

Keystone[®] DESIGN MANUAL & KEYWALL[™] OPERATING GUIDE



UNITS



GEOGRID



THEORY



DESIGN



KEYWALL[™]

Keystone® DESIGN MANUAL & KEYWALL™ OPERATING GUIDE

The information presented in this manual, including technical and engineering data, figures, tables, drawings, and procedures, is provided for general information only. While every effort has been made to ensure accuracy, this information should not be used or relied upon for any specific application without independent professional examination and verification of its accuracy, suitability, and applicability. Anyone using this material assumes any and all liability resulting from such use. Keystone Retaining Wall Systems disclaims any and all express or implied warranties of merchantability fitness for any general or particular purpose or freedom from infringement of any patent, trademark, or copyright in regard to the information or products contained or referred to herein. Nothing herein contained shall be construed as granting a license, express or implied, under any patents. Keystone is a registered trademark of Keystone Retaining Wall Systems, Inc. Keystone Retaining Wall Systems are covered by one or more U.S. Patents. Copyright © 1993, 1994, 1995, 2001 by Keystone Retaining Wall Systems, Inc. Printed in the United States.

Keystone® DESIGN MANUAL & KEYWALL™ OPERATING GUIDE



Keystone Retaining Wall Units



Geosynthetic Soil Reinforcement



Retaining Wall Design Theory



The Design Process



KeyWall Operating Instructions



Appendices



TABLE OF CONTENTS

| | |
|--|-----|
| Introduction..... | iv |
| Design Manual & KeyWall Operating Guide..... | v |
| Geotechnical Responsibility..... | vi |
| References..... | vii |



PART ONE | Keystone Retaining Wall Units

| | |
|---|-----|
| Keystone Retaining Wall Units..... | 1.1 |
| Keystone Materials..... | 1.2 |
| Standard Unit..... | 1.2 |
| Compac Unit..... | 1.3 |
| Keystone Century Wall Units..... | 1.3 |
| 133Elite Unit..... | 1.4 |
| Unit Shear Resistance..... | 1.4 |
| Base Shear Resistance..... | 1.4 |
| Inter-Unit Shear Resistance..... | 1.5 |
| Shear Data & Analysis..... | 1.5 |
| Typical Shear Resistance Between Units..... | 1.6 |



PART TWO | Geosynthetic Soil Reinforcement

| | |
|--|---------|
| Geosynthetic Soil Reinforcement..... | 2.1 |
| Design Strength..... | 2.2 |
| Reduction Factors..... | 2.3 |
| Connection Strength..... | 2.4-2.5 |
| Geosynthetic Soil Interaction Coefficient..... | 2.5-2.6 |
| Geogrid Manufacturers' Data..... | 2.6 |



PART THREE | Retaining Wall Design Theory

| | |
|---------------------------------------|-----------|
| Retaining Wall Design Theory..... | 3.1 |
| Important Technical Definitions..... | 3.2 |
| Lateral Earth Pressure Theories..... | 3.3 |
| Coulomb Earth Pressure Theory..... | 3.4 |
| Coulomb Earth Pressure Equation..... | 3.4-3.5 |
| Coulomb Failure Plane Location..... | 3.6 |
| Rankine Earth Pressure Theory..... | 3.6 |
| Rankine Earth Pressure Equations..... | 3.6-3.7 |
| Rankine Failure Plane Location..... | 3.7 |
| Trial Wedge Analysis..... | 3.8 |
| Bearing Capacity..... | 3.9 |
| Applied Bearing Pressure..... | 3.9 |
| Calculated Bearing Capacity..... | 3.10 |
| Bearing Capacity Factors..... | 3.10-3.11 |
| Settlement..... | 3.11 |
| Global Stability..... | 3.12 |
| Seismic Analysis..... | 3.13 |



TABLE OF CONTENTS



PART FOUR : The Design Process

| | |
|---------------------------------------|-----------|
| Introduction | 4.1 |
| Design Methodology..... | 4.2 |
| Unit Selection..... | 4.2 |
| Wall Batter..... | 4.3 |
| Hinge Height..... | 4.3 |
| Wall Geometry..... | 4.4 |
| Wall Embedment..... | 4.4 |
| Sloping Toe..... | 4.5 |
| Soil Properties..... | 4.5-4.6 |
| Surcharge..... | 4.6 |
| Reinforcement Type & Properties | 4.7 |
| Soil Reinforcement Length..... | 4.7 |
| External Stability Analysis..... | 4.8 |
| Battered Wall Design Options | 4.9 |
| AASHTO LRFD..... | 4.9-4.10 |
| Sliding Analysis..... | 4.11 |
| Coulomb - NCMA Sliding..... | 4.12 |
| Rankine - AASHTO Sliding..... | 4.12 |
| Overturning Analysis | 4.13 |
| Coulomb - NCMA Overturning | 4.14 |
| Rankine - AASHTO Overturning..... | 4.14 |
| Overturning | 4.14 |
| Bearing Capacity | 4.14 |
| Ultimate Bearing Capacity | 4.15 |
| Internal Stability..... | 4.15 |
| Tensile Capacity | 4.16 |
| Tension Level Calculation..... | 4.17 |
| AASHTO Internal Tension..... | 4.18 |
| Connection Capacity..... | 4.18 |
| Pullout Capacity | 4.19-4.20 |
| Stability of Facing | 4.20 |



PART FIVE : KeyWall Operating Instructions

| | |
|------------------------------------|----------|
| Installation & User's Guide..... | 5.1 |
| Installation | 5.2 |
| Registration..... | 5.3 |
| Keywall Installation & Setup | 5.4 |
| Launching the Keywall Program..... | 5.5 |
| Keywall Windows Interface..... | 5.6 |
| Keywall Interface Options..... | 5.7-5.8 |
| Keywall Registration Details..... | 5.8 |
| General Input..... | 5.9-5.10 |



TABLE OF CONTENTS



PART FIVE | KeyWall Operating Instructions *(continued)*

| | |
|--|-----------|
| Geometry Input..... | 5.10-5.11 |
| Soil Properties..... | 5.12 |
| Leveling Pad..... | 5.12 |
| Factors of Safety..... | 5.13 |
| LRFD Factors..... | 5.14 |
| Reinforcement Selection..... | 5.15-5.16 |
| Design Preferences..... | 5.16-5.17 |
| Results Screen..... | 5.18 |
| Edit Geogrid Data in Results Screen..... | 5.18 |
| Seismic Input..... | 5.19 |



PART SIX | Appendices Examples

| | |
|--|----------|
| Appendix A: Infinite Slope Surcharge NCMA 2nd Edition - Coulomb Methodology | A.1-A.5 |
| Appendix B: NCMA 2nd Edition - Coulomb Methodology Level Surcharge - 250 psf | B.1-B.10 |
| Appendix C: Rankine Methodology Level Surcharge - 250 psf..... | C.1-C.11 |
| Appendix D: 3H:1V Sloping Surcharge AASHTO Methodologies | D.1-D.15 |
| Appendix E: AASHTO LRFD Methodology Level Surcharge - 250 psf..... | E.1-E.11 |

Morris Corporate Park
Parsippany, New Jersey
Compac Classic Straightface
44,000 square feet



INTRODUCTION

The Keystone retaining wall system was created to provide an economical, easy-to-install, aesthetically appealing, and structurally sound system as an alternate to boulder, timber tie, concrete panel, or cast-in-place retaining walls. The Keystone system was initially conceived as a gravity wall system that could be constructed to heights of up to 6.5 feet (2 m). The original Keystone Standard unit was 2 feet (600 mm) from face to tail, providing weight and stability to resist the applied earth pressures. Later, the Keystone Compac unit was introduced, a smaller 1-foot (300 mm) deep unit. The units have the stability of a large mass, but are easier to handle, lighter to place, and quicker to install than boulders, crib structures or thin-shelled panel structures. Both units were designed with a structural pin connection and granular interlock, eliminating the need for grouting or mortar. Because of their structural strength with the fiberglass pins and granular drainage fill, the interlocked assembly is more stable than most other structures.

Concurrent with the development of the Keystone system, geosynthetic soil reinforcement was gaining approval and acceptance as a viable soil reinforcement material. With the structural pin and crushed stone fill for interlock, the combination of geogrids and Keystone units provides an integrated wall system that can be constructed to heights far exceeding the limits of simple gravity walls. Since 1986, millions of square feet of Keystone retaining walls have been successfully constructed, both as gravity and reinforced systems. Applications vary from residential landscaping walls to structural highway walls, some exceeding 50 feet (15 m) in height.

Wild Animal Park, San Diego, California; Keystone Compac





DESIGN MANUAL & KEYWALL OPERATING GUIDE

This manual concisely describes the retaining wall design components and related design theory based on accepted engineering principals and concepts discussed in the National Concrete Masonry Association (NCMA) *Design Manual for Segmental Retaining Walls, Second Edition and Third Edition* [Bernardi, et. al, 2009], The American Association of State Highway and Transportation Officials (AASHTO), *Standard Specifications for Highway Bridges*, and Federal Highway Administration (FHWA) design guidelines. Extensions, additions, or deviations from these methodologies are noted and explained.

It is important for the designer to understand that there are other design methodologies in use in the United States and around the world which will provide different results due to the simplifying design assumptions and methods of calculation utilized. Some use a Coulomb earth pressure analysis like NCMA, others use a Rankine earth pressure analysis. The 17th edition of the Standard Specifications for Highway Bridges published by AASHTO in 2002 recommends a “simplified” method of design in the allowable stress (ASD) format. Subsequent editions use load resistance factor design (LRFD), which is still based on the “simplified” method. FHWA has mandated all retaining walls be designed using LRFD after October 1, 2010.

It is our opinion that the NCMA design manual represents a comprehensive approach to Segmental Retaining Wall (SRW) design but tends to conflict in principal with existing methodologies such as those originally developed by the geogrid manufacturers and those contained in AASHTO design guidelines. The NCMA design manual recognizes the many technical nuances of segmental retaining wall design and provides needed criteria for proper engineering and design evaluation of modular systems. The more conservative AASHTO design standards remain the published standard for the transportation sector and covers many of the major structures constructed to date.

The KeyWall program allows the user to choose between different design methodologies and compare results. The designer should become comfortable with the differences in these design methodologies and be able to choose the appropriate design approach for any project with confidence. All pertinent data and design criteria is pre-programmed so the designer can focus on the wall geometry, loading conditions, and constructability. Actual test data is available to the designer for inter-unit shear capacity, soil reinforcement connection strength, and unit base shear resistance. This manual simplifies design by concentrating on the Keystone concrete wall units and specific geogrid products.



