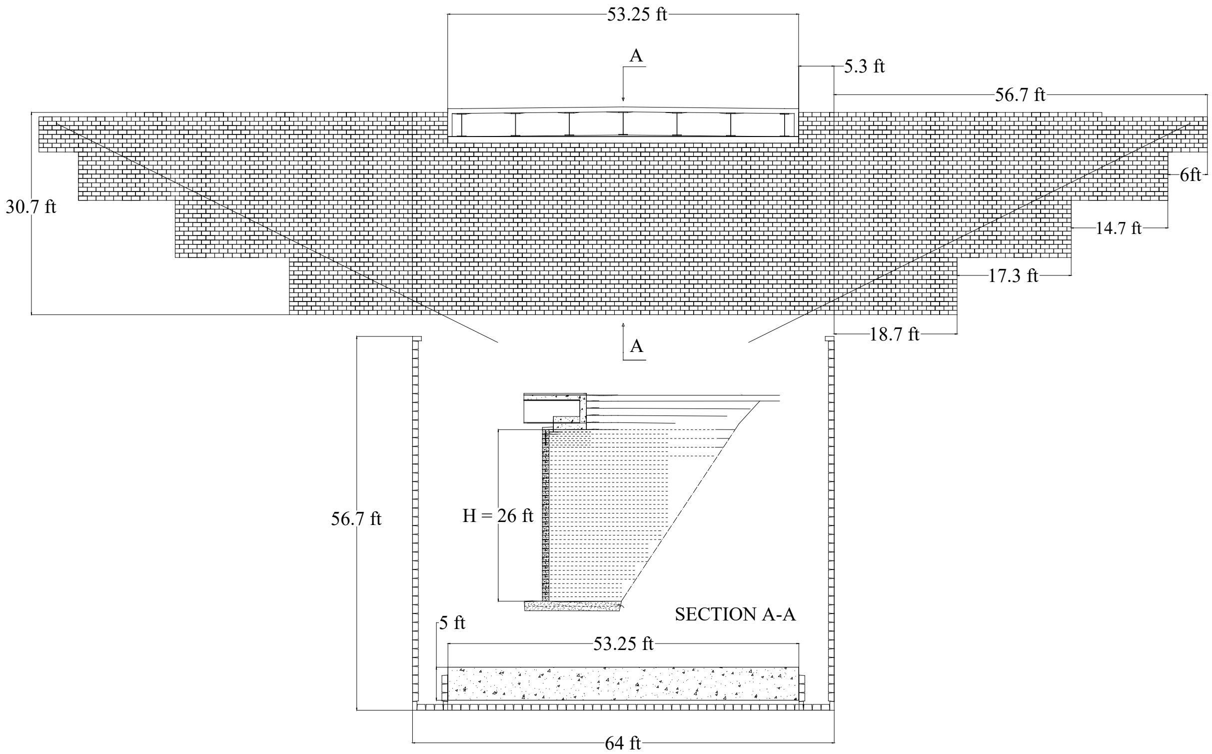
Based on table 22, the required reinforcement strength does not exceed the allowable strength or the strength at 2 percent at any reinforcement layer. Therefore, no bearing bed reinforcement is needed; however, the minimum requirement is that the bearing bed reinforcement should extend through three courses of blocks.

C.1.8 Step 8—Implement Design Details

All design details were considered for the project, including type of guardrails, utilities, etc.

C.1.9 Step 9—Finalize Material Quantities and Layout

The amount of reinforcement and other materials necessary is based on the reinforcement schedule, wing walls, and footprint (figure 114). Reinforcement material usually comes in 12- to 18-ft-wide rolls. The number of facing blocks will be determined from the height and length of the abutment and wing walls. The amount of backfill required is determined in a similar fashion. Once the final quantities are established, it is a good rule of thumb to order at least 10 percent more to account for unforeseen conditions.



Source: FHWA.

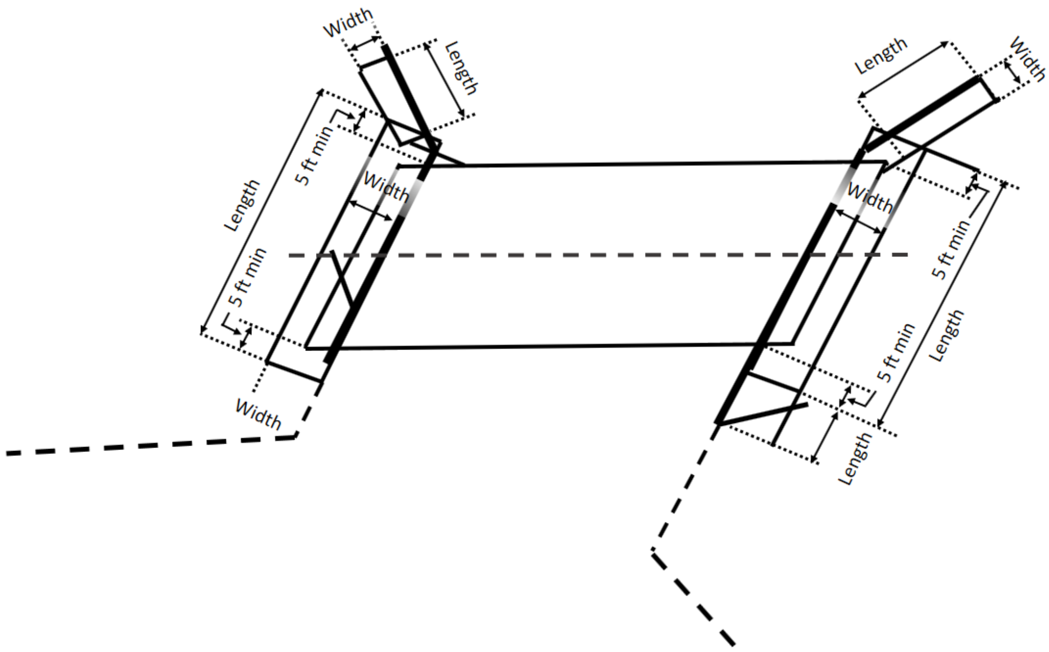
Figure 114. Illustration. Abutment face view, cross section, and footprint.

C.2 EXAMPLE 2: BOWMAN ROAD BRIDGE IN DEFIANCE COUNTY, OH

The construction of the Bowman Road Bridge in Defiance County, OH, was completed in October 2005. This project represents the initial deployment of GRS-IBS in the world. As this structure demonstrates many of the variables that can be accommodated by GRS-IBS technology, it was chosen for this design example to illustrate the versatility of this construction method.

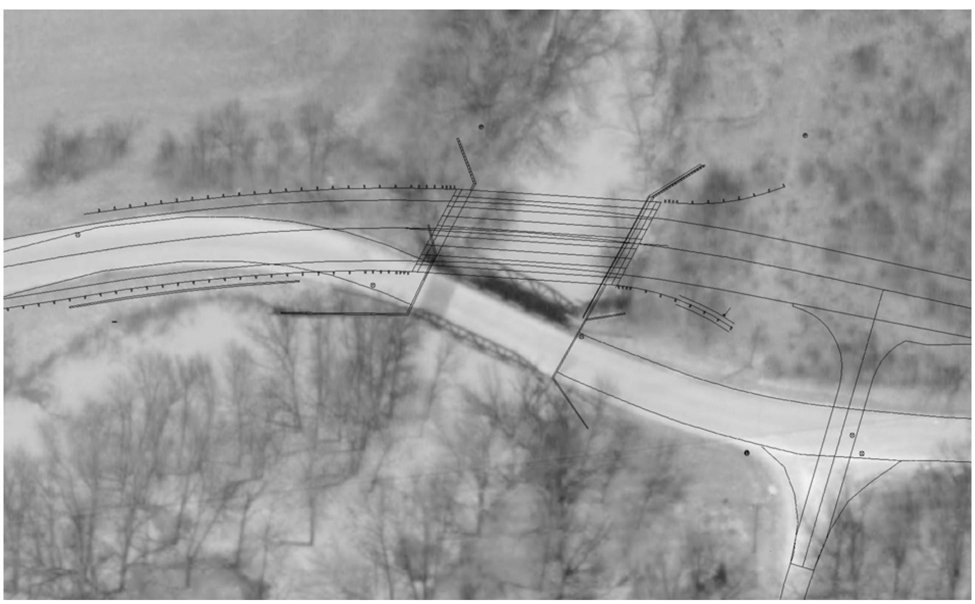
C.2.1 Step 1—Establish Project Requirements

A GRS-IBS was used for the Bowman Road Bridge project. This project included an abutment and a wing wall on each side of the bridge. A top view of the proposed project is shown in figure 115, while figure 116 gives an aerial view of the site with the proposed bridge superimposed.



Source: FHWA.

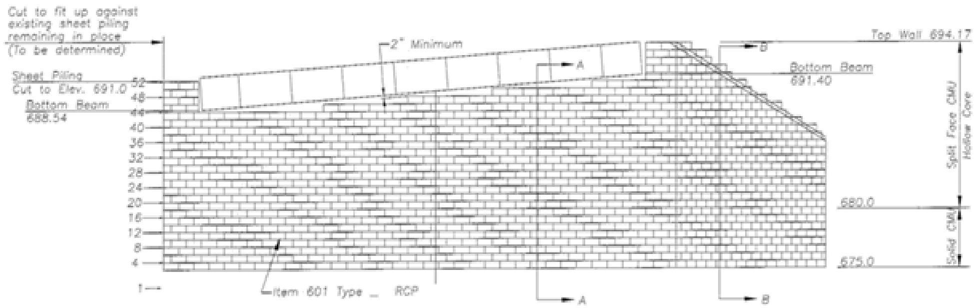
Figure 115. Illustration. Top view of Bowman Road Bridge project showing the bridge, two abutments, and wing walls.



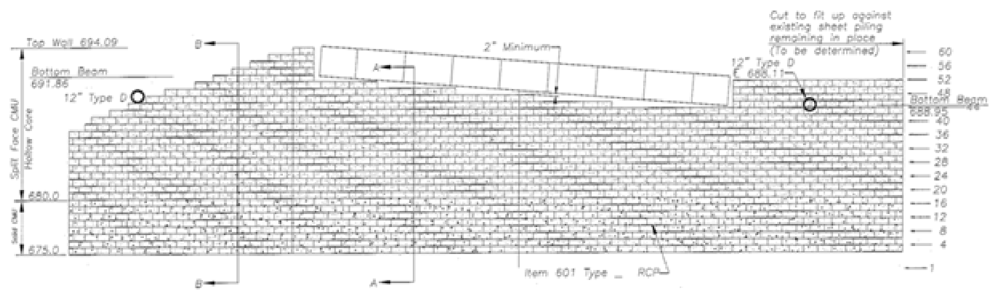
Copyright: Defiance County, OH.

Figure 116. Photo. Aerial view of the existing site with the new Bowman Road Bridge plans superimposed.

Schematics of the proposed abutments are shown in figure 117.



A. West abutment looking west.



B. East abutment looking east.

Figure 117. Illustrations. West and east abutments for the Bowman Road Bridge project.

The project requirements are described in the following lists.

Geometry requirements are as follows:

- **Height of GRS abutment (H_{abut})**: 15.25 ft.
- **Abutment length (L_{abut})**: 43.6 ft.
- **Batter angle:** 2 degrees.
- **Bridge width**: 34 ft.
- **Wall placement with respect to ground conditions**: No back slope or toe slope.
- **Skew angle**: 24 degrees.
- **Grade**: 0.006 ft/ft.
- **Superelevation angle**: 7.6 degrees.

Tolerable movement performance criteria are as follows:

- **Vertical settlement**: Vertical strain (ε_v) is limited to 0.5 percent, so vertical settlement of the GRS mass (D_v) is limited to 0.076 ft ($D_v = \varepsilon_v \times H$).

- **Lateral displacements**: Lateral strain (ε_L) is limited to 1 percent.

- **Differential settlement**: 1/200.

- **Angular distortion between abutments**: 1/200.
- **Design life**: 100 years.

Performance criteria constraints include the following:

- **Environmental:** None.
- **Construction:** Sheet piling from the existing bridge remains in place. This reduces the need for two wing walls in the IBS; only one wing wall is required.
- **Scour and stream stability:** A hydraulic analysis needs to be performed at the site because the bridge crosses water.

C.2.2 Step 2—Perform a Site Evaluation

The original bridge at this site was replaced because it was functionally obsolete and structurally deficient. The previous bridge did not experience any problems related to settlement and excessive deformations due to the site conditions. However, a sheet pile wall was installed to protect the stone wall abutments from erosion. The site evaluation determined that the existing sheet piling should remain in place to support the stream bank. This also eliminated the need for wing walls on one side adjacent to the old bridge.