GEOTECHNICAL RESPONSIBILITY

Most civil projects designed by professional firms require a subsurface investigation by a qualified geotechnical engineer as part of the site engineering process. The purpose of the investigation is to provide design recommendations for structures that interact with the site soils and to comment on construction considerations for the soils in general.

It is important that the geotechnical investigation and analysis include an assessment of the soil and water conditions in the area of the proposed retaining structures. The appropriate design recommendations should address the following items as they pertain to the retaining structures:

- Bearing capacity of the foundation soils
- Strength properties of the in-situ and proposed fill soils
- Long-term global stability of the structure and adjacent slopes
- Settlement estimates
- Groundwater and subsurface drainage considerations

If this information is not included in the initial soils report, the geotechnical engineer should be contacted to provide the additional information required for the retaining wall design.

The design and successful performance of Keystone retaining wall structures is dependent upon the quality of information obtained by the site investigation. We recommend that all walls of significant size or walls with poor soils and/or steep slopes be evaluated by a geotechnical engineer. For many sites, the geotechnical engineer will be able to provide estimates of the basic design parameters and long-term stability considerations without extensive and expensive testing. For larger structures and more difficult soil conditions, the geotechnical engineer may have to obtain more information about the site soils with additional borings and/or lab tests.

The KeyWall computer-assisted design approach for retaining walls gives the user a sense of simplicity and security in the design of these structures. KeyWall simplifies design to the point that anyone, technically qualified or not, can easily perform an analysis. Although we encourage the responsible use of KeyWall for wall design, we strongly recommend that the final design and site conditions be reviewed by a qualified engineer.

Geotechnical engineering is both an art and a science that requires education and years of experience to properly characterize site conditions and soil properties. KeyWall will save many hours in design and engineering time, but a qualified engineers' design can save considerable expense in the construction stage and ensure that the proper assumptions were made during design for long-term performance.



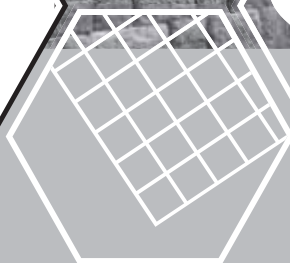
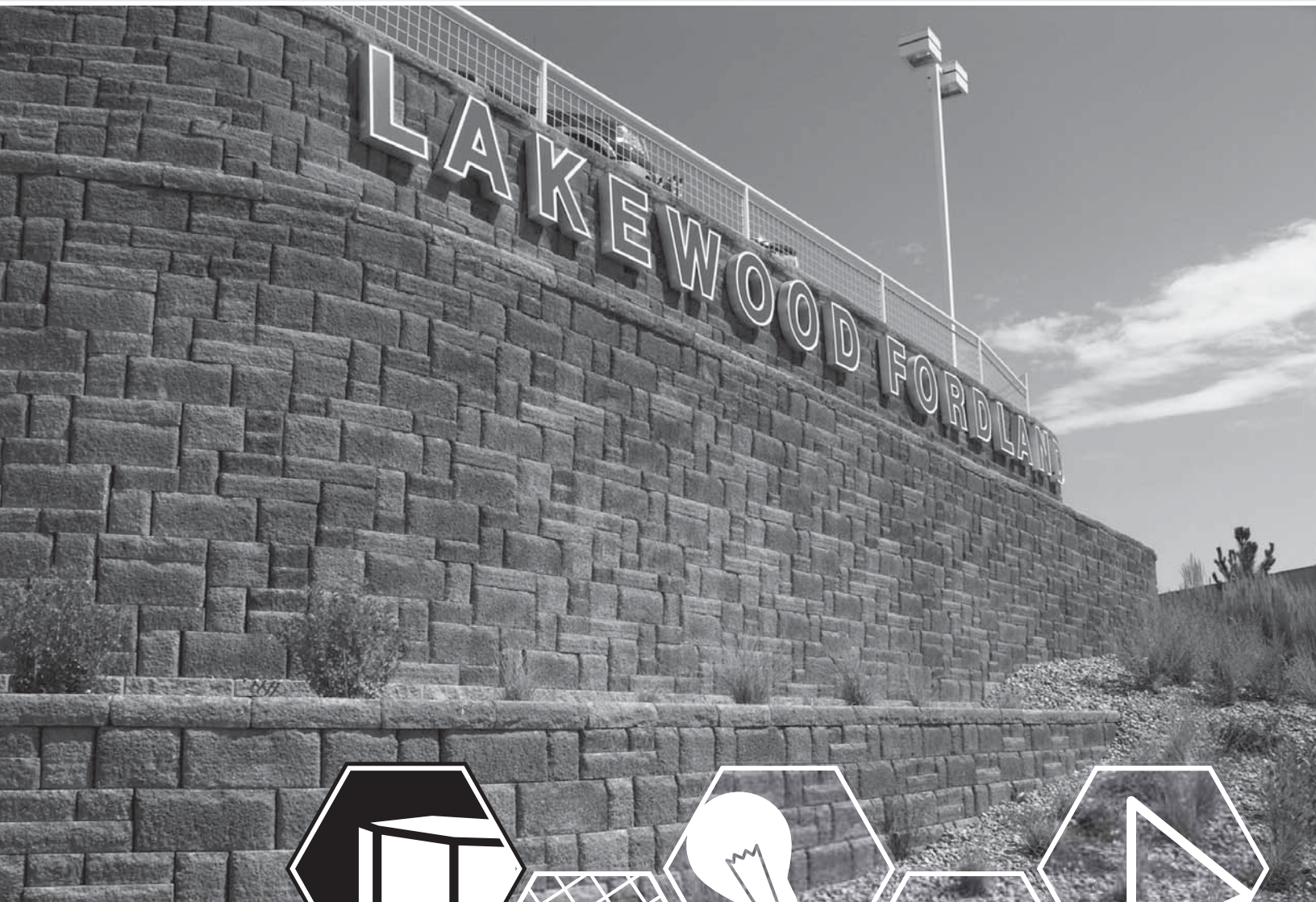
REFERENCES

- American Association of State Highway and Transportation Officials, *Standard Specifications for Highway Bridges*, Seventeenth Edition--2002, Washington, D.C.
- American Association of State Highway and Transportation Officials, *AASHTO LRFD Bridge Design Specifications*, 5th Edition, 2010, Washington, DC.
- American Society for Testing and Materials, Philadelphia, PA.
- Bowles, J.E., *Foundation Analysis and Design*, 5th ed., McGraw Hill Book Company, N.Y., 1996.
- Federal Highway Administration, *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines*, Report No. FHWA-NHI-00-043, National Highway Institute, Washington, DC, 2001.
- Federal Highway Administration, *Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes*, Report No. FHWA-NHI-10-024, National Highway Institute, Washington, DC, 2009.
- National Concrete Masonry Association, *Design Manual for Segmental Retaining Walls*, 2nd Edition, 1997.
- National Concrete Masonry Association, *Design Manual for Segmental Retaining Walls*, 3rd Edition, 2009.

PART ONE

KEYSTONE® RETAINING WALL UNITS

Fordland, Lakewood, Colorado; Keystone Century® & Half Century Wall®





KEYSTONE RETAINING WALL UNITS

Keystone retaining wall units are a zero-slump concrete masonry product developed specifically for use in earth retaining wall structures. Keystone has developed a wide variety of shapes and designs to accommodate most architectural and structural requirements. Local producers of the Keystone products have a variety of colors available, complementing most landscaping and structural retaining wall applications.

Keystone structural products currently available include:

- Standard I/Standard II, Figure 1:1
- Compac I/Compac II/Compac III, Figure 1:2
- Keystone Century Wall®, Figure 1:3
- 133Elite®, Figure 1:4

The Keystone units listed above are designed for use as structural retaining walls, i.e., those exceeding 6.5 feet (2m) in height and/or supporting structures or highway loading.

In addition to the above units, Keystone has a complete line of smaller landscape products that are marketed and sold through retail distribution and landscape supply outlets. These products are generally not considered for structural applications and are not discussed further in this manual.

Brentwood Gate, Burnaby, British Columbia; Keystone 133Elite®





PART ONE

Retaining Wall Units

Note:

Not all units types, face treatments and colors are available at all manufacturing locations. Please check with your local manufacturer or Keystone supplier for availability.

KEYSTONE MATERIALS

Keystone units are typically manufactured of concrete with a minimum compressive strength of 3000 psi (21MPa) at 28 days and a maximum absorption of 8%. All dimensions are plus or minus 1/8 inch (3mm) except for the unit depth, which varies due to the split rock finish. The manufacturing process is automated, so the mixing, compaction, and curing are performed under controlled conditions and provide consistent quality. The units have various face textures available, depending on your local manufacturer. Some of our most popular textures are molded or split-rock finish in various natural colors. Face shapes can be tri-plane, straight, Victorian, or Sculpterra™ molded face such as Hewnstone.

Standard, Compac, Keystone Century Wall & 133Elite are vertically interconnected using high-strength pultruded fiberglass pins. The Keystone units have cores that are filled with clean crushed stone to provide additional mechanical interlock and internal drainage. The pins assure a running bond configuration of the units and provide significant lateral connection strength between units. The pins improve the connection between the units and the structural soil reinforcement while assuring proper placement of the reinforcement materials.

The connection pins are available in straight and shouldered designs. Straight pins are 5 1/4 inches (133mm) long and 1/2 inch (12.7mm) in diameter. The Standard and Compac units use straight pins. Shouldered pins are 3 3/4 inches (95mm) long and 1/2 inch (12.7mm) in diameter. The shouldered length is 7/8 inch (22mm) and the shouldered diameter is 3/4 inch (20mm). The Century Wall and 133Elite units use shouldered pins. The minimum pin strength is 6,400 psi (44MPa) short beam shear strength and 110,000 psi (750MPa) tensile strength. The pins are manufactured of pultruded fiberglass and will not corrode or deteriorate. In addition, the fiberglass pin does not change properties (soften or become brittle) due to the temperature changes typical in retaining wall applications.

STANDARD UNIT

The Standard unit varies due to manufacturing considerations from 18 to 24 inches (457 to 600mm) in depth, with a typical face width of 18 inches (457mm) and height of 8 inches (203mm). The geometry yields exactly 1 square foot (0.09 m²) of face area per unit. Units weigh from 95 to 125 pounds (43 to 56kg) each, varying with local manufacturing and aggregates. The centroid of the unit is slightly forward of center toward the face, but for design purposes, it is taken at the center. For design purposes, the in-place density of the aggregate filled unit is 120 pcf (18.85 kN/m³).