The replacement structure required realignment to meet current road design standards for improved roadway safety because the location proved to be prone to accidents. The new Bowman Road Bridge crosses Powell Creek. The proposed location of the new abutments adjacent to the old bridge was not expected to cause any problems with the stream flow.

A hydraulic analysis confirmed that the existing bridge did not have any appreciable potential scour. Therefore, a RSF with appropriate scour countermeasures (riprap in this case) was used.

A subsurface evaluation was conducted by performing SPTs near the site (figure 118). The physical characteristics of the soil were determined through index tests taken on split spoon samples. The foundation soil at the site is an over-consolidated clay (with intermediate layers of sandy silt and gravels) with blow counts between 200 and 300 blows/ft at the elevation of the bottom of the abutment (found from figure 118). After talking with local experts, it was found that the clay has historically been preloaded with a nearly 1-mi-thick sheet of ice. The clay in this region is also known to be fat and sticky when wet. The bearing resistance of the stiff clay had not been a problem in similar past projects in the area.

PID No. 12211 | Sampler Type: Split Spoon | Surface Elev. 212.1 m | DEF-CR09 - 0.265 (0.18)

Holly Road Over Powell Creek

Date Started: 08/09/95 | Casing Length: 8.5 m | Water Elev. None m (Initial) | Highland Twp., Defiance Co., Ohio

207.5 m (Completion) | W SR 15

Date Completed: 08/09/95 | Station & Offset: 0+300, 1.5 m Left | 207.1 m (After 0.25 Hours) | Bowser-Morner Job No. 106179

Elev. (m)	Depth (m)	Blows per 150 mm	Description	Sample No.	Physical Characteristics % Agg.	% C. S.	% F. S.	% Silt	% Clay	LL (%)	PI (%)	W.C. (%)	ODOT Class.
212.1	0												
211.8			FILL - Asphalt (150 mm)										
		7-7-5	FILL - Crushed stone base (430 mm)	1A	8.4	9.0	23.3	23.4	35.9	33	17	9.9	A-6b (6)
211.5	1		Stiff brown and gray clay, some sand, some silt, trace gravel and roots, moist										
		3-3-4		2A								20.6	
210.3	2	3-3-9	Stiff brown clay, some silt, trace sand, moist	3A								31.0	
	3	5-5-8	...Thin silty sand seam at approx. 2.8 m	4A								32.7	
208.7		6-10-14	Very stiff brown and gray clay, some silt and sand, trace gravel, moist	5A	2.7	3.7	7.9	27.1	58.6	40	21	19.4	A-6b (12)
	4	8-11-11	...Thin silty sand seam at 4.3 m	6A								14.7	
	5	8-10-11		7A								17.9	
		10-14-33		8A								11.6	
206.3	6		Hard brown and gray clay and silt, some sand, trace gravel, moist										
		20-26-36		9A								11.0	
205.7	7		Very dense fine sand and silt, some clay, trace gravel, damp										
		65-100/100 mm		10A								6.4	
	8	87-100/100 mm		11A								7.0	
	9	47-77-100/100 mm		12A								6.9	

Figure 118. Illustration. SPT results for soil near Bowman Road.

The blow count of the foundation soil can be correlated into an undrained shear strength using published guidance.[34] For blow counts greater than 30 blows/ft, the unconfined compressive strength is greater than 8,000 lb/ft^2. The undrained shear strength is therefore at least 4,000 lb/ft^2. The important properties for the foundation soil are shown in table 23. The retained backfill consists of the same material as the foundation soil, as shown in table 24.

Table 23. Design example 2—foundation soil properties.

Property	Symbol	Value
Unit weight of the foundation soil (lb/ft^3)	γ_f	120
Effective unit weight of the foundation soil (lb/ft^3)	γ'_f	77.6
Friction angle of the foundation soil (degrees)	ϕ_f	28
Effective cohesion of the foundation soil (lb/ft^2)	c'_f	500

Table 24. Design example 2—retained backfill soil properties.

Property	Symbol	Value
Unit weight of the retained backfill (lb/ft^3)	γ_b	120
Friction angle of the retained backfill` (degrees)	ϕ_b	28
Cohesion of the retained backfill (lb/ft^2)	c_b	4,000

The road base is a granular fill material that was brought to the site for the integrated approach as well as the RSF. For the Bowman Road Bridge project, the properties of the road base are given in table 25.

Table 25. Design example 2—road base (integrated approach and RSF) soil properties.

Property	Symbol	Value
Unit weight of the road base (lb/ft^3)	γ_{rb}	140
Friction angle of the road base (degrees)	ϕ_{rb}	40
Cohesion of the road base (lb/ft^2)	c_{rb}	0

The reinforced fill for the GRS abutment is a select granular fill (AASHTO No. 89 stone).[12] Testing was performed on this fill to determine the friction angle and cohesion parameters. The properties of this fill are provided in table 26.

Table 26. Design example 2—reinforced fill properties.

Property	Symbol	Value
Unit weight of the reinforced fill (lb/ft^3)	γ_r	110
Maximum diameter of the reinforced fill (inches)	d_{max}	0.5
Friction angle of the reinforced fill (degrees)	ϕ_r	48
Cohesion of the reinforced fill (lb/ft^2)	c_r	0

C.2.3 Step 3—Evaluate Project Feasibility

As mentioned in step 2, scour was not a significant concern for this bridge. Scour protection was added as a precaution; the riprap was sized for 8.8 to 10.2 ft/s to create a scour protection apron adjacent and in front of the abutment face and wing walls. Prior to placement, a 5- to 8-ft-wide strip of geotextile reinforcement between the face of the RSF and the riprap was pinned under the first course of facing blocks to secure it in place and to create the riprap apron.

No evidence of scour had been seen at the site with the old bridge in place even after sequential high flooding events. The project was therefore considered feasible for this site.

C.2.4 Step 4—Determine Layout of GRS-IBS

Steps to determine the layout of the GRS-IBS are as follows:

1. **Define the geometry of the abutment face wall and wing walls**: The height of the GRS abutments (H_{abut}) is 15.25 ft (not including the clear space). The distance between the two abutment faces is 72 ft.

2. **Lay out the abutment with respect to the superstructure, including any skew, superelevation, or grade requirements** (see figure 117**)**: Because the span length of the superstructure was greater than 25 ft, the minimum bearing width (b) for the superstructure was 2.5 ft. A bearing width of 4 ft, however, had been chosen for this bridge. No special design details are needed related to the skew, superelevation, or grade for this project.

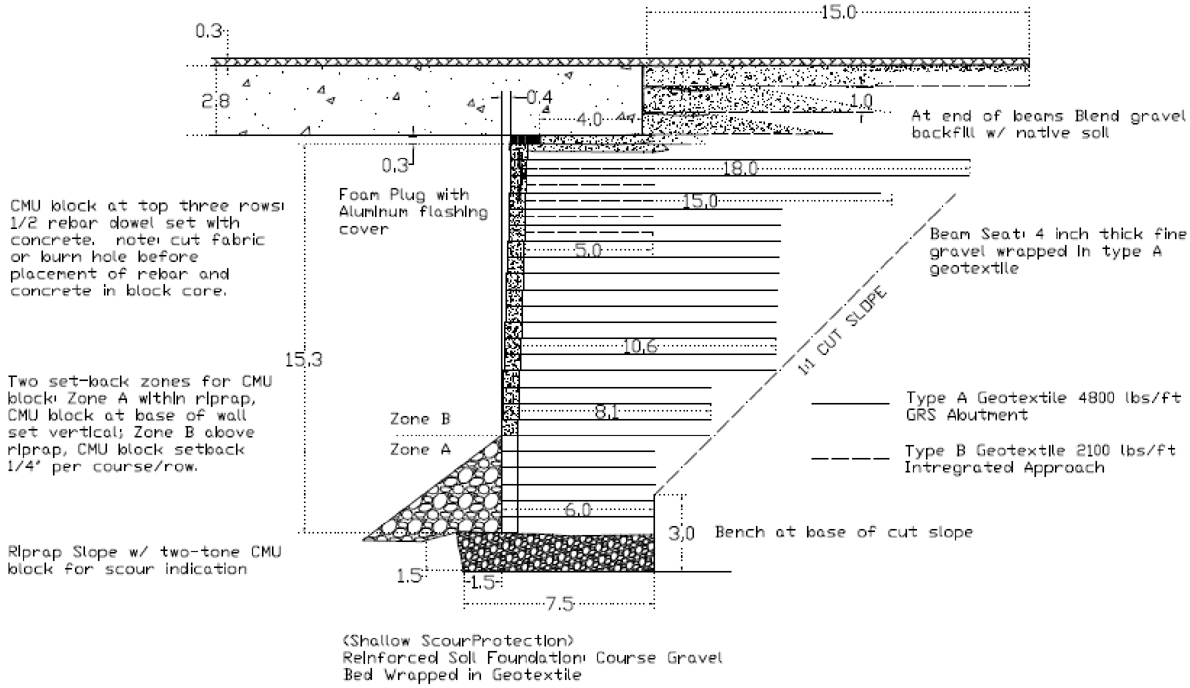
3. **Account for setback and clear space to calculate the elevation of the abutment face wall and the span length of the bridge**: The setback distance (α_b) is selected as 8 inches, and the clear space distance is 4 inches (which is greater than 2 percent of the wall height).

4. **Determine the depth and volume of excavation necessary for construction**: A base width of the wall (including the block face) of 6 ft was chosen for this abutment since the span length was greater than 25 ft and 0.3H is less than the 6-ft minimum. Subtracting the wall face width (7.625 inches), the reinforcement length at the base of the wall was 5.4 ft. This equates to a B/H of 0.35 (which is greater than the minimum B/H of 0.3). Excavation occurred at the base in front of the face of the wall of 1.5 ft (one-quarter the width, including the block face) to accommodate for construction of the RSF. The total width of the RSF was therefore 7.5 ft. The depth of the excavation for the RSF was selected as equal to one-quarter the width of the base (including the block face) or 1.5 ft.

5. **Select the length of reinforcement throughout the height of the abutment**: The reinforcement length at the base of the wall was equal to 6 ft (or 5.4 ft not including the reinforcement necessary for the frictional connection). The reinforcement lengths up the wall were chosen based on the cut slope angle and an optimization of the width of the reinforcement rolls. The reinforcement schedule is shown in figure 119.

6. **Add a bearing reinforcement zone underneath the bridge seat**: The primary reinforcement spacing was 7.625 inches at the wall face. The spacing of the bearing reinforcement bed was 3.81 inches (half of the primary spacing). The length of the bearing reinforcement bed was 5 ft. The depth of the bearing reinforcement bed will be determined when the internal stability analysis is conducted (see step 7 in section C.2.7). At a minimum, however, there should be three intermediate layers between the primary reinforcement layers (at 7.625-inch spacing).

7. **Place a beam seat above the bearing bed reinforcement zone to support the superstructure**: The beam seat extends up from the top of the abutment height by the distance of the clear space, or 4 inches. This means that for this project, the total height (H) is equal to 15.58 ft, which is the GRS abutment height (H_{abut}) of 15.25 ft plus the clear space distance of 0.33 ft. The reinforcement layers of the integrated approach behind the backwall of the superstructure will extend to the cut slope. Additional work is needed to integrate the substructure with the superstructure within the integration zone at the approach way. There are three layers of wrapped geotextile reinforcement spaced at 0.9 ft. The height of the integrated approach is 2.75 ft.



Copyright: Defiance County, OH.
Note: Units of measure are in feet.

Figure 119. Illustration. Reinforcement schedule and RSF dimensions for Bowman Road Bridge.

C.2.5 Step 5—Calculate Applicable Loads and Pressures

The applicable surcharges and loads associated with the structure are a combination of vertical and lateral components. The vertical components include the surcharges due to the DLs (superstructure and road base from the integrated approach) and the LLs (superstructure and roadway), along with the weight of the GRS abutment. The lateral earth pressure due to the retained backfill (see table 27) must also be considered. Lateral loads resulting from the DLs and LLs are calculated separately during the external and internal stability calculations.

Note that the weight of the GRS abutment is calculated with *B* equal to the shortest reinforcement layer (not including the width of the wall face). This is a conservative assumption to simplify hand calculations. Note that several software programs are available that can account for the varying shape due to different reinforcement lengths along the height of the abutment.

Table 27. Design example 2—loads and surcharges for Bowman Road Bridge.