Figure 1:1 Standard/Standard II Unit

Standard units are manufactured with a dual pin hole configuration. The front pin setting allows the units to be placed at a minimum setback of approximately 1/8-inch (3.2mm) per 8 inch (203mm) unit height (1° batter, for design purposes use 0°). The rear pin setting allows placement of the units at a minimum 1 1/4-inch (31.7mm) setback per 8 inch (203mm) unit height (8° batter). An alternate placement of front/back pin hole allows a setback of 5/8-inch (15.9mm) per 8 inch (203mm) unit height (4° batter).



COMPAC UNIT

The Keystone Compac unit is a 12 inch (305mm) deep unit with a typical face width of 18 inches (457mm) by 8 inches (203mm) high. This geometry yields exactly 1 square foot (0.09 m²) of face area per unit. Depth may vary from 11 to 12.5 inches (280 to 317mm) depending upon local manufacturing and splitting requirements. Units weigh from 70 to 95 pounds (32 to 43kg) each, varying with local manufacturing and aggregates. For design purposes, the in-place density of the aggregate filled unit is 120 pcf (18.85 kN/m³).



Figure 1:2 Compac / Compac II / Compac III Unit

The dual pin hole configuration allows the same 1° (0° for design purposes), 4°, and 8° setback as the Standard unit.

KEYSTONE CENTURY WALL® UNITS

Century Wall® is a three piece system that consists of a small, medium, and large unit. The width of the units is the varying dimension that dictates the size. The small unit is 7 inches (178mm) wide, the medium unit is 11 inches (279mm) wide, and the large unit is 18 inches (457mm) wide. The three Century Wall units are 12 inches (305mm) deep and 8 inches (203mm) high. The small unit weighs 45 pounds (20kg), the medium unit weighs 58 pounds (26kg), and the large unit weighs 90 pounds (41kg). Weights may vary with local manufacturing and aggregates.



Figure 1:3 Century Wall® Units

Similar to the Compac and Standard units, a dual pin hole configuration allows 1° (0° for design purposes), 4°, and 8° setback.



133ELITE® UNIT

133Elite® units are 8 inches (203mm) high and 24 inches (610mm) wide to create a face area of 1.33 square feet (0.124m²), hence the name 133Elite®. The depth of the unit is 11.5 inches (292mm). Depending on face treatment, the weight of the 133Elite® unit is approximately 100 pounds (45kg).



Figure 1:4 133Elite® Unit

133Elite units are manufactured with one pin position that creates a near vertical setback equal to $\frac{3}{8}$ inch (9.6mm) per 8 inches (203mm) of unit height (2.5° batter).

UNIT SHEAR RESISTANCE

There are two areas where the shear resistance is important:

- Leveling pad shear resistance
- Inter-unit shear resistance

Both are important to the wall's ability to resist lateral movements during construction and to hold the retained soil in place. The shear and moment capacity of the wall facing prevents bulging of the wall face.

BASE SHEAR RESISTANCE

A prepared leveling pad is required to provide a firm, level surface on which to place the base course units at the design elevations and provide localized bearing capacity for the units.

Leveling pads may be constructed of well-compacted gravel/crushed stone or unreinforced concrete. For most walls, the gravel/crushed stone leveling pad is adequate. For taller walls (over 15 feet or 5m), contractors have found that concrete can lead to faster wall installation and is easier to use on the larger projects. The concrete pad requires more care in placement and more expensive materials (concrete versus aggregate), but the speed of placing the first course generally offsets the extra cost of materials.

In Keystone walls with no earth reinforcement (gravity walls), the total resistance of the wall to lateral movement (sliding) is provided by the friction along the base of the units. In soil reinforced Keystone walls, unit base friction is a lesser component of the sliding calculation as the reinforced zone provides most of the resistance along the base. Since the leveling pad may be constructed of various materials, the frictional resistance varies with the roughness and shear strength of the materials.



INTER-UNIT SHEAR RESISTANCE

Laboratory testing has been performed to determine the inter-unit shear resistance of the various Keystone units. The inter-unit shear resistance is the internal shear capacity of the wall facing. Without adequate shear resistance between units, a wall could bulge between layers of reinforcing, shear during construction, or in the case of a gravity wall, shear between any unit above the base.

For gravity walls, the inter-unit shear capacity is obtained based on the calculated normal force. When a layer of geogrid reinforcing is included in the wall system, the shearing resistance between units may be reduced because the reinforcing can reduce friction between units. The granular interlock is decreased and the unit-to-unit friction may be reduced. For many systems, reinforcing may actually decrease the stability of the face while providing stability to the overall earth mass.

SHEAR DATA AND ANALYSIS

Inter-unit shear testing has been performed on all Keystone structural units. Testing was initially done at Utah State University on the Compac and Standard units. The inter-unit shear testing on the remainder of the units has been completed by Bathurst, Clarabut Geotechnical Testing Inc. The results of the test for the Compac and Standard units are graphically depicted in Figures 1:5 and 1:6. Laboratory testing provides the following derived equations for shear resistance based on a total calculated normal force, N, in lbs/lf.

INTER-UNIT SHEAR TABLE

$$N = h W_u \gamma_{\text{unit}}$$

where:

$$h = \text{Depth to Interface}$$

$$W_u = \text{Width of unit face}$$

$$\gamma_{\text{unit}} = \text{Unit weight of unit face}$$

| | Unit to Unit | Unit to Unit w/geogrid |
|--------------|--------------------------|--------------------------|
| Standard | $F = 2427 + N \tan 17.4$ | $F = 1550 + N \tan 17.4$ |
| Standard II | $F = 1375 + N \tan 35$ | $F = 1556 + N \tan 19$ |
| Compac | $F = 769 + N \tan 26.9$ | $F = 769 + N \tan 26.9$ |
| Compac II | $F = 1475 + N \tan 29$ | $F = 1250 + N \tan 29$ |
| Compac III | $F = 1393 + N \tan 34$ | $F = 900 + N \tan 34$ |
| Century Wall | $F = 900 + N \tan 30$ | $F = 900 + N \tan 30$ |
| 133Elite | $F = 1100 + N \tan 29$ | $F = 1100 + N \tan 29$ |

Additional direct shear tests were completed at Utah State University to evaluate base shear using three types of leveling pad materials. The results of that testing is listed below:

BASE SHEAR TABLE

| | Standard | Compac |
|-------------------|-------------------|-------------|
| Crushed Stone Pad | $F = 995 + 0.31N$ | $F = 0.92N$ |
| Concrete Pad | $F = 205 + 0.30N$ | $F = 0.90N$ |
| Sand Pad | $F = 290 + 0.29N$ | $F = 0.49N$ |

To determine metric equivalents in kN/m, divide the “y-intercept” by 68.5. For example, the Standard unit to unit shear equation would be $F = 35.43 + N \tan 17.4$. Shear test reports are available from Keystone.



TYPICAL SHEAR RESISTANCE BETWEEN UNITS

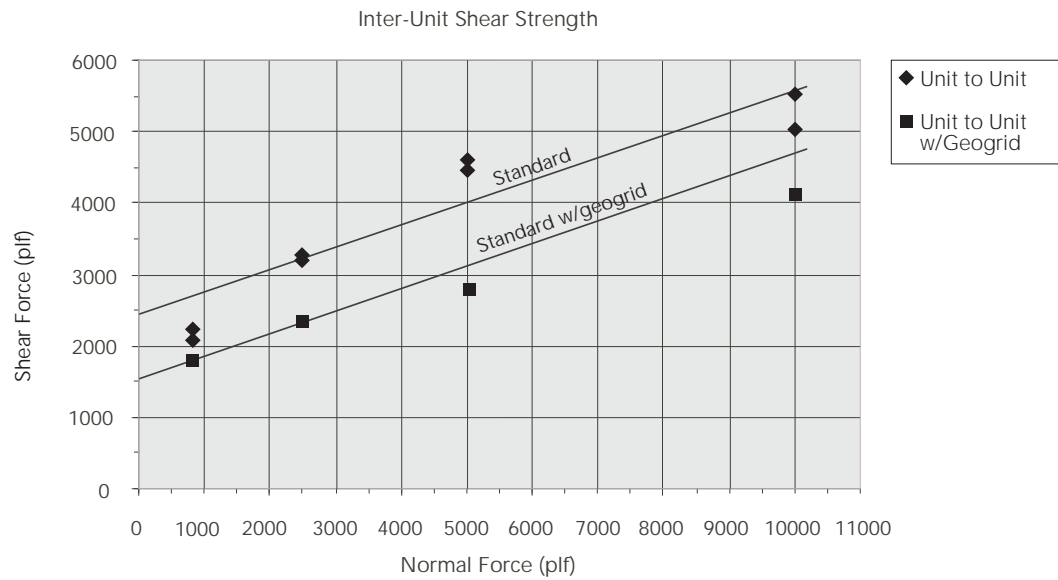


Figure 1:5 Standard Unit Inter-unit Shear Strength

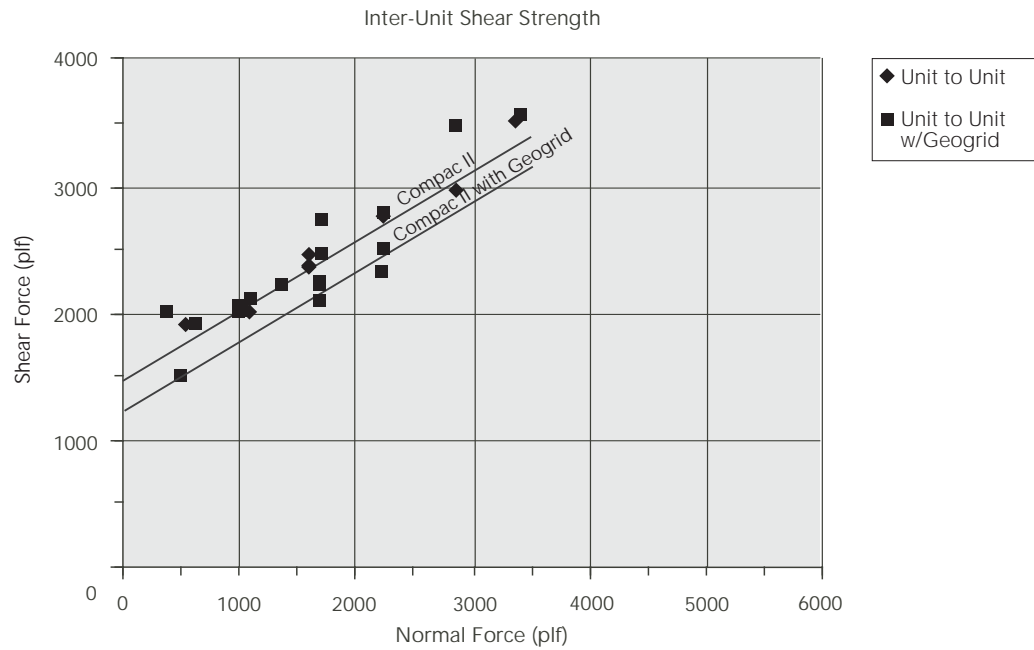


Figure 1:6 Compac II Unit Inter-unit Shear Strength

PART TWO

GEOSYNTHETIC SOIL REINFORCEMENT

Martin Street Improvements, Fredonia, Wisconsin; Keystone Compac Hewnstone





GEOSYNTHETIC SOIL REINFORCEMENT

Keystone retaining walls may perform as gravity retaining walls for heights up to 6 ft (1.8m) for Standard units and 3.3ft (1m) for Compac units, depending on geometry, soil type, and specific loading. When a wall exceeds safe gravity heights, soil reinforcement is required to provide stability against overturning and sliding. The majority of Keystone retaining walls are constructed using geosynthetic reinforcement, which is the focus of this manual and the KeyWall program. Design for inextensible steel reinforcement is discussed in the Keystone KeySystem design manual and HITEC Evaluation, August 2000.

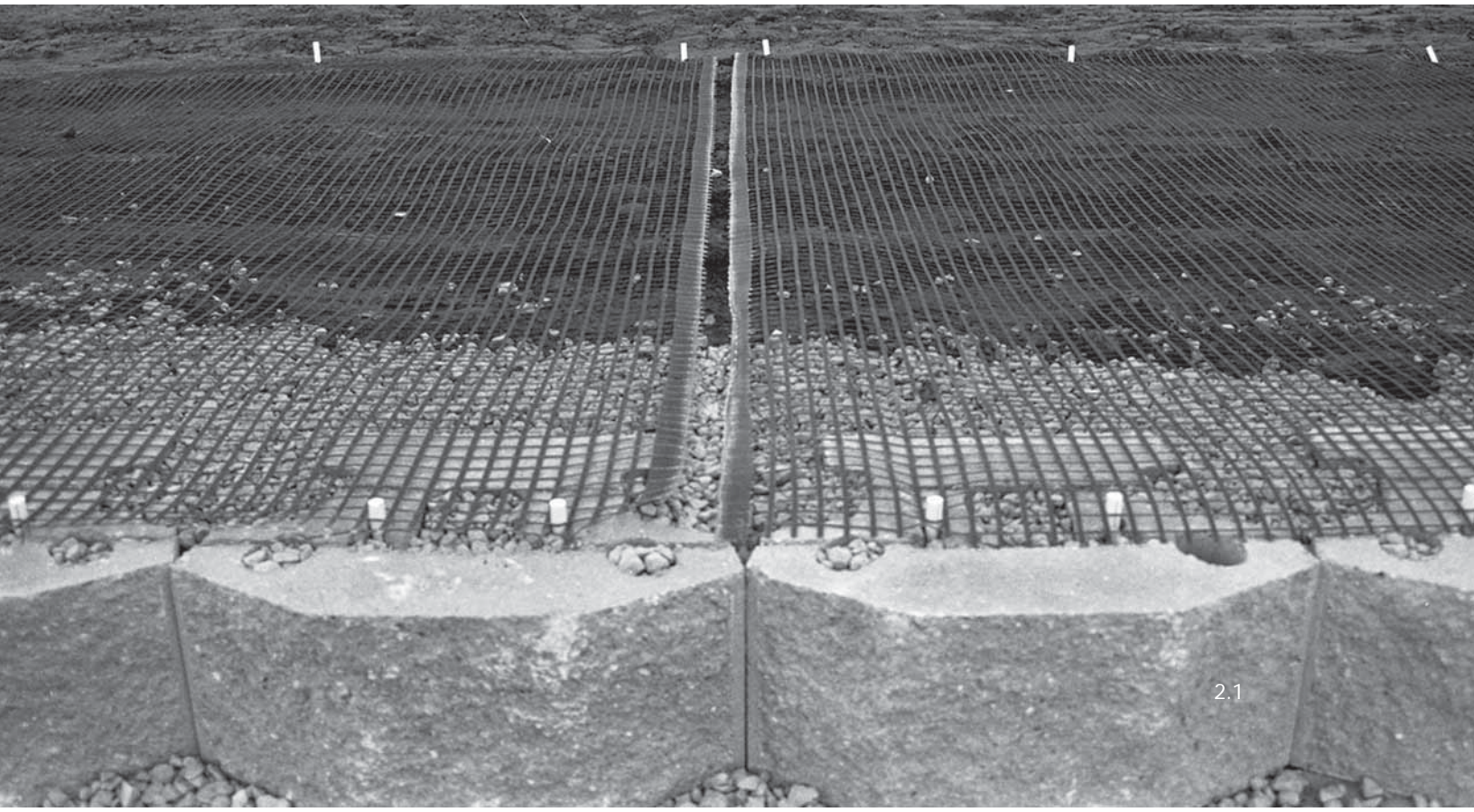
Soil-reinforced walls typically consist of geosynthetic materials, primarily geogrids, which are connected to the Keystone units and placed in horizontal layers in the compacted backfill. A limit equilibrium design procedure is used to determine the number, strength, length, and distribution of geosynthetic reinforcement layers required to form a stable soil-reinforced mass.

Geosynthetic material design parameters in the limit equilibrium analysis are:

- Long-Term Design Strength (LTDS) and allowable strength, T_{al}
- Geosynthetic-Keystone unit connection strength, T_{cl} , T_{sc}
- Geosynthetic-soil pullout interaction coefficient, C_i
- Geosynthetic-soil direct shear coefficient, C_{ds}

Terminology used to define geosynthetic soil reinforcement tensile strength varies somewhat between authors, specifiers, and suppliers. The terminology used within this section is consistent with that of NCMA and AASHTO/FHWA, unless otherwise noted.

Wall construction with Geogrid Reinforcement.





DESIGN STRENGTH

The practice for determining the allowable tensile strength of geosynthetic reinforcement, (T_{al}), is based upon the U.S. Federal Highway Administration guidelines. This method establishes the Long Term Design Strength (LTDS), be based on sustained load testing, extrapolated to a design life for the structure.

The Long-Term Design Strength (LTDS), for geogrid reinforcement utilized in the Keystone Retaining Wall design is:

$$\text{Equation (2a)} \quad LTDS = \frac{T_{ult}}{RF_{cr} \times RF_{ld} \times RF_d}$$

There are two design philosophies currently employed in retaining wall design and analysis. Allowable Stress Design (ASD) is the conventional working stress and factor of safety method of analysis that has been used for years. Limit State Design (LSD) or Load and Resistance Factor Design (LRFD) is the newer method that compares factored loads to factored resistances.

Allowable Stress Design

The long-term design strength (LTDS), is reduced by an overall safety factor (FS), in Allowable Stress design to account for all factors on loads, uncertainties, etc.

$$\text{Equation (2b)} \quad T_{al} = \frac{LTDS}{FS}$$

Limit State Design (LRFD)

The Long Term Design Strength (LTDS) is reduced by a resistance factor (ϕ) in Limit State Design/ (LRFD) to account for material uncertainty.

$$\text{Equation (2c)} \quad T_{al} = \phi_{GEO} LTDS$$

The NCMA, Rankine, AASHTO 96, and AASHTO Simplified design methods in KeyWall employ Allowable Stress design procedures. The AASHTO LRFD, Canadian LRFD and Australian design methods in KeyWall employ Limit State design procedures.

Tensile Strength (T_{ult})

T_{ult} is the ultimate strength of geosynthetic reinforcing when tested in a wide width test per ASTM D4595 (geotextile) or D6637 (geogrid). This value is reported as the Mean Average Roll Value (MARV), as determined by the manufacturer's quality control process and accounting for statistical variation.



REDUCTION FACTORS

RF_{cr}

RF_{cr} is the reduction factor to account for the long-term creep characteristic of polymeric materials. The long-term tension-strain-time behavior of polymeric reinforcement is determined from results of controlled laboratory creep tests conducted on finished-product specimens for periods up to 10,000 hours per ASTM D5262 and D6992. The data is then extrapolated to the project design life of the structure, 75 years or 100 years. Creep rupture testing is similar to the procedure described above, however, the load at which rupture may occur at the end of the design life is predicted. A combination of 10,000 hour testing and creep rupture testing appears to be the current standard for evaluating geosynthetic material creep. Typical range of RF_{cr} is 1.4 to 5.0.

RF_{id}

RF_{id} is the reduction factor for installation damage (i.e., cuts, nicks, tears, etc.) created by fill placement and construction equipment operations with various backfill material that can potentially reduce reinforcing strength and performance. The recommended reduction factor for reinforcement installation damage is based on results of full-scale construction damage tests. Site specific values may be determined by performing construction damage tests for the selected geosynthetic material with project specific backfill and equipment. Typical range of RF_{id} is 1.05 to 2.00.

RF_d

RF_d is the reduction factor to account for the effects of chemical and biological exposure to the reinforcement that are dependent on material composition, including resin type, resin grade, additives, manufacturing process, and final product physical structure. For most soils used with the Keystone System, the manufacturers have included recommended factors to account for possible chemical and biological degradation. In soils where high alkalinity or other aggressive factors (ph < 3 or > 9) may be present, the manufacturer should be contacted for specific recommendations. Typical range of RF_d is 1.0 to 2.0.

For further information on the chemical and biological durability of a reinforcement, a review of durability is presented in FHWA-NHI-09-087 "Corrosion/Degradation of Soil Reinforcement for MSE Walls and Reinforced Soil Slopes."

FS

FS is the overall tension safety factor for material, geometric, and loading uncertainties that cannot be specifically accounted for. FS is similar to other overall safety factors in Allowable Stress design. A minimum factor of safety of 1.5 is required for most permanent applications. For unusual loading conditions, variable or poorly defined soil conditions, this factor may be increased at the discretion of the designer.

ϕ_{GEO}

ϕ_{GEO} is the geosynthetic resistance factor for material uncertainty used in Limit State Design. The US AASHTO LRFD Code uses 0.90 for the tensile resistance factor. Resistance factors will vary in Limit State Design based on the load/resistance factor system adopted by specific design codes.

Note:

KeyWall permits the choice of three different backfill materials (fine-grained soils, 3/4- inch (20 mm) minus, and 2 inch (50 mm) minus), which establish a default value for the installation damage factors used in the LTDS determination, based on the manufacturer's recommendations and testing.