Loads and Pressures	Value	Notes
Bridge DL (q_b) (lb/ft^2)	2,600	Given
Bridge LL (q_{LL}) (lb/ft^2)	1,400	Given
Roadway LL (q_t) (lb/ft^2)	298	$q_t = h_{eq}\gamma_b$ Where: h_{eq} = 2.48 ft.(see 29) γ_b = 120 lb/ft^3.
Road base DL(q_{rb}) (lb/ft^2)	385	$q_{rb} = H_{rb}\gamma_{rb}$ Where: H_{rb} = 2.75 ft. γ_{rb} = 140 lb/ft^3.
Weight of GRS abutment (W) (lb/ft)	9,186	$W = BH\gamma_r$ Where: B = 5.36 ft. H =15.58 ft. γ_r = 110 lb/ft^3.
Weight of RSF (W_{RSF}) (lb/ft)	1,575	$W_{RSF} = B_{RSF}D_{RSF}\gamma_{RSF}$ Where: B_{RSF} = 7.5 ft. D_{RSF} = 1.5 ft. γ_{RSF} = 140 lb/ft^3.
Weight of facing (W_{face}) (lb/ft)	774	$W_{face} = N_{block}\dfrac{W_{block}}{L_{block}}$ Where: N_{block} = 24. h_{block} = 7.625 inches. W_{block} = 42 lb. L_{block} = 15.625 inches.

C.2.6 Step 6—Conduct an External Stability Analysis

The external stability of a GRS-IBS is evaluated by looking at the following potential external failure mechanisms:

- Direct sliding (figure 25).
- Bearing capacity (figure 26).
- Global stability (figure 27).

C.2.6.1 Direct Sliding

Lateral translation, or direct sliding, must be resisted for stability. For an IBS, direct sliding shall be evaluated at both the interface between the GRS abutment and RSF and between the RSF and the foundation soils. For a standalone GRS abutment, direct sliding should be evaluated at the interface between the GRS abutment and the foundation soils.

C.2.6.1.1 Direct Sliding at the Base of the GRS Abutment

The driving forces on the GRS abutment comprise the lateral forces due to the retained backfill, the road base, and the traffic surcharge behind the abutment.

The nominal resultant force due to the retained backfill is calculated in equation 113 using the previously presented formula in equation 9.

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2 = \frac{1}{2}\left(120\,\frac{\text{lb}}{\text{ft}^3}\right)(0.36)(15.58\ \text{ft})^2 = 5{,}243\ \text{lb/ft} \tag{113}$$

The lateral force due to the road base and traffic surcharges are calculated in equation 114 and equation 115 using the previously presented formulae in equation 10 and equation 11, respectively.

$$F_{rb} = q_{rb} K_{ab} H = \left(385\,\frac{\text{lb}}{\text{ft}^2}\right)(0.36)(15.58\ \text{ft}) = 2{,}159\ \text{lb/ft} \tag{114}$$

$$F_t = q_t K_{ab} H = \left(298\,\frac{\text{lb}}{\text{ft}^2}\right)(0.36)(15.58\ \text{ft}) = 1{,}671\ \text{lb/ft} \tag{115}$$

The total factored driving force (F_R) is then calculated in equation 116 using the previously presented formula in equation 12.

$$\begin{aligned} F_R &= \gamma_{EH\,MAX}(F_b + F_{rb}) + \gamma_{LS} F_t = \\ &1.5\left(5{,}243\,\frac{\text{lb}}{\text{ft}} + 2{,}159\,\frac{\text{lb}}{\text{ft}}\right) + 1.75\left(1{,}671\,\frac{\text{lb}}{\text{ft}}\right) = 14{,}027\,\frac{\text{lb}}{\text{ft}} \end{aligned} \tag{116}$$

The factored resisting force (R_R) is a product of the sliding resistance factor (equal to 1.0), the total factored resisting weight and the friction factor between the base of the abutment and the RSF. The total factored resisting weight ($W_{T,R}$) is calculated in equation 117 using the previously presented formula from equation 14. Because of the batter setback of 0.4 ft in this project, the length of $b_{rb,t}$ is equal to 0.3 ft.

$$\begin{aligned} &W_{T,R} = \gamma_{EV\,MIN} W + \gamma_{DC\,MIN}(q_{DL} b) + \gamma_{DC\,MIN}(W_{face}) + \gamma_{EV\,MIN}(q_{rb} b_{rb,t}) = \\ &1.0(9{,}186\,\frac{\text{lb}}{\text{ft}}) + 0.9\left(2{,}600\,\frac{\text{lb}}{\text{ft}^2}\right)(4\ \text{ft}) + 0.9\left(774\,\frac{\text{lb}}{\text{ft}}\right) + 1.0\left(385\,\frac{\text{lb}}{\text{ft}^2}\right)(0.3\ \text{ft}) = 19{,}358\,\frac{\text{lb}}{\text{ft}} \end{aligned} \tag{117}$$

The friction force (μ) is equal to tanϕ_{crit}. There was no interface shear testing performed, so the friction factor is assumed to equal two-thirds times the tangent of the reinforced granular backfill friction angle (ϕ_r). Therefore, the factored resisting force (R_R) is then calculated in equation 118 using the previously provided formula in equation 13.

$$R_R = \Phi_\tau\left(W_{T,R}\mu\right) = 1.0\left[\left(19{,}358\,\frac{\text{lb}}{\text{ft}}\right)\left(\frac{2}{3}\right)\tan(48\text{ degrees})\right] = 14{,}333\,\frac{\text{lb}}{\text{ft}} \tag{118}$$

The factored resisting force is greater than the factored driving force, so the design satisfies this requirement, as shown in equation 119.

$$\frac{R_R}{F_R} = \frac{14{,}333\,\frac{\text{lb}}{\text{ft}}}{14{,}027\,\frac{\text{lb}}{\text{ft}}} = 1.02\ \geq 1.0 \tag{119}$$

C.2.6.1.2 Direct Sliding at the Base of the RSF

The driving forces on the GRS abutment (and RSF) comprise the lateral forces due to the retained backfill, the road base, and the traffic surcharge. The nominal resultant driving forces behind the GRS abutment and RSF from the retained backfill ($F_{b,RSF}$), the road base ($F_{rb,RSF}$), and the roadway LL surcharge ($F_{t,RSF}$) are determined along the height of the GRS abutment and depth of the RSF in equation 120 through 122 using the previously presented formulae in equation 17 through equation 19.

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2 = \frac{1}{2}\left(120\,\frac{\text{lb}}{\text{ft}^3}\right)(0.36)(15.58\text{ ft} + 1.5\text{ ft})^2 = 6{,}301\,\frac{\text{lb}}{\text{ft}} \tag{120}$$

$$F_{rb,RSF} = q_{rb} K_{ab}(H + D_{RSF}) = \left(385\,\frac{\text{lb}}{\text{ft}^2}\right)(0.36)(15.58\text{ ft} + 1.5\text{ ft}) = 2{,}367\,\frac{\text{lb}}{\text{ft}} \tag{121}$$

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF}) = \left(298\,\frac{\text{lb}}{\text{ft}^2}\right)(0.36)(15.58\text{ ft} + 1.5\text{ ft}) = 1{,}832\,\frac{\text{lb}}{\text{ft}} \tag{122}$$

The total factored driving force at the base of the RSF ($F_{R,RSF}$) is then calculated in equation 123 using the previously presented formula in equation 20.

$$\begin{aligned} &F_{R,RSF} = \gamma_{EH\,MAX}\left(F_{b,RSF} + F_{rb,RSF}\right) + \gamma_{LS}F_{t.RSF} = \\ &1.5\left(5{,}776\,\frac{\text{lb}}{\text{ft}}\ + 2{,}170\,\frac{\text{lb}}{\text{ft}}\right) + 1.75\left(1{,}680\,\frac{\text{lb}}{\text{ft}}\right) = 16{,}208\,\frac{\text{lb}}{\text{ft}} \end{aligned} \tag{123}$$

The factored resisting force including the RSF ($R_{R,RSF}$) is a product of the sliding resistance factor (equal to 1.0), the total factored resisting weight and the friction factor between the base of the abutment and the RSF (equation 13). The total factored resisting weight including the RSF ($W_{T,R,RSF}$) is calculated in equation 124 using the previously presented formula in equation 22.

$$W_{T,R,RSF} = W_{T,R} + \gamma_{EV\,MIN} W_{RSF} = 19{,}358\,\frac{\text{lb}}{\text{ft}} + 1.0\left(1{,}575\,\frac{\text{lb}}{\text{ft}}\right) = 20{,}933\,\frac{\text{lb}}{\text{ft}} \tag{124}$$

The friction force (μ) is equal to tanϕ_{crit}. Prior to placement of the RSF, a gravel mat using the reinforced backfill was placed as a working platform. There was no interface shear testing performed, so the friction factor is assumed to equal two-thirds times the tangent of that reinforced backfill friction angle (ϕ_r) before mobilization of sliding along the continuous reinforcement would occur. Therefore, the factored resisting force including the RSF ($R_{R,RSF}$) is then calculated in equation 125 using the previously presented formula in equation 21.

$$R_{R,RSF} = \Phi_\tau\left(W_{T,R,RSF}\mu_{RSF}\right) = 1.0\left[\left(20{,}933\,\frac{\text{lb}}{\text{ft}}\right)\left(\frac{2}{3}\right)\tan(48\text{ degrees})\right] = 15{,}499\,\frac{\text{lb}}{\text{ft}} \tag{125}$$

The factored resisting force is greater than the factored driving force at the interface between the RSF and the underlying foundation soils (equation 126), so the design satisfies this requirement.

$$\frac{R_{R,RSF}}{F_{R,RSF}} = \frac{15{,}499\,\frac{\text{lb}}{\text{ft}}}{16{,}208\,\frac{\text{lb}}{\text{ft}}} = 0.96 \leq 1.0 \tag{126}$$

While this check did not technically pass, it is very close to unity and therefore is assumed satisfactory. This is based on engineering judgment and the fact that the design is conservative because it assumes (1) no reinforced soil behind the base length of the GRS and (2) no material in front of the RSF, which is founded below the calculated depth of scour. In reality, the total factored driving force at the base of the RSF would be considerably less than that estimated using this simplistic analysis whereby the wedge of reinforced soil is neglected.

4.4.6.2 External Bearing Resistance

Before calculating the applied vertical bearing pressure ($\sigma_{v,base,R}$), the total factored vertical load (ΣV_R) must be calculated using the previously provided equation presented in equation 26 as follows:

$$\begin{aligned}\sum V_R =\ & \gamma_{EV\,MAX}(W) + \gamma_{EV\,MAX}(W_{RSF}) + \gamma_{DC\,MAX}(W_{face}) + \gamma_{LS}(q_t b_{rb.t}) + \\ & \gamma_{EV\,MAX}(q_{rb} b_{rb,t}) + \gamma_{DC\,MAX}(q_{DL} b) + \gamma_{LS}(q_{LL} b) = 1.35\left(9{,}186\,\frac{\text{lb}}{\text{ft}}\right) + \\ & 1.35\left(1{,}575\,\frac{\text{lb}}{\text{ft}}\right) + 1.25\left(774\,\frac{\text{lb}}{\text{ft}}\right) + 1.75\left(298\,\frac{\text{lb}}{\text{ft}^2}\right)(0.3\text{ ft}) + 1.35\left(385\,\frac{\text{lb}}{\text{ft}^2}\right) \\ & (0.3\text{ ft}) + 1.25\left(2{,}600\,\frac{\text{lb}}{\text{ft}^2}\right)(4\text{ ft}) + 1.75\left(1{,}400\,\frac{\text{lb}}{\text{ft}^2}\right)(4\text{ ft}) = 38{,}607\,\frac{\text{lb}}{\text{ft}}\end{aligned} \tag{127}$$

In addition, the eccentricity of the resulting force at the base of the wall ($e_{B,R}$) must be calculated (equation 27). To do this, the factored driving ($M_{D,R}$) and resisting moments ($M_{R,R}$) must be found in equation 128 and equation 129 using the formulae previously presented in equation 28 and equation 29, respectively.