Note:

Reinforced soil wall designs are unique to the specific Keystone units and geosynthetic reinforcement used. Connection data is specific to each combination and reinforcement level. Substitution of any materials invalidates a given wall design.

CONNECTION STRENGTH

The connection strength is the strength state reinforcement-facing connection strength. The capacity is dependent upon the vertical depth to the reinforcement, wall geometry, type of Keystone unit utilized, and the specific geosynthetic utilized.

Laboratory testing is required to define the connection strength for specific units and geosynthetic materials at varying normal pressures. Typical graphs for an individual stress-strain test and complete series plot is shown in Figure 2:1 and Figure 2:2 per NCMA Test Method SRWU-1/ASTM D6638.

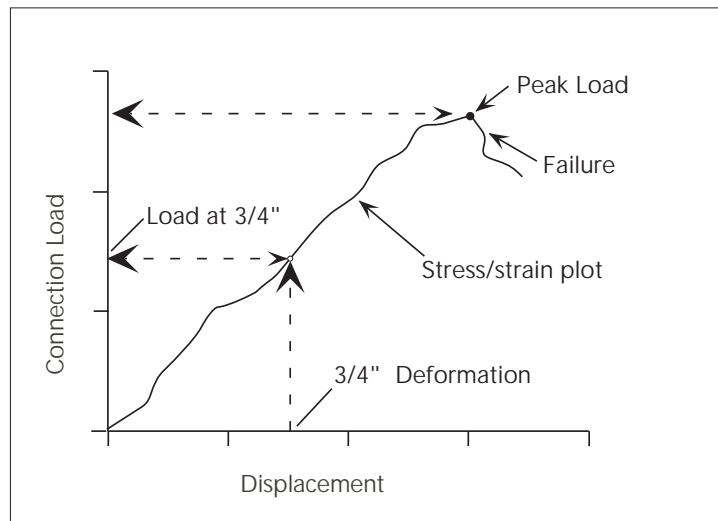


Figure 2:1 Load Test at one Normal Force

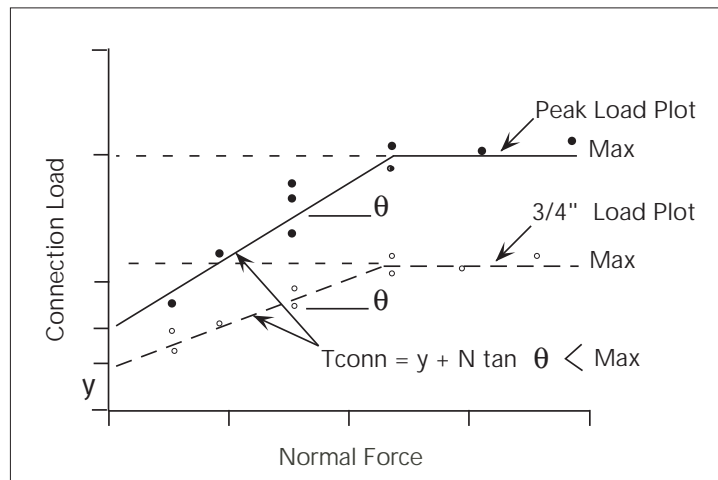


Figure 2:2 Connection Load Plots at Different Normal Forces



CONNECTION STRENGTH

In Allowable Stress design, the calculated tensile load at each reinforcement level in the geosynthetic must be less than 1) the allowable geosynthetic design strength, T_{al} , and 2) an ultimate connection strength limit, T_{cl} , divided by a safety factor (T_{cl} / FS). The recommended minimum factor of safety on the ultimate connection strength is 1.5.

In Limit State/LRFD design, the factored tensile load at each reinforcement level in the geosynthetic must be less than 1) the allowable geosynthetic design strength, T_{al} , and 2) the ultimate connection strength limit, T_{cl} , times the appropriate resistance factor (ϕ).

Serviceability strength, T_{sc} , is defined as the connection strength at a maximum 0.75-inch (20mm) movement, as determined with the NCMA Test Method SRWU-1/ASTM D6638. Serviceability criteria is only considered when a service state analysis is being performed. This is rarely considered in current MSE wall design as service state deformation is not well understood. NCMA and AASHTO design codes generally ignore the service state condition and require a strength analysis only.

GEOSYNTHETIC-SOIL INTERACTION COEFFICIENT

Two types of soil-reinforcement interaction coefficients or interface shear strength parameters are used for design of soil reinforced structures: pullout interaction coefficient, C_i , and direct shear coefficient, C_{ds} .

The pullout interaction coefficient is used in stability analysis to compute the frictional resistance along the reinforcement/soil interface in the zone beyond a defined plane of failure. The calculation yields the capacity to resist pullout of the reinforcement from the soil.

The direct shear coefficient is used to determine the factor of safety against outward sliding of the wall mass along the layers of reinforcement. The coefficients are determined in the laboratory and are a function of soil and geosynthetic material types.

Design pullout resistance of the geosynthetic reinforcement is defined as the ultimate tensile load required to generate movement of the reinforcement through the soil mass measured at a maximum $\frac{3}{4}$ inch (19 mm) displacement. The recommended minimum factor of safety against geosynthetic pullout is 1.5. Equivalent resistance factors are used in limit state design. ASTM D6706 may be used to determine pullout coefficients for geogrids.

Note: AASHTO requires that 1000 hour sustained load testing be performed on all geosynthetic connection schemes for MSE walls per FHWA guidelines. This testing can result in an additional reduction factor that reduces connection capacity over the life of the structure. There is no evidence that connection creep is a long term performance consideration based on the thousands of walls constructed since the 1980's. NCMA does not recognize the concept in their "Design Manual for Segmental Retaining Walls" and Keystone has not observed this in practice. The most probable explanation is that the "Design" loads never materialize at the connection as extensible reinforcement can "yield" to release any stress build up while the surrounding soil and reinforcement picks up the load (ie, arching). Current US practice is to analyze the connection at 100% of the theoretical design load in the reinforcement which overstates the load and probably explains the lack of creep related connection issues.



Note:

These coefficients do not apply to Geotextiles or “fat” soils.

GEOSYNTHETIC-SOIL INTERACTION COEFFICIENT

KeyWall 2010 uses the following default values for C_i and C_{ds} based upon the ϕ angle inputted for the reinforced fill material.

| Soil Type (USC*) | ϕ ANGLE | C_i | C_d |
|--|----------------------|-------|-------|
| Crushed Stone, Gravel (GW, GM) | $\phi \geq 32^\circ$ | 0.9 | 0.9 |
| Sand, Gravel, Silty Sands (SW, SM, SP) | $\phi \geq 28^\circ$ | 0.8 | 0.8 |
| Sandy Silt, Lean Clay (SC, ML, CL) | $\phi \geq 25^\circ$ | 0.7 | 0.7 |
| Other clay (CL/CH) | $\phi < 25^\circ$ | 0.6** | 0.6** |

* (USC is the Unified Soil Classification System per ASTM D-2487)

** (Consult geogrid manufacturer)

Note:

As new materials are developed or new data is provided, KeyWall's data files will be updated. Keystone provides the data as a service to its customers and to ensure data uniformity for Keystone design. This information is provided with no written or implied warranty as to the accuracy of the data supplied. Data values should be verified with the manufacturers.

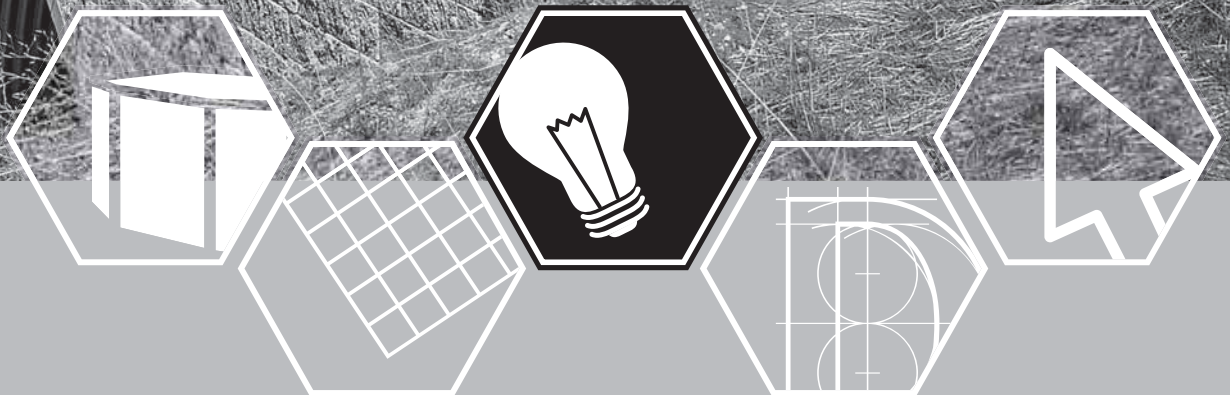
GEOGRID MANUFACTURERS' DATA

Geogrid manufacturers were contacted during the development of this Keystone Design Manual and asked to provide the required information to evaluate their design parameters. The material provided and testing performed by Keystone is the basis for the values programmed into KeyWall. Information provided to Keystone by the geogrid manufacturers/suppliers is available directly from the geogrid manufacturers.

PART THREE

RETAINING WALL DESIGN THEORY

Arbor Lakes, Beaverton, Oregon; Keystone Century Wall®





RETAINING WALL DESIGN THEORY

Earth retaining wall structures require three primary areas of design analysis: 1) lateral earth pressures 2) foundation bearing capacity, and 3) global or overall stability. The analysis of each is based on the following engineering properties of the soil(s): angle of internal friction (ϕ), soil cohesion (c), and the density (γ) of the soils.

In this chapter, the basic mechanisms of lateral earth pressures and stability of foundations are presented. Global stability and seismic analysis are beyond the scope of this design manual but a brief description is provided. Once the basic concepts and mechanisms of earth pressures are understood, simplification of the calculations to develop the Coulomb and Rankine earth pressure theories can be examined. There are further simplifications made to the theories when adapted for design of mechanically stabilized earth (MSE) structures. By starting with the basic theory, it is easier to understand the mechanisms of performance and failure and adapt the design to special conditions not directly addressed by the simplified methods.

The user should refer to recent geotechnical textbooks, the NCMA *Design Manual for Segmental Retaining Walls*, FHWA *Design and Construction of Mechanically Stabilized Earth Walls and Slopes*, and AASHTO *Standard Specifications for Highway Bridges*, for additional material and information on soils and MSE structures. This manual is solely intended to provide insight into the KeyWall design software and the general principles of modular wall design without being an exhaustive summary of soil mechanics.

Thurgood Marshall Middle School, San Diego, California; Keystone Standard & Compac





IMPORTANT TECHNICAL DEFINITIONS

Effective Stress Design

The soil strength parameters are based on drained conditions that are applicable to granular soils and fine grained soils for long term, drained conditions. (Note: these properties are referred to as ϕ' and c' in most textbooks. In this manual, ϕ and c will be used for simplicity and represents effective stress analysis.)

Angle of Internal Friction (ϕ)

This value represents the frictional shear strength of the soil when tested under compacted and confined conditions. This value should not be confused with a soil “angle of repose,” which reflects the angle that a pile of loose soil will naturally stand.

Peak Strength

The peak shear strength of a soil is the maximum load measured during a test at a nominal displacement. This manual will utilize peak shear strength values in effective stress analysis unless otherwise noted. Residual strength values require greater movement of the soil than is intended by the design of reinforced soil structures but may be appropriate in some cases with cohesive soils.

Global Stability

Conventional retaining wall design only looks at simple sliding, overturning, and bearing as failure modes. This manual refers to global stability as all other combinations of internal and external stability, slope stability, and compound failure planes that may compromise the wall structure.

Failure Plane

Soil failure planes are typically non-linear and are often represented by a log-spiral curve. Internally, the failure plane (locus of maximum stress points) is modeled as a straight line following the appropriate Rankine or Coulomb definition of the slope angle for simplification.

Bearing Capacity Factors

The general bearing capacity formula as proposed by Terzaghi is used. However, different bearing capacity factors have been published by Meyerhof, Hansen, and Vesic over the years. This manual uses the factors proposed by Vesic (1975), which is consistent with the other documents discussing MSE walls. (Note: All factors assume level ground and must be adjusted for sloping ground conditions.)

Soil mechanics text books include sections on passive and active earth pressures. They describe the theories of Coulomb and Rankine and methods of solution via formulas, graphical methods, and computer analysis. This manual will briefly discuss the methods of active earth pressure calculation as it relates to reinforced soil structures and accepted design principals. Passive pressures are typically neglected and not covered in this manual.



LATERAL EARTH PRESSURE THEORIES

The NCMA *Design Manual for Segmental Retaining Walls, Second and Third editions*, is based on Coulomb earth pressure theory. The basic assumptions for this active wedge theory were developed by Coulomb (1776). The other major methodology is Rankine earth pressure theory (1857), which is based on the state of stress that exists in the retained soil mass. Both theories essentially model the weight of the soil mass sliding along a theoretical plane of failure (Figure 3.2 and 3.3). The lateral earth pressure, P_a , is the net force required to hold the wedge of soil in place and satisfy equilibrium.

The major difference between the two theories is that the Coulomb model and equations account for friction between the back of the wall and the soil mass as well as wall batter. Rankine equations more conservatively assume no wall friction at the soil-wall interface and a vertical wall structure which greatly simplifies the mathematics of the problem. The friction at the back of the wall face and at the back of the reinforced zone for external stability computations, provides an additional force component that helps support the unstable wedge of soil. Because of these additional resisting forces, the lateral earth pressure calculated by Coulomb is generally less than the earth pressure that would be predicted by the Rankine equations.

AASHTO design methodologies generally applies Rankine earth pressure theory for earth reinforced structures. AASHTO design methodology is required on most transportation related projects operating under this more conservative design criteria.

Note:

When the backslope is equal to the assumed friction at the back of the wall ($\beta=\delta$), Coulomb and Rankine formulas provide identical earth pressure coefficients and resultant forces for vertical walls.

Note:

For those interested in comparing Coulomb versus Rankine versus AASHTO, KeyWall allows the user to select each design methodology. NCMA is the Coulomb analysis per the NCMA design manual; Rankine and AASHTO use a Rankine approach, but will account for wall batter if entered in the KeyWall "Geometry" selection.



COULOMB EARTH PRESSURE THEORY

The reader should note that for horizontal surfaces (level surcharge) or infinite sloping surfaces (extending beyond the theoretical Coulomb failure plane), a closed-form equation solution is applicable and easily derived. For geometries where the slope changes within the zone of failure (broken back slope), the simple equations are no longer applicable and may be unnecessarily conservative. For example, if a short broken back slope is modeled as an infinite slope, the design may require significantly more reinforcement and excavation than if modeled correctly. For these conditions, the trial wedge method is used in the analysis. This is an iterative trial wedge process where successive failure surfaces are modeled until a maximum earth pressure force is calculated for the geometry and loading given (See Figure 3:4).

The earth pressure behind the wall face or at the back of the reinforced zone is represented by a triangular pressure distribution for active soil pressure and a rectangular distribution for uniform surcharge pressure as is shown in Figure 3:1.

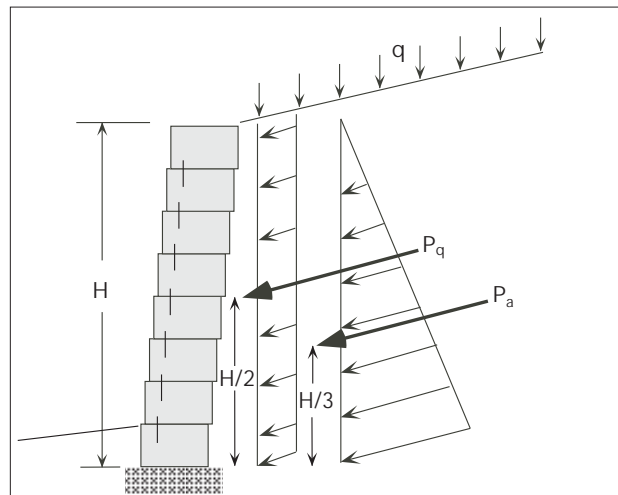


Figure 3:1 Earth Pressure Diagram

COULOMB EARTH PRESSURE EQUATION

The appropriate Coulomb earth pressure equations for earth and surcharge pressure are as follows:

Equation (3a) $P_a = \frac{1}{2} \gamma H^2 k_a$

Equation (3b) $P_q = qHk_a$

where:

- k_a = coefficient of active earth pressure
- γ = moist unit weight of the soil
- H = total design height of the wall
- q = uniform surcharge



COULOMB EARTH PRESSURE EQUATION

The active earth pressure coefficient, k_a , is determined from an evaluation of the Coulomb wedge geometry shown in Figure 3:2 and results in the following k_a coefficient:

$$\text{Equation (3c)} \quad k_a = \frac{\sin^2 (\alpha + \phi)}{\sin^2 \alpha \sin (\alpha - \delta) \left[1 + \sqrt{\frac{\sin(\phi + \delta) \sin(\phi - \beta)}{\sin(\alpha - \delta) \sin(\alpha + \beta)}} \right]^2}$$

where:

- α = angle of batter from horizontal
- ϕ = angle of internal friction of soil
- β = slope angle above wall
- δ = angle of friction at back of wall

This equation is found in differing forms in other texts due to the trigonometric assumptions made in the formula derivation. The derivation of this Coulomb formula can be found in geotechnical textbooks such as *Foundation Analysis and Design* by Bowles (1996).

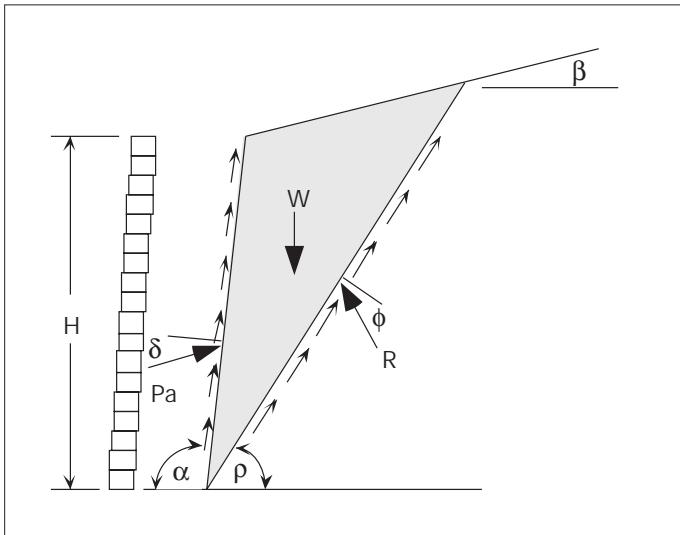


Figure 3:2 Coulomb Wedge Diagram



COULOMB FAILURE PLANE LOCATION

The Coulomb failure plane varies as a function of the wall geometry and friction angles for both the soils and the soil/wall interface. For level surcharge and infinite slope conditions, the relationship for ρ is:

$$\text{Equation (3d)} \quad \tan(\rho - \phi) = \frac{-\tan(\phi - \beta) + \sqrt{\tan(\phi - \beta)[\tan(\phi - \beta) + \cot(\phi + \iota)][1 + \tan(\delta - \iota)\cot(\phi + \iota)]}}{1 + \tan(\delta - \iota)[\tan(\phi - \beta) + \cot(\phi + \iota)]}$$

where:

- ϕ = angle of internal friction
- ι = batter of wall measured from vertical ($\alpha - 90^\circ$)
- β = slope angle above the wall
- δ = angle of friction at back of wall
(or reinforced mass)

Tables are available in the NCMA Design Manual and elsewhere that tabulate these values and assist in determining the appropriate Coulomb earth pressure coefficients and failure plane orientation based upon the wall geometry and soil parameters. The KeyWall program calculates these values for each geometry. For broken back conditions, a trial wedge calculation is used instead of the formulas.

RANKINE EARTH PRESSURE THEORY

Rankine earth pressure is a state of stress evaluation of the soil behind a retaining structure that traditionally assumes a vertical wall and no friction between the soil/wall interface. The orientation of the resultant earth pressure is parallel to the backslope surface.

RANKINE EARTH PRESSURE EQUATIONS

The earth pressure behind the wall face or at the back of the reinforced zone is represented by a triangular pressure distribution similar to that shown in Figure 3:1. The earth pressure equations are the same as Coulomb:

$$\text{Equation (3e)} \quad P_a = \frac{1}{2} \gamma H^2 k_a$$

$$\text{Equation (3f)} \quad P_q = qHk_a$$

where:

- k_a = coefficient of active earth pressure
- γ = moist unit weight of the soil
- h = total design height of the wall
- q = uniform live load surcharge



TRIAL WEDGE ANALYSIS

The limitation of closed form solutions, such as the Coulomb and Rankine equations, is that only simple level and infinite sloping surcharges with uniform loadings can be analyzed. It is necessary to look at a “trial wedge” method or “approximation” method when attempting to analyze broken back slopes or other slope/load combinations.