$$\sum M_{D,R} = \gamma_{EH\,MAX} F_{b,RSF}\left(\frac{H + D_{RSF}}{3}\right) + \gamma_{LS} F_{t,RSF}\left(\frac{H + D_{RSF}}{2}\right) + \gamma_{EH\,MAX} F_{rb,RSF}$$
$$\left(\frac{H + D_{RSF}}{2}\right) = 1.50\left(6{,}301\ \frac{\text{lb}}{\text{ft}}\right)\left(\frac{15.58\text{ ft} + 1.5\text{ ft}}{3}\right) + 1.75\left(1{,}832\,\frac{\text{lb}}{\text{ft}}\right)$$
$$\left(\frac{15.58\text{ ft} + 1.5\text{ ft}}{2}\right) + 1.50\left(2{,}367\ \frac{\text{lb}}{\text{ft}}\right)\left(\frac{15.58\text{ ft} + 1.5\text{ ft}}{2}\right) = 111{,}511\,\frac{\text{ft-lb}}{\text{ft}} \tag{128}$$

$$\sum M_{R,R} = \left(\gamma_{DC\,MAX} q_{DL} b + \gamma_{LS} q_{LL} b\right)\left[\left(\frac{b}{2} + a_b\right) - \left(\frac{B_{RSF}}{2} - x_{RSF} - b_{block}\right)\right] +$$
$$\left(\gamma_{LS} q_t b_{rb,t} + \gamma_{EV\,MAX} q_{rb} b_{rb,t}\right)\left(\frac{B_{RSF}}{2} - \frac{b_{rb}}{2}\right) + \gamma_{EV\,MAX} W\left(\frac{B_{RSF}}{2} - \frac{B}{2}\right) +$$
$$\gamma_{DC\,MAX} W_{face}\left(B + \frac{b_{block}}{2} - \frac{B_{RSF}}{2}\right) = \left(1.25\left(2{,}600\,\frac{\text{lb}}{\text{ft}^2}\right)(4\text{ ft}) + 1.75\left(1{,}400\,\frac{\text{lb}}{\text{ft}^2}\right)(4\text{ft})\right)$$
$$\left[\left(\frac{4\text{ ft}}{2} + 0.67\text{ ft}\right) - \left(\frac{7.5\text{ ft}}{2} - 1.5\text{ ft} - 0.635\text{ ft}\right)\right] +$$
$$\left(1.75\left(298\,\frac{\text{lb}}{\text{ft}^2}\right)(0.3\text{ ft}) + 1.5\left(385\,\frac{\text{lb}}{\text{ft}^2}\right)(0.3\text{ ft})\right)\left(\frac{7.5\text{ ft}}{2} - \frac{0.3\text{ ft}}{2}\right) +$$
$$1.35\left(9{,}186\,\frac{\text{lb}}{\text{ft}}\right)\left(\frac{7.5\text{ ft}}{2} - \frac{5.36\text{ ft}}{2}\right) + 1.25\left(774\,\frac{\text{lb}}{\text{ft}}\right)$$
$$\left(5.36\text{ ft} + \frac{0.635\text{ ft}}{2} - \frac{7.5\text{ft}}{2}\right) = 40{,}375\ \frac{\text{ft-lb}}{\text{ft}} \tag{129}$$

The eccentricity is then calculated using the previously provided formula in equation 27 as follows:

$$e_{B,R} = \frac{\sum M_{D,R} - \sum M_{R,R}}{\sum V_R} = \frac{111{,}511\text{ ft-lb/ft} - 40{,}375\text{ ft-lb/ft}}{38{,}607\ \frac{\text{lb}}{\text{ft}}} = 1.8\text{ ft} \tag{130}$$

The vertical pressure is a result of the weight of the GRS abutment, RSF, and integrated approach, along with the bridge seat load and traffic surcharge. This is solved for in equation 131 using the previously presented formula in equation 25.

$$\sigma_{v,base,R} = \frac{\sum V_R}{B_{RSF} - 2e_{B,R}} = \frac{38{,}607\ \frac{\text{lb}}{\text{ft}}}{7.5\text{ ft} - 2(1.8\text{ ft})} = 9{,}899\ \frac{\text{lb}}{\text{ft}^2} \tag{131}$$

The bearing resistance is then calculated in equation 132 using the previously provided formula in equation 30. The bearing capacity factors, N_c, N_γ, and N_q, were found using table 9 for the foundation friction angle of 28 degrees.

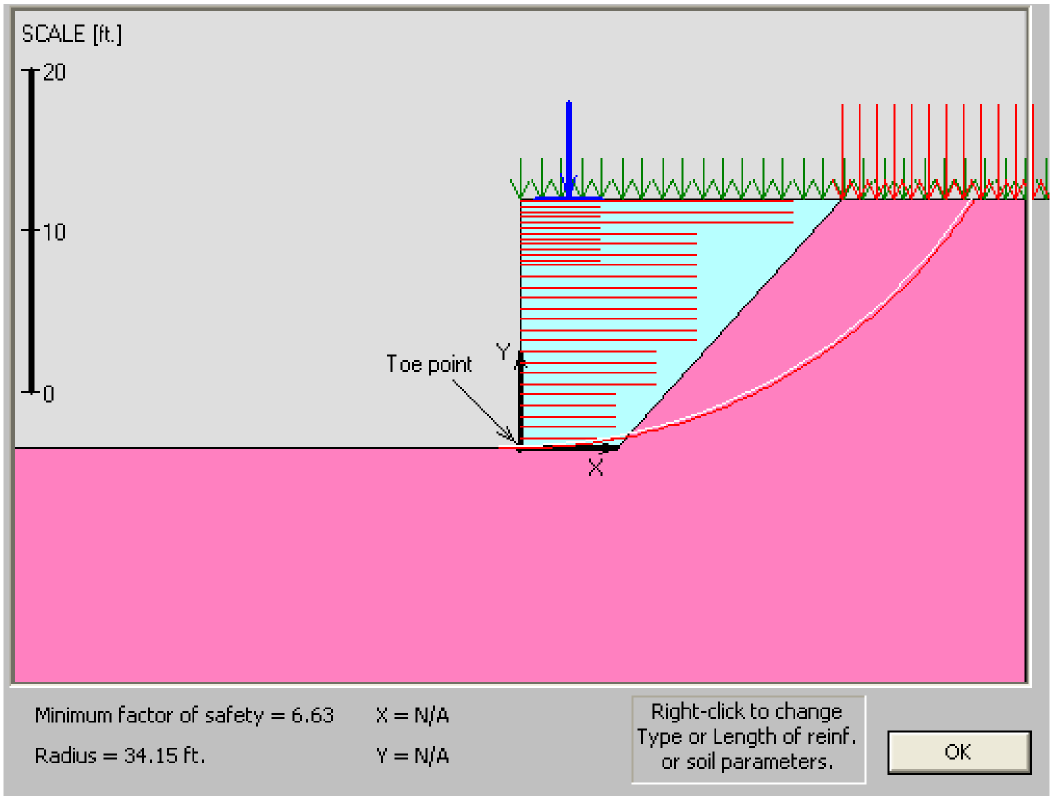
$$q_R = \Phi_{bc}\left(c_f^{'}\mathrm{N}_c + \frac{1}{2}B^{'}\gamma_f^{'}N_\gamma + \gamma_f^{'}D_f N_q\right)$$
$$= 0.65\left[\left(500\frac{lb}{ft^2}\right)(25.8) + \frac{1}{2}(7.5ft - 2(1.8ft))\left(57.6\frac{lb}{ft^3}\right)(16.7) + 57.6\frac{lb}{ft^3}(1.5ft)(14.7)\right] = 10{,}430\frac{lb}{ft^2} \tag{132}$$

The ratio of the factored bearing resistance and the factored applied pressure (equation 133) must be greater than or equal to 1.0, which it is.

$$\frac{q_R}{\sigma_{v,base,\ R}} = \frac{10{,}430\ \frac{\mathrm{lb}}{\mathrm{ft}^2}}{9{,}899\frac{\mathrm{lb}}{\mathrm{ft}^2}} = 1.05 \geq 1.0 \tag{133}$$

C.2.6.3 Global Stability

Global and compound stability was checked using the software program ReSSA. A screenshot of the global stability failure mode is shown in figure 120. The factor of safety is found to equal 6.6, which is much greater than the minimum requirement of 1.5. Global and compound stability were satisfied.



Source: FHWA.

Figure 120. Screenshot. ReSSA results for global stability of Bowman Road Bridge.

C.2.7 Step 7—Conduct Internal Stability Analysis

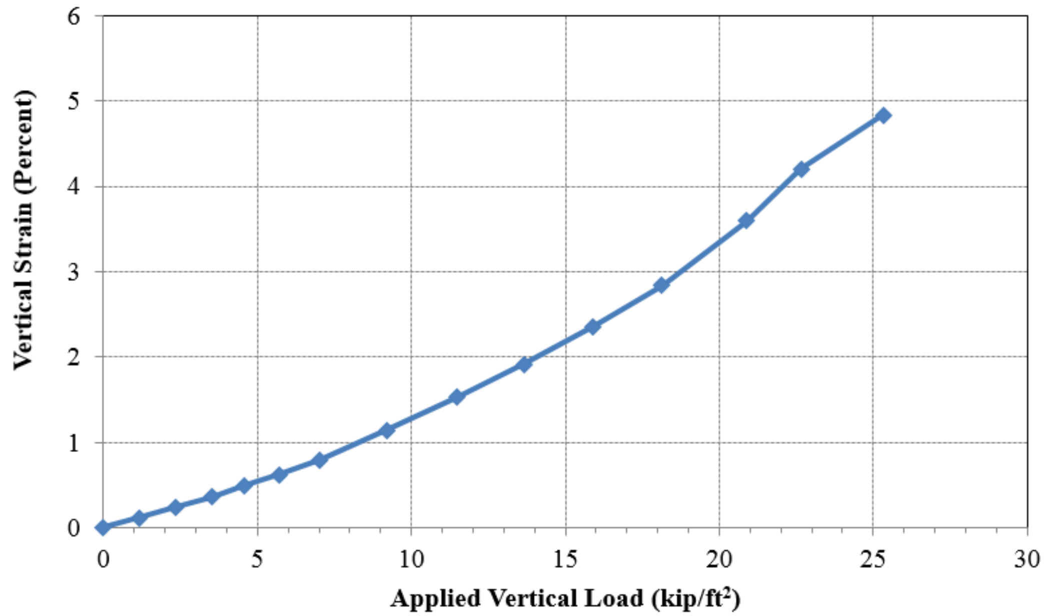
Internal stability for GRS includes ensuring adequate internal bearing resistance, tolerable deformations, and required reinforcement strength.

C.2.7.1 Internal Bearing Resistance

The bearing resistance of a GRS abutment can be determined using two different methods: empirical or analytical.

C.2.7.1.1 Empirical Method

The empirical method uses the load test results of a performance test on an identical (or very similar) GRS composite material to that used in the field. For this project, a performance test was conducted up to the limits of the facility (figure 121). While it did not reach failure, the ultimate value of approximately 26,000 lb/ft^2 was used as a check for internal bearing resistance.



Source: FHWA.

Figure 121. Graph. Stress–strain curve for Bowman Road Bridge showing ultimate bearing resistance.

The total allowable pressure on the GRS abutment is shown in equation 134.

$$V_{allow,emp} = \frac{q_{ult,emp}}{FS_{capacity}} = \frac{26{,}000\ \frac{\text{lb}}{\text{ft}^2}}{3.5} = 7{,}429\ \frac{\text{lb}}{\text{ft}^2} \tag{134}$$

Where:

$V_{allow,emp}$ = factored applied stress on top of the geosynthetic reinforced soil mass using the empirical method
$q_{ult,emp}$ = ultimate load-carrying capacity of GRS using the empirical method.

The applied vertical stress ($V_{applied}$) (calculated in equation 135 using the previously provided formula in equation 69), which is equal to the unfactored sum of the vertical pressures on the bridge bearing area, must be less than $V_{allow,emp}$ (from equation 134). This includes the DL from the bridge (q_b) and the LL due to the notional HL-93 load model (q_{LL}).

$$V_{applied} = q_{DL} + q_{LL} = 2{,}600\ \frac{\text{lb}}{\text{ft}^2} + 1{,}400\ \frac{\text{lb}}{\text{ft}^2} = 4{,}000\ \frac{\text{lb}}{\text{ft}^2} \leq 7{,}429\ \frac{\text{lb}}{\text{ft}^2} \tag{135}$$

C.2.7.1.2 Analytical Method

Alternatively, the internal bearing resistance was found analytically, where σ_c is assumed to equal 0, S_v equals 8 inches, d_{max} equals 0.5 inch (see table 20), T_f equals 4,800 lb/ft, and ϕ_r equals 48 degrees (table 20). Note that although the reinforcement spacing under the bridge bearing area is 4 inches, 8 inches was chosen in this calculation for the entire abutment to be conservative.

Equation 136 and equation 137 solve for the bearing resistance and the passive earth pressure coefficient using the previously presented formulae in equation 34 and equation 35, respectively.

$$q_{n,an} = \left[0.7^{\left(\frac{S_v}{6d_{max}}\right)} \frac{T_f}{S_v}\right] K_{pr} = \left[0.7^{\left(\frac{7.625 \text{ inches}}{6(0.5 \text{ inch})}\right)} \frac{4{,}800 \frac{\text{lb}}{\text{ft}}}{0.635 \text{ ft}}\right] 6.79 = 20{,}731 \ \frac{\text{lb}}{\text{ft}^2} \tag{136}$$

$$K_{pr} = \frac{1 + \sin\phi_r}{1 - \sin\phi_r} = \frac{1 + \sin(48 \text{ degrees})}{1\text{-}\sin(48 \text{ degrees})} = 6.79 \tag{137}$$

The applied factored vertical pressure on the abutment ($V_{applied,f}$) is solved for in equation 138 using the previously presented formula in equation 32.

$$V_{applied,f} = \gamma_{DC\,MAX} q_{DL} + \gamma_{LL} q_{LL} = 1.25\left(2{,}600 \frac{\text{lb}}{\text{ft}^2}\right) + 1.75\left(1{,}400 \frac{\text{lb}}{\text{ft}^2}\right) = 5{,}700 \tag{138}$$

The factored applied pressure is less than the factored bearing resistance (equation 139), so the GRS abutment as designed can support the applied loads.

$$\frac{\Phi_{cap}\left(q_{n,an}\right)}{V_{applied,f}} = \frac{0.45\left(20{,}731 \ \frac{\text{lb}}{\text{ft}^2}\right)}{5{,}700 \frac{\text{lb}}{\text{ft}^2}} = 1.6 \ \geq 1.0 \tag{139}$$

C.2.7.2 Deformations

The following subsections provide information on both vertical and lateral deformations.

C.2.7.2.1 Vertical

The performance test results for the same GRS composite were used to estimate vertical settlement of the GRS abutment. The vertical strain is estimated by using figure 121 for the bridge DL (q_{DL}) of 2,600 lb/ft^2. The vertical strain is therefore about 0.3 percent. Note that the road base surcharge is not included since it does not act over the same location.

The vertical deformation is the product of the vertical strain and the height of the GRS abutment, as shown in equation 140.

$$D_v = \varepsilon_v H = 0.003(15.58 \text{ ft}) = 0.047 \text{ ft} = 0.6 \text{ inch} \tag{140}$$

C.2.7.2.2 Lateral

The lateral strain and deformation are calculated in equation 141 and equation 142, respectively, which are solved based on the previously presented formulae in equation 39 and equation 38, respectively.

$$\varepsilon_L = 2\varepsilon_v = 2(0.3 \text{ percent}) = 0.6 \text{ percent} \tag{141}$$

$$D_L = \frac{2D_v}{H}(b + a_b) = \frac{2(0.046 \text{ ft})}{15.58 \text{ ft}}(4 \text{ ft} + 0.67 \text{ ft}) = 0.028 \text{ ft} = 0.3 \text{ inch} \tag{142}$$

C.2.7.3 Required Reinforcement Strength

The ultimate strength of the reinforcement used in the project is 4,800 lb/ft. According to the manufacturer, $T_{@\varepsilon = 2\%}$ is equal to approximately 1,371 lb/ft.

C.2.7.3.1 Strength Limit

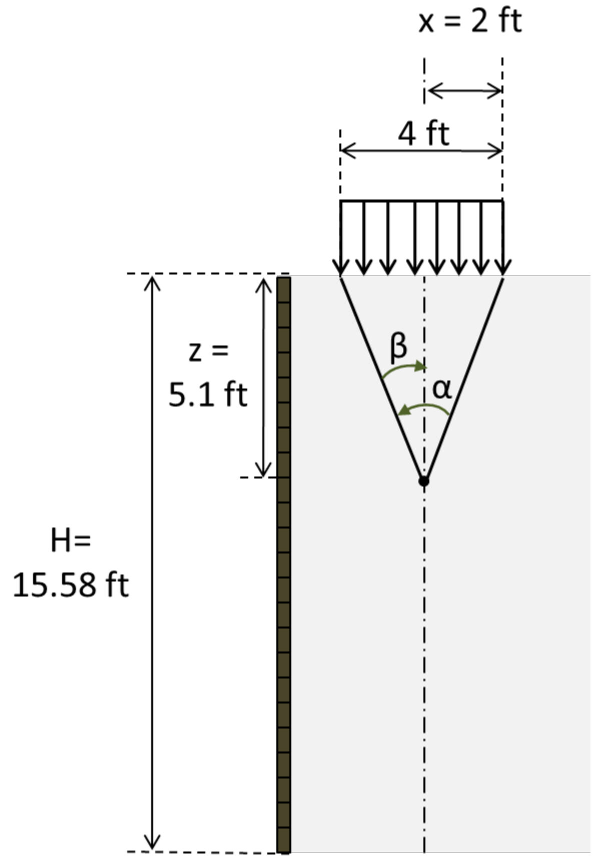
The factored maximum required reinforcement strength is found as a function of depth, reinforcement spacing, and maximum aggregate size (equation 40). The factored total lateral pressure ($\sigma_{h,f}$) is a combination of the factored lateral pressures due to the equivalent bridge load ($\sigma_{h,bridge,f}$), the road base surcharge ($\sigma_{h,rbf}$), the traffic surcharge ($\sigma_{h,tf}$), and the GRS reinforced soil ($\sigma_{h,Wf}$). The lateral stress is calculated for each depth of interest (each layer of reinforcement). All lateral stressesare calculated and shown in table 28.

Table 28. Design example 2—required reinforcement strength calculations.

	Equivalent Bridge Load				Road Base DL and Approach LL (lb/ft^2)				GRS Abutment Fill (lb/ft^2)		Total Lateral Pressure Service (lb/ft^2)		Strength Check		2 Percent Service Check	
z (ft)	α	β	$\sigma_{h,bridge}$ (lb/ft^2)	$\sigma_{h,bridge,f}$ (lb/ft^2)	$\sigma_{h,rb}$	$\sigma_{h,rb,f}$	$\sigma_{h,t}$	$\sigma_{h,t,f}$	$\sigma_{h,W}$	$\sigma_{h,W,f}$	$\sigma_{h,f}$	σ_h	$T_{req,f}$ (lb/ft)	$T_{req,f} > T_{f,f}$ (Yes/No)	T_{req} (lb/ft)	$T_{req} > T_{@\ \varepsilon=2\%}$ (Yes/No)
0.6	2.53	–1.26	483	670	57	85	44	77	10	15	847	594	1,333	No	934	No
1.3	2.01	–1.00	454	629	57	85	44	77	21	31	822	575	1,293	No	904	No
1.9	1.62	–0.81	407	565	57	85	44	77	31	46	773	539	1,216	No	848	No
2.5	1.33	–0.67	359	498	57	85	44	77	41	62	721	500	1,135	No	787	No
3.2	1.12	–0.56	315	437	57	85	44	77	51	77	676	467	1,064	No	735	No
3.8	0.97	–0.48	278	386	57	85	44	77	62	93	641	441	1,008	No	693	No
4.4	0.85	–0.42	248	344	57	85	44	77	72	108	614	421	966	No	662	No
5.1	0.75	–0.37	223	309	57	85	44	77	82	124	594	406	935	No	638	No
5.7	0.67	–0.34	202	280	57	85	44	77	93	139	581	395	913	No	621	No
6.4	0.61	–0.30	184	255	57	85	44	77	103	154	572	388	899	No	610	No
7.0	0.56	–0.28	169	234	57	85	44	77	113	170	566	383	891	No	602	No
7.6	0.51	–0.26	156	217	57	85	44	77	124	185	564	380	887	No	598	No
8.3	0.48	–0.24	145	201	57	85	44	77	134	201	564	380	887	No	597	No
8.9	0.44	–0.22	135	188	57	85	44	77	144	216	566	380	890	No	598	No
9.5	0.41	–0.21	127	176	57	85	44	77	154	232	570	382	896	No	601	No
10.2	0.39	–0.19	119	166	57	85	44	77	165	247	575	385	904	No	605	No
10.8	0.37	–0.18	113	156	57	85	44	77	175	263	581	388	914	No	611	No
11.4	0.35	–0.17	107	148	57	85	44	77	185	278	588	393	925	No	618	No
12.1	0.33	–0.16	101	140	57	85	44	77	196	294	596	398	937	No	625	No
12.7	0.31	–0.16	96	134	57	85	44	77	206	309	604	403	951	No	634	No
13.3	0.30	–0.15	92	128	57	85	44	77	216	324	614	409	966	No	643	No
14.0	0.28	–0.14	88	122	57	85	44	77	227	340	624	415	981	No	653	No
14.6	0.27	–0.14	84	117	57	85	44	77	237	355	634	422	997	No	663	No
15.3	0.26	–0.13	81	112	57	85	44	77	247	371	645	428	1,014	No	674	No

An example calculation for the required reinforcement strength at a depth (z) of 5.1 ft or reinforcement layer number 8 from the top of the abutment (figure 122) is shown in equation 143, which is solved based on the formula previously presented in equation 41. First, the lateral pressure must be found. Remember, the location of interest is directly under the centerline of the bridge load, where $x = 0.5b = 0.5(4 \text{ ft}) = 2$ ft.

$$\sigma_{h,f} = \sigma_{h,W,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f} = 124\frac{\text{lb}}{\text{ft}^2} + 311\frac{\text{lb}}{\text{ft}^2} + 87\frac{\text{lb}}{\text{ft}^2} + 78\frac{\text{lb}}{\text{ft}^2} = 600\ \frac{\text{lb}}{\text{ft}^2} \quad (143)$$



Source: FHWA.

Figure 122. Illustration. Lateral pressure due to the bridge load.

The lateral pressure resulting from the weight of the GRS abutment is found in equation 144 using the previously presented formula in equation 42.

$$\sigma_{h,W,f} = \gamma_{EH\ MAX}(\gamma_r z K_{ar}) = 1.5\left[110\frac{\text{lb}}{\text{ft}^3}(5.1\text{ft})\left(\frac{1 - \sin(48 \text{ degrees})}{1 + \sin(48 \text{ degrees})}\right)\right] = 124\ \frac{\text{lb}}{\text{ft}^2} \quad (144)$$

To simplify calculations, the pressure due to an equivalent bridge load is found in equation 145 using the formula presented in figure 43 by assuming the surcharges due to the road base and the traffic uniformly extend to the edge of the GRS abutment (and thus must be subtracted out).

$$\sigma_{h,bridge,f} = \frac{(\gamma_{DCMAX}\, q_{DL} + \gamma_{LL} q_{LL}) - (\gamma_{EHMAX}\, q_{rb} + \gamma_{LS} q_t)}{\pi}[\alpha_b + sin(\alpha_b)cos(\alpha_b + 2\beta_b)]K_{ar}$$

$$= \left[\frac{\left(1.25\left(2{,}600\frac{lb}{ft^2}\right) + 1.75\left(1{,}400\frac{lb}{ft^2}\right)\right) - 1.5\left(385\frac{lb}{ft^2}\right) + 1.75\left(298\frac{lb}{ft^2}\right)}{\pi}\right][0.74\, radians$$

$$+ \sin(0.74 radians)\cos(0.74 radians + 2(-0.37\, radians))\,(0.15)] = 311\frac{lb}{ft^2} \tag{145}$$

The values for α and β were found using equation 146 and equation 147 based on the previously presented formulae in equation 46 and equation 47, respectively.

$$\alpha_b = \tan^{-1}\left(\frac{b}{2z}\right) - \beta_b = \tan^{-1}\left(\frac{4\text{ ft}}{2(5.1\text{ ft})}\right) - (-21.4\text{ degrees}) = 42.6\text{ degrees} = 0.74\text{ radians} \tag{146}$$

$$\beta_b = \tan^{-1}\left(\frac{-b}{2z}\right) = \tan^{-1}\left(\frac{-4\text{ ft}}{2(5.1\text{ ft})}\right) = -21.4\text{ degrees} = -0.37\text{ radians} \tag{147}$$

The lateral pressures due to the road base and traffic surcharges are computed using equation 148 and equation 149 based off the formulae presented previously in equation 44 and equation 45, respectively.

$$\sigma_{h,rb,f} = \gamma_{EH\,MAX} q_{rb} K_{ar} = 1.5\left(385\frac{\text{lb}}{\text{ft}^2}\right)(0.15) = 87\frac{\text{lb}}{\text{ft}^2} \tag{148}$$

$$\sigma_{h,t,f} = \gamma_{LS} q_t K_{ar} = 1.75\left(298\frac{\text{lb}}{\text{ft}^2}\right)(0.15) = 78\frac{\text{lb}}{\text{ft}^2} \tag{149}$$

The factored maximum required reinforcement strength ($T_{req,f}$) at this reinforcement level can then be found in equation 150 using the previously presented formula in equation 40.

$$T_{req,f} = \left[\frac{\sigma_{h,f}}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v = \left[\frac{600\frac{\text{lb}}{\text{ft}^2}}{0.7^{\left(\frac{7.625\text{ inches}}{6(0.5\text{ inch})}\right)}}\right] 0.635\text{ ft} = 943\ \frac{\text{lb}}{\text{ft}} \tag{150}$$

To determine the factored reinforcement strength ($T_{f,f}$), equation 110 is used based off the previously presented formula in equation 48, where $T_{f,f}$ equals 1,920 lb/ft. $T_{f,f}$ is then checked against $T_{req,f}$, as shown in equation 151, which satisfies the strength limit.

$$\frac{T_{f,f}}{T_{req,f}} = \frac{1{,}920\frac{\text{lb}}{\text{ft}}}{943\ \frac{\text{lb}}{\text{ft}}} = 2.0 \geq 1.0 \tag{151}$$

C.2.7.3.2 Service Limit

The check at the service limit is identical to that at the strength limit except no load factors are applied. At the same depth shown in the previous example (z = 4 ft), the required reinforcement strength at the strength limit is shown in equation 152 using the previously presented formula in equation 50.

$$T_{req} = \left[\frac{\sigma_h}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v = \left[\frac{403\,\frac{\text{lb}}{\text{ft}^2}}{0.7^{\left(\frac{7.625\text{ inches}}{6(0.5\text{ inch})}\right)}}\right] 0.635\text{ ft} = 634\ \frac{\text{lb}}{\text{ft}} \tag{152}$$

The required reinforcement strength at this depth is less than the manufacturer's supplied $T_{@\varepsilon = 2\%}$ of 1,371 lb/ft.