AASHTO and NCMA suggest an approximation method for broken-back slope conditions that defines equivalent design slopes for the external analysis. However, the internal analysis is not well defined for unusual slopes and loading conditions and the designer is expected to use engineering judgement with the simplified methods.

The KeyWall program uses a “trial wedge” analysis for determining the internal and external forces in order to provide the “correct” results for more complicated design geometries. The “trial wedge” calculation is an iterative process that determines the loading at successive failure plane orientations until a maximum loading is determined for the geometry and surcharge loading (See Figure 3:4).

Note:

The AASHTO Simplified, AASHTO LRFD, and CAN LRFD methods use the AASHTO “Simplified” method for calculating internal pressures and the trial wedge for calculating external loading conditions.

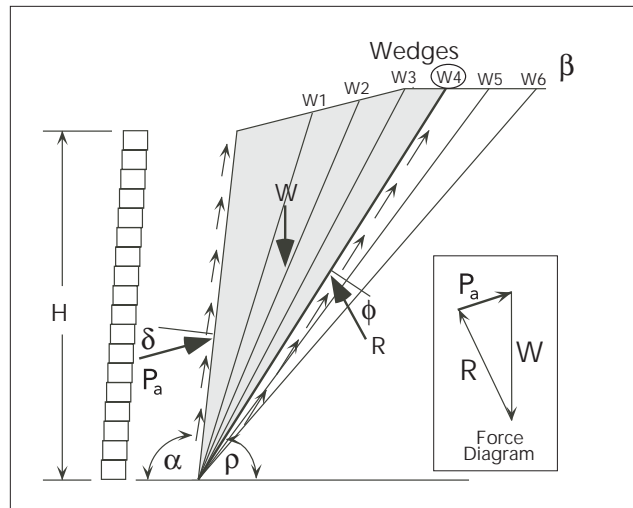


Figure 3:4 Trial Wedge Diagram

The KeyWall “trial wedge” analysis used is consistent with the fundamental assumptions of the applicable Coulomb and Rankine theories by setting $\delta = \beta$. “Trial wedge” results match the equation solutions for the level and infinite slope conditions, but will determine the “correct” internal and external values for broken back slope conditions and offset live and dead loads. This method of analysis permits the designer to properly model many typical design conditions and not overly simplify the analysis due to limitations of equation solutions and other design software.



BEARING CAPACITY

Bearing capacity is the ability of the foundation soil to support additional loading imposed on the surface from the completed wall system. Bearing capacity is analyzed considering two criteria:

- Shear capacity of the soil
- Total and differential settlement

Shear capacity of the soil is a function of the foundation soil strength, the soil mass equivalent footing size, the depth of embedment, and any groundwater conditions as determined by the geotechnical investigation.

APPLIED BEARING PRESSURE

Figure 3:5 shows the Meyerhof distribution of applied bearing pressure for flexible foundation systems that is typically utilized with earth reinforcement structures.

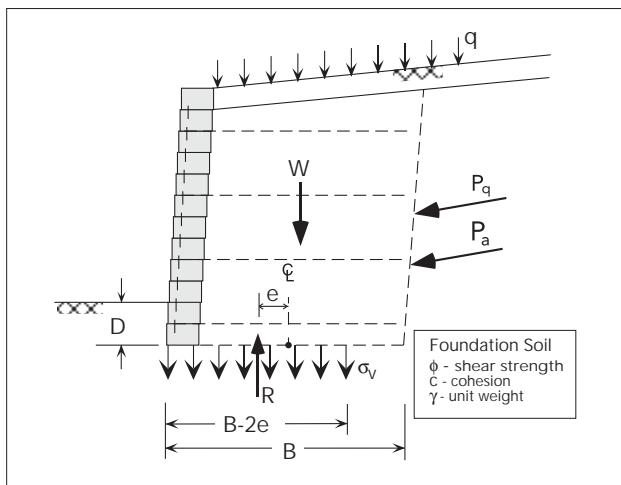


Figure 3:5 Applied Bearing Pressure Diagram

The equivalent footing width and applied bearing pressure are calculated as follows:

Equation (3i) $e = B/2 - (M_r - M_o)/R_v$

Equation (3j) $\sigma_v = R_v/(B - 2e)$

where:

- e = eccentricity of reaction
- B = total length of base
- M_r = sum of resisting moments
- M_o = sum of overturning moments
- R_v = sum of vertical reactions



CALCULATED BEARING CAPACITY

Q_{ult} is the ultimate bearing capacity of the foundation soils based on the soil and geometry parameters. The ultimate foundation bearing capacity can be calculated from the following Meyerhof equation:

$$Q_{ult} = cN_c s_c d_c + \gamma D N_q s_q d_q + 0.5 \gamma B N_\gamma s_\gamma d_\gamma$$

For which infinitely long strip footing with shape and depth factors = 1.0, and effective base width of $B - 2e$, simplifies to:

$$\text{Equation (3k)} \quad Q_{ult} = cN_c + \gamma D N_q + 0.5 \gamma (B - 2e) N_\gamma$$

where:

- c = cohesion of foundation soil
- γ = unit weight of foundation soil
- D = depth of embedment below grade
- $B - 2e$ = effective footing width
- N_c = bearing capacity factor for cohesion
- N_q = bearing capacity factor for embedment
- N_γ = bearing capacity for footing width

BEARING CAPACITY FACTORS

Bearing capacity factors for the bearing capacity equation (through Vesic 1975) are as follows:

$$\begin{aligned} N_c &= (N_q - 1) \cot \phi \\ N_q &= e^{\pi \tan \phi} \tan^2(45 + \phi/2) \\ N_\gamma &= 2(N_q + 1) \tan \phi \end{aligned}$$

The factor of safety for bearing capacity is the ratio of ultimate bearing capacity to the calculated applied bearing pressure.

$$\text{Equation (3l)} \quad FS_{\text{bearing}} = Q_{ult} / \sigma_v$$

Note:

In some cases, bearing capacity is determined without considering the wall embedment portion of the equation, $\gamma D N_q$. See KeyWall Design Preferences for this option.

A minimum safety factor of 2.0 (NCMA) and 2.5 (AASHTO ASD) against bearing capacity failure is considered acceptable for flexible earth reinforced structures.

AASHTO LRFD computes a capacity demand ratio (CDR) for bearing capacity using the equation below:

$$\text{Equation (3m)} \quad CDR_{\text{bearing}} = Q_{ult} RF_b / \sigma_v \text{ (factored loads)}$$

The MSE wall bearing resistance factor is $RF_b = 0.65$. As is always the case, the bearing capacity demand ratio is > 1.0 .



BEARING CAPACITY FACTORS

Bearing capacity in KeyWall is based solely on the soil parameters input for the foundation soil and the embedment depth assuming that the ground is level in front of the wall. The designer may check for a total stress condition by inserting total stress parameters for the foundation soil.

| ϕ | N_c | N_q | N_γ |
|--------|-------|-------|------------|
| 0 | 5.14 | 1.00 | 0.00 |
| 5 | 6.49 | 1.57 | 0.45 |
| 10 | 8.34 | 2.47 | 1.22 |
| 15 | 10.98 | 3.94 | 2.65 |
| 20 | 14.83 | 6.40 | 5.39 |
| 25 | 20.72 | 10.66 | 10.88 |
| 30 | 30.14 | 18.40 | 22.40 |
| 35 | 46.12 | 33.30 | 48.03 |
| 40 | 75.31 | 64.20 | 109.41 |

Bearing Capacity Factors (Vesic 1975)

Note:

Limit State or LRFD analysis requires a service state analysis in addition to the strength analysis. KeyWall provides this bearing pressure in addition to the factored bearing pressure for comparison.

SETTLEMENT

Settlement criteria may limit design bearing pressures for structures having large footing areas, such as mat-type foundations and bearing areas under MSE wall systems. By reviewing equation (3k), it is easy to see that with a large “B”, the shear capacity of the foundation is usually sufficient. However, with larger footing widths, the area of influence below the loaded area becomes quite large, typically 2B, and the addition of this vertical stress over a large area can induce significant settlement. It is important that the designer distinguishes between allowable bearing capacity for shear failure (a catastrophic failure mechanism) and a settlement criterion (a non-catastrophic event).

Total settlement is limited by the designer's performance criteria and impact on adjacent structures or tolerances on vertical movements. As long as the structure settles uniformly, there is no significant structural effect on the wall system. Differential settlement, however, will cause a flexural movement in the wall face and may lead to unit realignment and cracks to relieve tensile stresses in the concrete. Differential settlements typically should be limited to 1% (i.e., 1 foot in 100 feet) (NCMA) or ½% (i.e., 1 foot in 200 feet) (AASHTO).

Settlement analysis is beyond the scope of this document, and is not included in the KeyWall analysis. Due to the variability of foundation conditions, potential influences of groundwater, and other subsurface conditions, it is recommended to consult a qualified geotechnical engineer for proper analysis and specifications.

Note:

The designer is cautioned about spanning box culverts, constructing walls over uncompacted utility trenches, going directly from a rock foundation to a soil foundation, or transitioning from any elastic bearing surface to a rigid foundation surface. Differential settlements in such a short distance will be detrimental to any structure, and vertical construction slip joints are recommended.



GLOBAL STABILITY

Global stability analysis is beyond the scope of this document and is not included in the KeyWall program. However, it can be a necessary part of a comprehensive design analysis on larger projects and is best performed by the site geotechnical engineer.

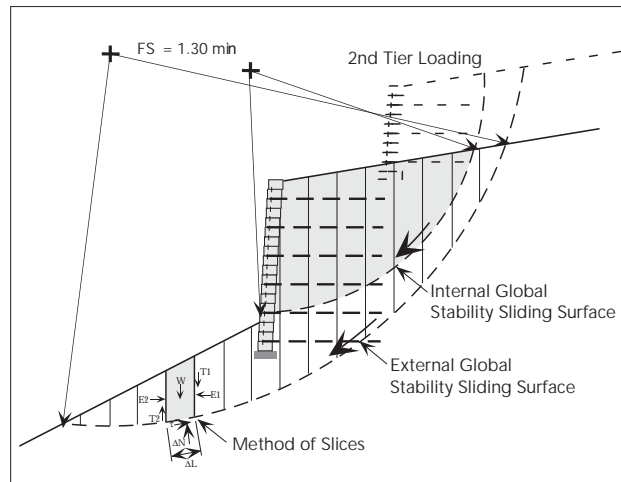


Figure 3:7 Global Stability Section

Global stability should be investigated any time the following situations occur:

- Steep slopes away from the toe of wall
- Steep slopes above the top of wall
- Tiered wall construction
- Poor foundation soils

Slope stability is a complicated analysis that depends on site geometry, construction methods, tested soil parameters and potential influence of groundwater. It is recommended that a qualified geotechnical engineer be consulted for proper analysis and recommendations.