Based on table 28, the required reinforcement strength does not exceed the allowable strength or the strength at 2 percent at any reinforcement layer. Therefore, no bearing bed reinforcement is needed; however, the minimum requirement is that the bearing bed reinforcement should extend through three courses of blocks. In actuality, six courses of block were chosen to extend the bearing reinforcement bed in this case (to a depth of 4 ft below the top of the wall). This was chosen to be conservative since this was the first bridge built with GRS technology.

C.2.8 Step 8—Implement Design Details

All design details were considered. Since it is a skewed bridge, the bearing area of 3 ft was maintained along the length of the face wall. The bearing bed reinforcement schedule was also maintained across the abutment face due to the superelevation (figure 123).

Source: FHWA.

Figure 123. Illustration. Secondary reinforcement for superelevation at Bowman Road Bridge.

C.2.9 Step 9—Finalize Material Quantities and Layout

The amount of reinforcement necessary is based on the reinforcement schedule. Reinforcement material usually comes in 12- to 18-ft-wide rolls. The number of facing blocks is determined from the height and length of the abutment and wing walls. The amount of backfill required is determined in a similar fashion. Once the final quantities are established, it is a good rule of thumb to multiply by 10 percent as an added factor of safety.

APPENDIX D. DESIGN REQUIREMENTS FOR HYDRAULIC CONDITIONS

The following appendix is largely from appendix D of *Hydraulic Performance of Shallow Foundations for the Support of Vertical-Wall Bridge Abutments*.(60)

Shallow foundations (e.g., spread footings) have been successfully used for bridge abutments at river and stream crossings. However, when a shallow abutment foundation is being considered for use in a river or stream environment, it is vitally important to fully understand the hydraulic design requirements. [This guidance will ensure the resulting design will comply with standards and requirements of FHWA regulations 23 CFR 625, 650.115, 650.117 and 650.313.(61)] The guidance provided in this appendix identifies the major hydraulic requirements that, if followed, provide greater assurance that the shallow foundation will perform as intended. This guidance supersedes that currently provided in HEC-18, fifth edition, [HEC-20, 4th edition], and HEC-23, third edition.(34,5,14)

The design of bridges in a river environment is a very complex endeavor because of the complex interactions among the structural components, the soils in which they are founded, and the moving water that imparts hydraulic loading to both. For this reason, an interdisciplinary team of structural, geotechnical, and hydraulic engineers should always be fully engaged in the bridge scoping, design, and construction processes. From a strictly hydraulics perspective, it is of utmost importance that a qualified hydraulics engineer, experienced in river mechanics, sediment transport, and bridge hydraulics be part of the interdisciplinary team.

When bridges are constructed over a waterway, [FHWA regulation 23 CFR 625 (Design Standards) requires their foundations must be designed, detailed, and constructed in compliance with section 2.6 (Hydrology and Hydraulics) of the American Association of State Highway and Transportation Officials *Load Resistance Factor Design Bridge Design Specifications* [for National Highway System (NHS) projects] or an FHWA division office-approved drainage or bridge manual [for Non-NHS projects].(61, 29) In addition, the bridge impact on the numerous floodplain values that may be present must be evaluated in accordance with the U.S. Code of Federal Regulations (23 CFR Part 650 SubpartA).(61) These compliances are required for the project to be eligible for Federal assistance.

HYDRAULIC DESIGN CONSIDERATIONS

The minimum hydraulic design considerations that should be evaluated when deciding whether a shallow abutment foundation is appropriate for a waterway bridge follow:

- **Site selection:** The optimum stream-crossing site has a fully stable channel, which is characterized by banks and bed that are not prone to change. Detailed information on what constitutes a stable channel is contained in FHWA publication, HEC-20, *Stream Stability at Highway Structures*.(5)

- **Abutment location:** Bridge abutments are typically set back from stream channel banks to minimize potential stability problems, scour, and impact loads. Impact loads can be expected on streams that transport ice, large cobbles, boulders, or large woody debris such as tree trunks. It is recommended that abutments be set back from the channel banks a minimum distance of twice the design flow depth in the channel or 25 ft (7.6 m), whichever is less. Flow depth should be established by water surface modeling [i.e., software tools such as [HEC River Analysis System (HEC-RAS) or Sedimentation and River Hydraulics—Two-Dimensional (SRH-2D)].(62, 63)

- **Adverse flow conditions:** Adverse flow conditions generate complex hydraulics and increase the potential scour and stream instability at a bridge site. Adverse flow conditions result from bridges that have the following characteristics: (1) are highly skewed to the flow, (2) severely constrict the flow, (3) encroach on flows in steep channels, and (4) produce overtopping of the bridge or an approach roadway. Crossings with one or more of these adverse conditions must be evaluated with advanced hydraulic modeling techniques (two-dimensional modeling) to identify accurate flow depths and velocities at the necessary locations.

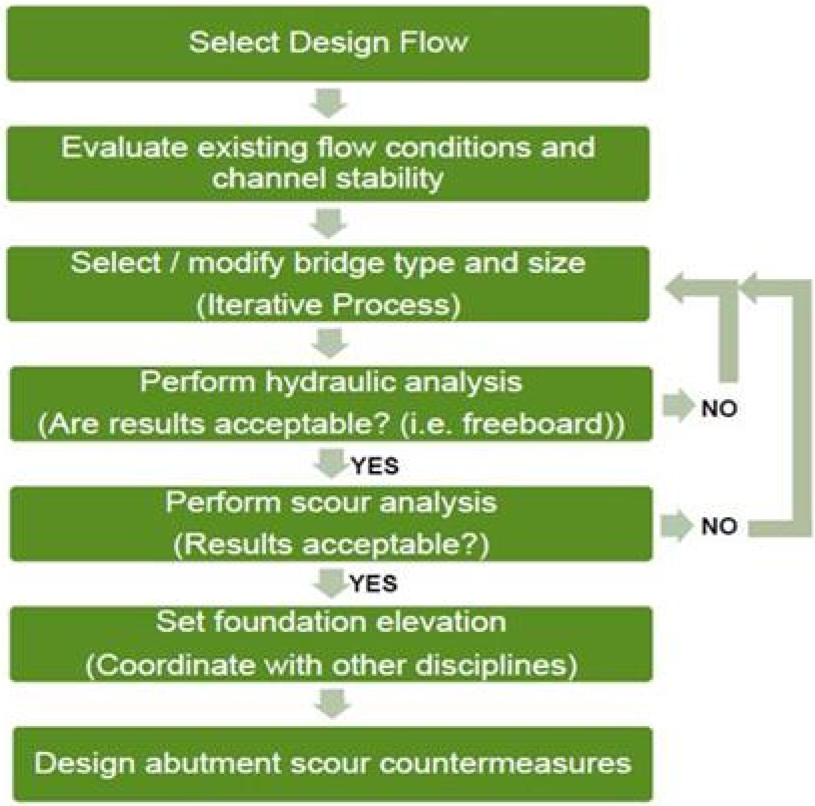
- **Risk-based standards:** [In accordance with statutory provisions of the *Moving Ahead for Progress in the 21st Century Act* (MAP-21)], risk-based standards have been adopted [by FHWA] so bridge owners can better balance the flood frequencies used for bridge design with the risks associated with the crossing (e.g., cost of the bridge, importance of bridge, traffic characteristics). Once the owner has identified the appropriate risk-based flood frequency standard for hydraulic design (i.e., bridge waterway capacity), FHWA has linked that flood frequency to recommended frequencies for the scour design and scour check floods. Reference chapter 2 of HEC-18 (i.e., *Evaluating Scour at Bridges)* for a discussion of risk-based standards and the recommended relationship between the hydraulic design and the scour floods.(34)

- **Local drainage:** The potential for unbalanced water pressure exists when an abutment wall can become partially submerged by a flood or when surface drainage is not controlled. All vertical-wall abutments should include provisions to accommodate surface and subsurface drainage. The following critical areas should be considered: behind the wall, at the base of the wall, and any location where a fill slope meets the wall face. For example, the design needs to include provisions for surface drainage along the fill slope adjacent to wing walls [section 7.11 discusses drainage details].

The listed descriptions of hydraulic design considerations reflect preferred conditions for shallow foundation abutments. The greater the departure from these preferred conditions, the more likely that alternative abutment types or drainage structure types (e.g., reinforced concrete box culverts or pipe culverts) should be considered. Under these circumstances, [FHWA regulation (23 CFR 650 A) requires analyses of such design alternatives with consideration given to capital costs and risks; economic, engineer, social and environmental concerns; and including risk assessments or risk analyses].(61)

HYDRAULIC DESIGN PROCESS

The bridge hydraulic design process applicable to shallow foundation abutments is illustrated in figure 124.



Source: FHWA.

Figure 124. Flowchart. Bridge hydraulic design process.

The steps in this multidisciplinary process are described as follows:

- **Select design flow:** A minimum of three flood frequencies must be evaluated for shallow foundation design: (1) hydraulic design flood, (2) scour design flood, and (3) scour check flood. The hydraulic design flood is used to identify the necessary size (i.e., length and elevation) and orientation of the bridge opening. The scour design and check floods, which are typically larger than the hydraulic design flood, are used to design scour countermeasures and determine the minimum depth of the spread footing foundation. The appropriate frequency for these floods is typically defined by the hydraulic standards established by the bridge owner. If such standards do not exist, the appropriated flood frequencies can be identified by conducting a risk assessment or analysis for the stream crossing. Reference chapter 2 of HEC-18 for detailed discussions of risk-based standards and the recommended relationship between the hydraulic design flood and the scour floods.(34)

- **Evaluate existing flow conditions and channel stability:** [The designer should always evaluate] existing flow conditions and patterns should always be evaluated to establish a hydraulic baseline for the new or replacement bridge design. Also, [as stated in the Site Selection Consideration above], stream channels should be stable, both horizontally and vertically, in the vicinity of the bridge to provide a suitable crossing environment for the life of the bridge. Details on the evaluation of channel stability are contained in HEC-20.(5) Details on stream instability countermeasure design can be found in HEC-23 (*Bridge Scour and Stream Instability Countermeasures)*.(14)

- **Select/modify bridge type and size:** As indicated in figure 124, this step is the beginning of an iterative process that evaluates the hydraulics and potential scour resulting from the alignment and grade of the approach roadways, as well as the size and orientation of the bridge. Proposed layouts of these elements must be hydraulically modeled for a range of discharges that includes the hydraulic design flood and the scour floods to accurately estimate the hydraulic parameters (e.g., depths and velocities) affecting the bridge, the approach roadways, and the floodplain.

- **Perform hydraulic analysis:** The hydraulic model used must be capable of developing water-surface profiles upstream, downstream, and through the bridge to identify reasonable estimates of the key hydraulic parameters. This requires that a one-dimensional water-surface profile model, such as HEC River Analysis System (i.e., HEC-RAS), be used at minimum.[62] If adverse flow conditions exist, or are created at the crossing, a two-dimensional model, such as the Sedimentation and River Hydraulics—Two-Dimensional (i.e., SRH-2D), should be used.[63] Also, if channel geometry can change over time, multiple hydraulic models may be needed to identify worst-case hydraulics and scour. The possible alignments, grades, and bridge geometries are evaluated with the hydraulic model(s) until an acceptable crossing configuration is found for the design floods. Refer to FHWA publication *Hydraulic Design of Safe Bridges* for detailed guidance on one- and two-dimensional hydraulic modeling.(64)

- **Perform scour analysis:** The hydraulics for floods up to and including the scour check flood standard are used to identify the worst-case scour depths for the applicable scour components and the total scour that can be generated at each bridge foundation. If a computed worst-case scour depth is not acceptable (i.e., too deep for the abutment to be economically built and/or protected), adjustments to the bridge type, size, or even location need to be made until an acceptable total scour value results. Such adjustments should be made only after in-depth consultations with the project geotechnical and structural engineers. Reference HEC-18 and the following for detailed guidance on conducting scour evaluations and analyses.(34)

- **Set foundation elevation:** There are two basic options for establishing the spread footing elevation: 1) the top of the footing may be set below the total scour depth for the scour check flood at the abutment, without the need for an abutment scour countermeasure, or 2) the top of the footing may be set at or below the contraction scour elevation for the scour design flood (includes any [long-term degradation] (LTD)), with a properly designed and constructed abutment scour countermeasure. As indicated in figure 124, the task of setting the final spread footing elevation can only be done effectively through in-depth consultation and coordination with the project geotechnical and structural engineers.

- **Design abutment scour countermeasures:** When it is not practical to set the abutment foundation below the total scour depth, a designed abutment scour countermeasure is required to ensure stability during the scour check flood. See the sections entitled Scour Design and Scour Countermeasure Design later in this appendix for more information on the design of scour countermeasures for shallow abutment foundations.

APPLICABLE SCOUR COMPONENTS

For shallow abutment foundation design, the following primary scour components must be computed and evaluated: (1) LTD, (2) contraction scour, and (3) abutment scour. Both the contraction scour and abutment scour components are sensitive to what sediment transport regime exists upstream of the bridge (i.e., live-bed versus clear-water) and whether the scour floods are under free-surface flow or pressure flow conditions (e.g., bridge girders are in the flow) at the bridge. Because of the dramatic increase in potential scour depth, pressure scour [PS] should be avoided, if at all possible. In addition, the abutment scour component is sensitive to the location of the abutment relative to the main channel. When the abutment is located close to the channel, the scour is computed differently than when it is some distance away (i.e. more than twice the main channel flow depth for the scour check flood).

The manner in which these individual scour components are evaluated for floods up to and including the scour check flood standard is summarized as follows by flow condition. [HEC-18 provides detailed guidance on how to compute scour components.(34)]

Free-Surface Flow

- LTD = Greater of the following two evaluations:
    - Computed depth from equilibrium slope or armoring analyses, based on HEC-20 guidance.(5)
    - A specified depth for other degradation phenomenon, such as head cut depth, or historical observation.
- Contraction scour [CS] (horizontal):
    - For clear-water conditions = Clear-water contraction scour estimate.
    - For live-bed conditions = Lesser of the following:
      – Live-bed contraction scour estimate.
      – Clear-water contraction scour estimate.
- Abutment scour [AS]:
    - Amplification factor, based on abutment location, multiplied by one of the following, as appropriate:
      – Clear-water contraction scour estimate.
      – Live-bed contraction scour estimate.

Pressure Flow

- LTD (same as for free-surface flow).
- Contraction scour (vertical) = Pressure scour.
- Abutment scour (component presently undefined for pressure flow; research in progress).

As indicated, the conditions and interaction of the scour components can be complicated and must be identified and analyzed by a qualified hydraulics engineer. Refer to HEC-18 for detailed definitions of the individual scour components and conditions that apply to abutment analysis and design, and for the various methods available to compute the scour magnitude for each component.(34)

SCOUR DESIGN

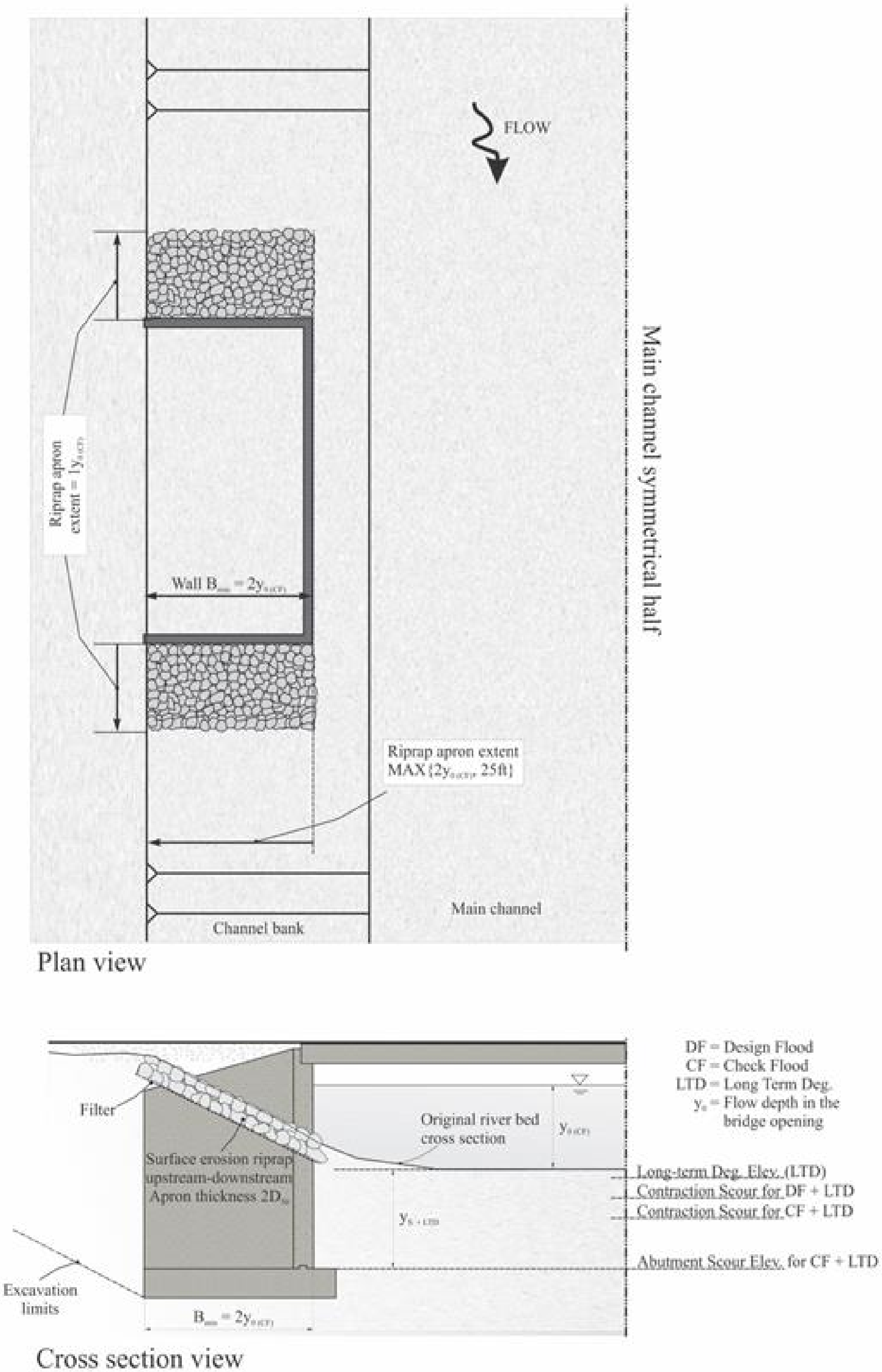
The scour used to establish the spread footing foundation elevation is the worst-case combination of applicable scour components estimated for floods up to and including the appropriate scour flood standard (e.g., [a storm-event having a probability of occurrence of one every 100 years [Q100]] for scour check flood), and depends on the flow condition (i.e., whether free-surface or pressure flow exists at the bridge). The manner in which the individual scour components are combined for scour design is summarized below for each flow condition and illustrated in figure 125 through figure 128. The combinations listed are for abutments located close to the channel bank. Note that the underlining is a reminder to the user that measurements for scour check flood should be used and not those for scour design flood.

Free-Surface Flow

- Option 1 (no countermeasure): Minimum depth to top of footing = Total scour at abutment = LTD + [AS] for the scour check flood (see figure 125).
- Option 2 (wide-opening countermeasure; for [the ratio of the bottom width of contracted section (W_2) and the depth of water in the bridge opening (y_0)] (W_2/y_0) > 6.2 only): Minimum depth to top of footing = LTD + [CS] depth for the scour design flood; minimum depth to top of abutment countermeasure apron = LTD + [CS] for scour check flood (see figure 126).
- Option 3 (narrow-opening countermeasure): Minimum depth to top of footing = LTD + [CS] depth for the scour design flood; full-width countermeasure protection required from abutment to abutment; top of full-width countermeasure below LTD + [CS] depth for scour check flood (figure 127).

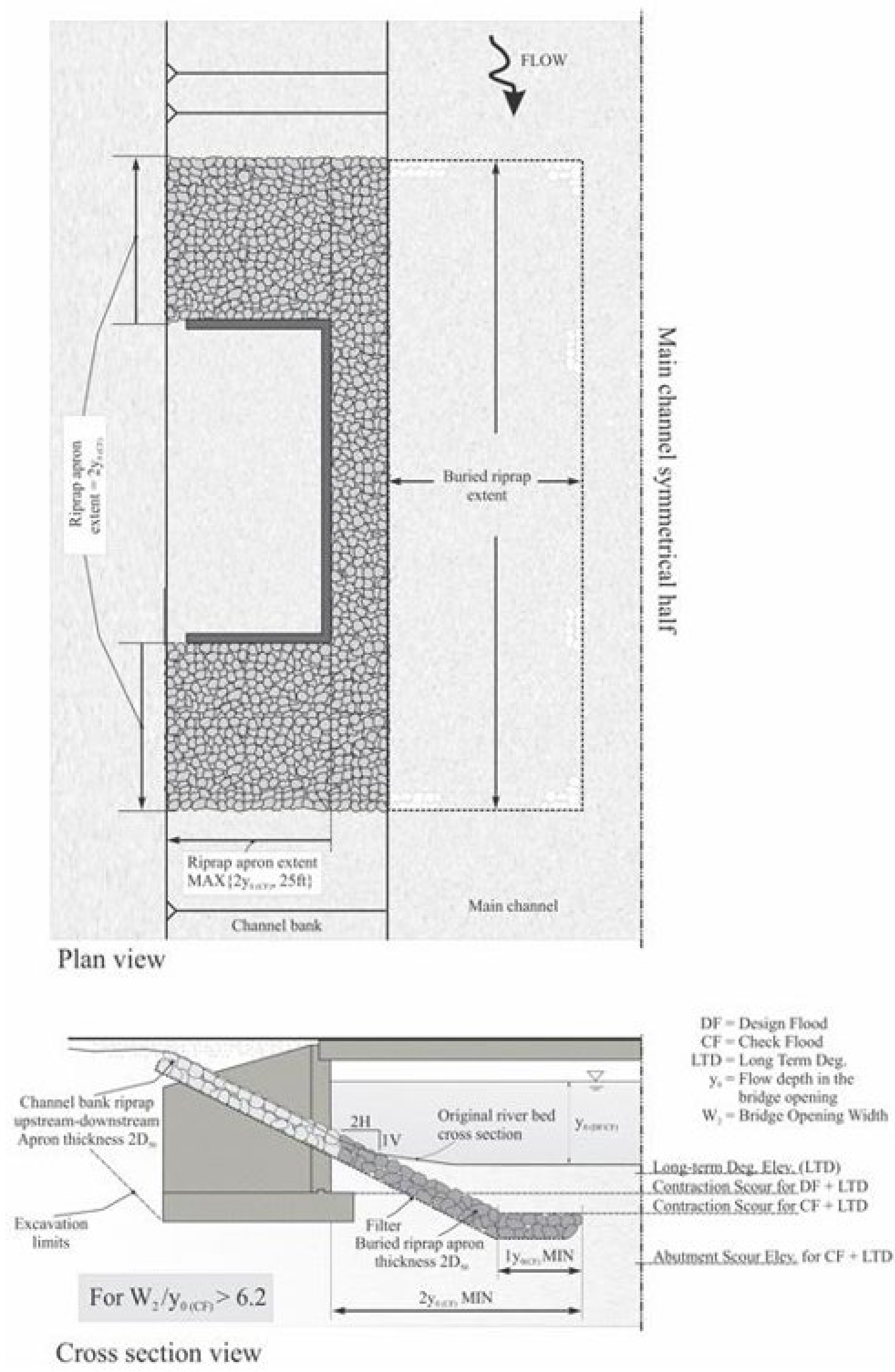
Pressure Flow

Pressure flow countermeasure: Minimum depth to top of footing = greater of LTD or [PS] depth for the scour design flood; full-width countermeasure protection required from abutment to abutment; top of full-width countermeasure below greater of LTD or [PS] depth for scour check flood (figure 128).



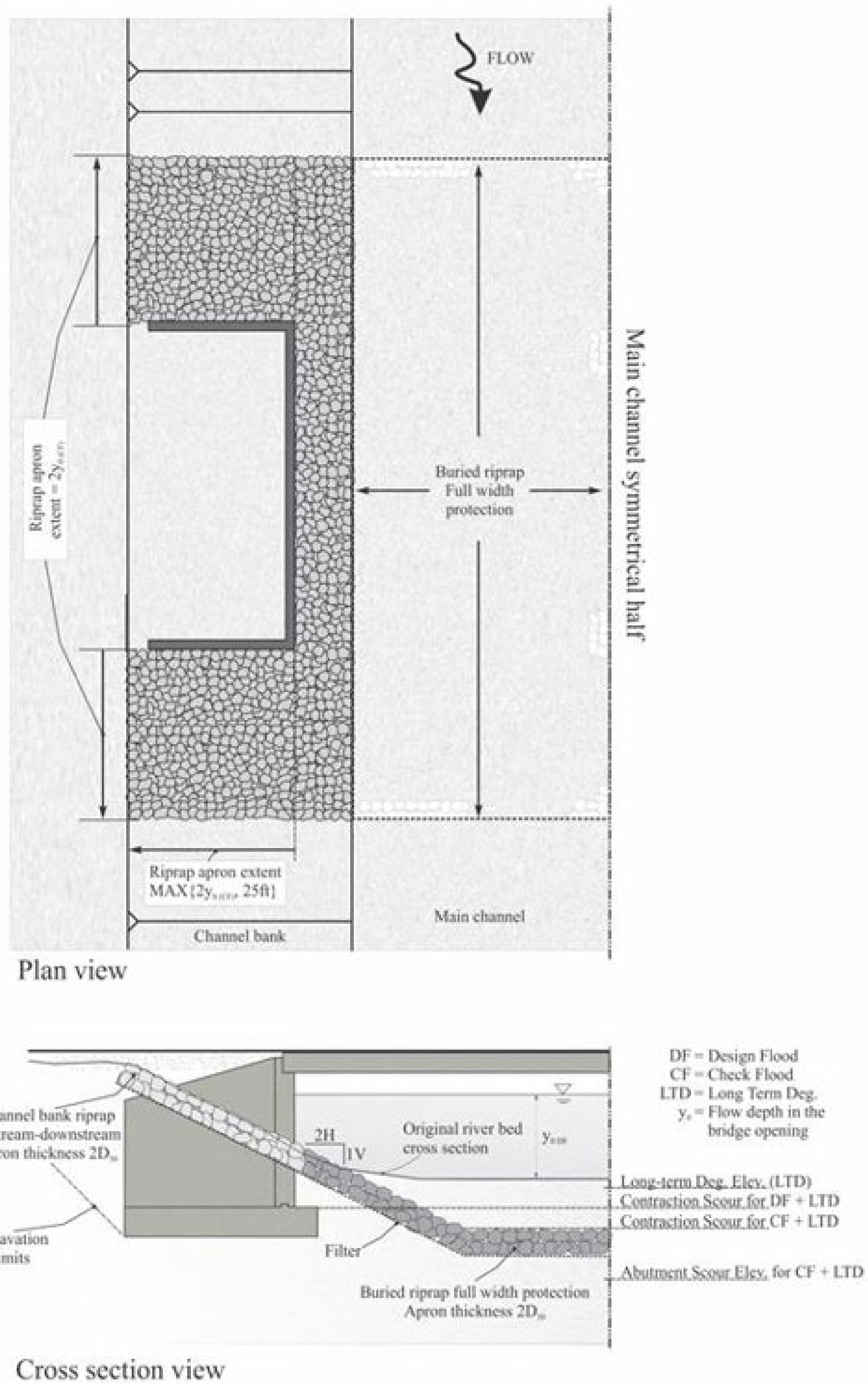
Source: FHWA.

Figure 125. Illustration. Free-surface flow with no scour countermeasure (option 1).



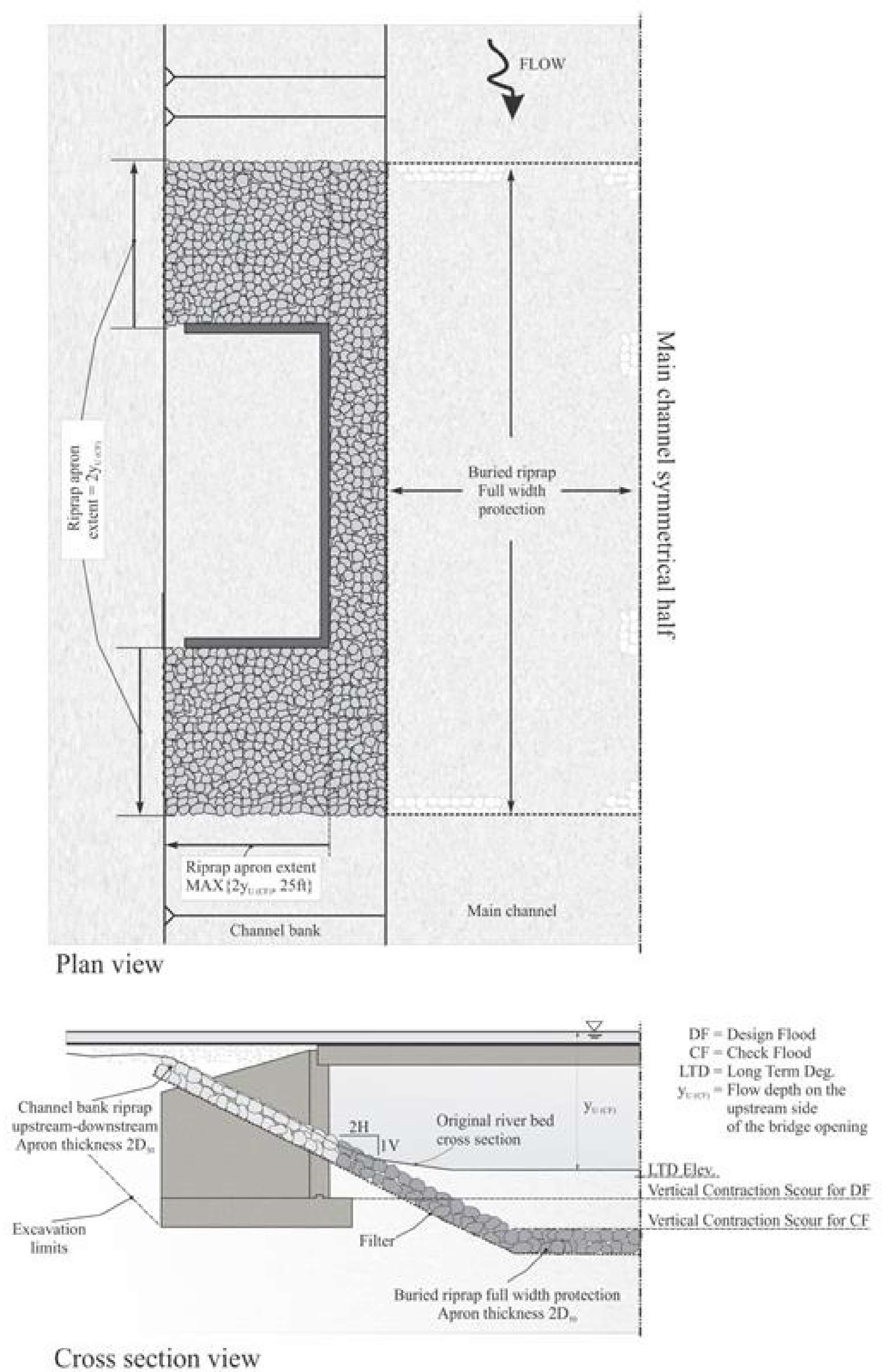
Source: FHWA.

Figure 126. Illustration. Free-surface flow with wide-opening scour countermeasure (option 2).



Source: FHWA.

Figure 127. Illustration. Free-surface flow with full-width scour countermeasure (option 3).



Source: FHWA.

Figure 128. Illustration. Pressure flow scour countermeasure.

It is important to note that scour depths must be tied to an appropriate reference elevation. The channel thalweg elevation should be used as the reference elevation for abutments located near the main channel.

SCOUR COUNTERMEASURE DESIGN

When a shallow abutment foundation requires installation of a scour countermeasure, the countermeasure must include a horizontal apron designed to be stable for the scour check flood. The apron should surround the entire abutment face and extend upstream and downstream of the abutment. For free-surface flow options 2 and 3, the extensions should be a distance equal to twice the main channel flow depth through the bridge ($2y_0$). For pressure flow, the extensions should be a distance equal to twice the main channel flow depth upstream of the bridge ($2y_u$). In addition, the same designed countermeasure should run up the channel bank and protect the abutment embankment. To do this effectively, the countermeasure should be configured to cover the embankment to an appropriate height (including freeboard) and for a distance of 2 times the main channel flow depth or 25 ft, whichever is greater, behind the abutment and parallel to the roadway.

Figure 125 through figure 128 illustrate the appropriate scour and countermeasure design configurations for the described flow conditions and options. Note that, although the countermeasure configurations are all similar, there are dimensional differences that make each case unique. Also note that the figures reflect the use of rock riprap as the countermeasure type. When riprap is used as the designed countermeasure, an appropriate filter must be placed under the riprap to prevent the underlying soils from being winnowed out through the interstitial openings between rocks.

There is one additional scour countermeasure design consideration for shallow foundations that support GRS and similar abutments. When the scour check flood overtops an approach roadway, a potential exists for the GRS abutment to be attacked from the back side. The likelihood depends on the depth of the overtopping flow, its duration, and the erosion resistance of the embankment. Research that defines the potential failure mechanism and most appropriate countermeasure configuration has yet to be conducted. However, the following additional countermeasure treatment is recommended when abutment embankment overtopping occurs for the worst-case scour condition:

- Wrap the geotextile layers on back side of abutment full height to enable the abutment to stand unsupported should the embankment fail.

- Extend the abutment embankment protection to the top of the embankment.

There is also one hydraulic/structural design consideration for GRS type abutments. When a scour flood inundates an abutment, either by free-surface overtopping flow or pressure flow conditions, the top of the GRS mass is subject to unknown hydrodynamic forces that may scour GRS material from around the beam seat. Should either of these flow conditions be present, it is recommended that the GRS abutment be constructed wide enough to accommodate placement of the scour countermeasure on top of the GRS mass.

Designing countermeasures for shallow foundation abutments in a river environment can also be a very complicated endeavor because of the complex interaction between the hydraulics, the multiple scouring mechanisms that are typically present, and the structural components. For these reasons, it is again of utmost importance that a qualified hydraulics engineer, experienced in river mechanics, sediment transport, and bridge hydraulics, perform the analyses required for countermeasure design.

At the time of this report, the complete details of scour countermeasure design specifically for shallow foundation abutments, which are too extensive to include herein, were being compiled and summarized for inclusion in the next edition of FHWA publication HEC-23.(14)

RIPRAP COUNTERMEASURE SPECIFICATIONS

As indicated in the preceding discussions, engineers must rely on countermeasures to ensure embankment, and at times, foundation stability during the scour check flood. Because of its flexibility, availability, and relative cost, the countermeasure of choice is often rock riprap. Accordingly, engineers must be aware that there are many sources of uncertainty associated with the design, manufacture, installation, performance, and maintenance of a riprap mass. Among the causes of premature riprap failure related to design and construction are the following:

- Inadequate rock quality, size, and/or gradation.
- Inadequate embedment and/or toe-down depths.
- Inadequate thickness.
- Segregation of rock sizes.
- No or improperly installed filter.
- Damaged filter material.

A designed granular or geotextile filter must be installed under all riprap installations to prevent finer-grained base soils from being winnowed out through the passages between individual rocks, causing premature failure.

Without comprehensive construction acceptance testing, there is little assurance that the riprap mass will perform as intended. Consequently, when a riprap countermeasure is used, rock quality, acceptance criteria, and testing frequency requirements must be developed and included in the construction contract specifications to properly control the manufacture, placement, and acceptance of the riprap. In addition, the size and gradation test methods to be used for accepting the riprap mass must be included in, or referenced by, the contract. Including such provisions in the contract will reduce the chances of premature riprap failure. The FHWA Office of Federal Lands Highways' *Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects*, FP-14, Sections 251 and 705, provide sampling, testing, and acceptance requirements; and material requirements, respectively, for rock riprap.(17)

After construction, riprap countermeasure condition and channel instability should be assessed during each regular bridge inspection and after large flood events. Any countermeasure failure or significant change in channel stability should be noted and scheduled for repair or stabilization. Without proper inspection and maintenance, a scour countermeasure may fail or a channel may become unstable, which can lead to bridge abutment failure.

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