A minimum Factor of Safety of 1.3 is required by NCMA and AASHTO. A higher factor ($FS = 1.5$) may be required for critical structures such as bridge abutments. AASHTO LRFD requires similar ratios.

INTERNAL COMPOUND STABILITY ANALYSIS

NCMA's Design Manual, 3rd Edition, introduced the concept of internal compound stability (ICS) which is a limited form of global stability analysis that checks circular failure planes through the reinforced zone for a limited set of conditions. Keystone believes that global stability analysis should be done on a comprehensive basis, when required, to avoid the limitations of the ICS analysis and does not currently include that function in KeyWall.



SEISMIC ANALYSIS

Keystone retaining wall structures have proven to be earthquake resistant due to the system's inherent flexibility that permits minor yielding during a major seismic event.

The most recent seismic design standards are contained in the AASHTO Standard Specifications for Highway Bridges (Chapter 11) and in the 3rd edition of the NCMA Design Manual, which describe a pseudo-static method of analysis based on the Mononobe-Okabe application of conventional earth pressure theory.

A schematic of pseudo-static analysis considerations is shown in Figure 3:8 below as it pertains to reinforced soil structures.

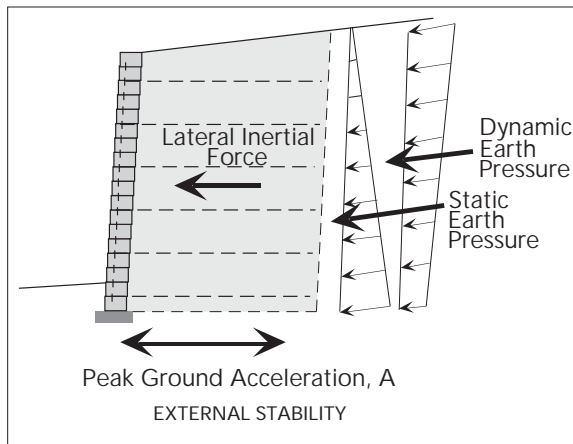


Figure 3:8 External Stability

The details of seismic analysis are beyond the scope of this manual and other documents should be consulted. There are many ways to evaluate seismic forces, which are quite complicated. The KeyWall program uses three different methods that parallel the three different design methodologies of Coulomb, Rankine, and AASHTO.

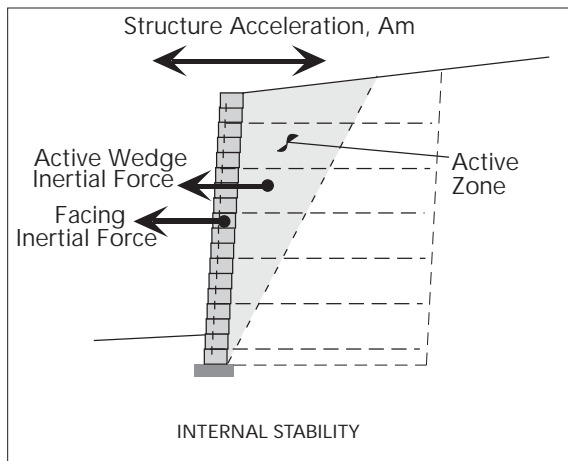


Figure 3:9 Internal Stability

PART FOUR

THE DESIGN PROCESS

High Pointe, Kelowna, British Columbia, Canada; Keystone Compac





INTRODUCTION

After discussion of the Keystone units' properties, geogrid soil reinforcement, and earth pressure theory, it's time to put the pieces together into a Keystone Wall System design. The parameters required for the design of a Keystone earth retaining structure are: choice of design methodology, Keystone unit, wall batter, wall geometry, soil types and properties, surcharge loading conditions, and reinforcement type and properties. The external stability items checked during the design include: sliding of the gravity wall, sliding at the base of the reinforced zone or along the lower layers of geogrid reinforcement, overturning about the toe, and applied bearing pressure.

The internal stability items checked during the design include: tensile strength of reinforcing, Keystone unit reinforcement connection, pullout capacity of the reinforcing beyond the theoretical failure plane, and local stability of the facing - shear and bending.

Note: For walls with slopes below the toe, potentially weak foundations, tiered walls, or tall slopes above the walls, global slope stability should be analyzed as part of a geotechnical investigation. Gross and differential settlement should also be checked as required by the structure design.

The following sections describe the steps taken in the design analysis; however, the trial and error process for actual reinforcement selection and placement is not discussed in detail. It is assumed the reader has used the KeyWall program for the proposed design and is ready to confirm the results obtained. The details of a complete design analysis are tedious and are best performed by computer and verified by hand.

Park School, Pikesville, Maryland; Keystone Standard



**Note:**

A common misuse of Coulomb theory is to calculate the lower earth pressure and then fix the failure plane at the steeper Rankine slope ($45^\circ + \phi/2$) for the purposes of the pullout resistance calculation. This is not in accordance with any established design methodology.

Note:

A common concern is that the facing and soil mass can move down together in a flexible wall system without a rigid foundation, which would eliminate the stabilizing effects of assumed wall friction on the failure wedge and the resulting lower Coulomb earth pressure.

DESIGN METHODOLOGY

It is first necessary to select and understand the design methodology that will be used for a specific project. The NCMA Coulomb methodology is based on fundamentally different principals than the Rankine and AASHTO methodologies and will provide different results in both the internal and external calculations due to these differences. The most important issue is that the designer understand and be comfortable with a design methodology and its limitations, then follow the methodology in its entirety.

The advantage of using a Coulomb earth pressure methodology is that it can provide the lowest calculated earth pressure in a given situation by taking all beneficial components into account (wall batter and wall friction). However, it also requires that the reinforcement lengths be significantly longer at the top of wall than bottom of wall due to the flatter slope of the calculated Coulomb failure plane. Also, the reduced earth pressure may permit vertical spacing of the reinforcement in lower walls that exceed the wall facing's ability to remain stable during construction and in the final configuration (facial stability at top of wall and between reinforcement levels).

The advantage of using a Rankine earth pressure methodology is that no assumption has to be made with regard to friction between the wall structure and retained soil mass. Also, the Rankine theory provides simpler formula and failure plane definitions which are easier to use and check. Rankine earth pressure theory has been the established methodology for earth reinforcement design in the public sector since the early 1970's which provides a certain level of comfort to many designers. KeyWall provides the designer choices for NCMA 2nd Edition, NCMA 3rd Edition, Rankine, AASHTO-96, AASHTO-Simplified, AASHTO-LRFD, Australian, and CAN-LRFD as the design methods.

UNIT SELECTION

To begin the design process, a selection of the preferred Keystone Unit is required. As a general guideline, for small gravity walls 3 - 6 feet high (1-2m), the Keystone Standard unit is the preferred choice. Below 3 feet (1m) high, the Compac Unit, Keystone Century Wall or 133Elite may be selected. For taller walls or walls supporting surcharge loadings, where soil reinforcement generally will be required, any of the structural units can be constructed to a desired height using the appropriate soil reinforcement design. The Standard unit is considerably more stable during construction and is preferred for the larger, more critical wall structures. The Compac, Keystone Century Wall and 133Elite units require that the soil reinforcement be placed in smaller vertical lifts due to the decreased facial stability of the smaller units and reduced connection strength. These design options can be quickly checked with KeyWall software to determine the most effective design.



WALL BATTER

Batter of the wall is the designer's or contractor's choice depending on the appearance desired, right-of-way available, and degree of wall curvature expected. Battered walls work poorly in tight curves and in sharp corners as the wall units move away or towards each other resulting in cut pieces or increasing gaps. Since the Keystone Standard, Compac, and Keystone Century Wall units can be constructed with either a "near-vertical" or 1" (25mm) minimum setback batter per course, it is recommended that only reasonably straight walls be constructed with the 1" setback and walls with tight curves and corners be constructed with the near vertical alignment to facilitate construction.

It is very important that if a batter is assumed in the design, that the wall be specified and constructed with the designed batter, due to the fact that wall batter reduces the calculated earth pressure.

Note:

With greater batter, more lateral space is required for the wall system, i.e., for a 8° batter, 2.8 feet (0.85m) of right-of way will be lost for a 20-foot (6.1m) wall height.

HINGE HEIGHT

Hinge height is a significant concept introduced by the NCMA Design Manual that accounts for wall batter on the maximum calculated normal pressure exerted at the unit base and at any reinforcement level. Hinge height was removed as a design criteria in the 3rd Edition of the NCMA Design Manual but remains a part of many specifications. Simply stated, the hinge height is that height at which the wall would topple over backward if the units were stacked without a soil backfill. Hinge height is defined by:

Equation (4a) $H = \frac{2CG}{\tan(u)}$

where:

- CG = the center of gravity of the unit from back
u = wall batter from vertical (in degrees)

This is a simplification of the NCMA manual equation and the inclination terms have been eliminated.

Hinge height defines the maximum confining pressure at any reinforcing-unit interface and defines the normal force that will be exerted at the base when calculating sliding resistance. For the Keystone units, the calculated hinge heights are as follows:

| Unit/Batter | Hinge Height Ft(m) | | | |
|-------------------|--------------------|------------|------------|----------|
| | 0° | 4° | 8° | 2.5° |
| Standard | H | 25.6'(7.8) | 12.7'(3.9) | -- |
| Standard II | H | 21.4'(6.5) | 10.6'(3.2) | -- |
| Compac I, II, III | H | 14.3'(4.3) | 7.1'(2.1) | -- |
| Century Wall | H | 14.3'(4.3) | 7.1'(2.1) | -- |
| 133Elite | -- | -- | -- | 22.9'(7) |

Note:

KeyWall provides a Design Preference setting to turn Hinge Height criteria on and off based on product requirements.



Note:
KeyWall models the slope from the Keystone Unit to crest of back slope. Beyond that, the backfill is assumed horizontal and may have a live or dead load surcharge applied on this surface.

WALL GEOMETRY

The design wall height is always measured from the top of leveling pad to the top of wall. The design height of the wall is determined from the site geometry including the appropriate embedment. Designs must be prepared for the different wall sections as defined by the site conditions. The backslope geometry is modeled by defining a slope angle (β) in degrees from horizontal and measuring the length of the slope (horizontal offset) above the top of the wall.

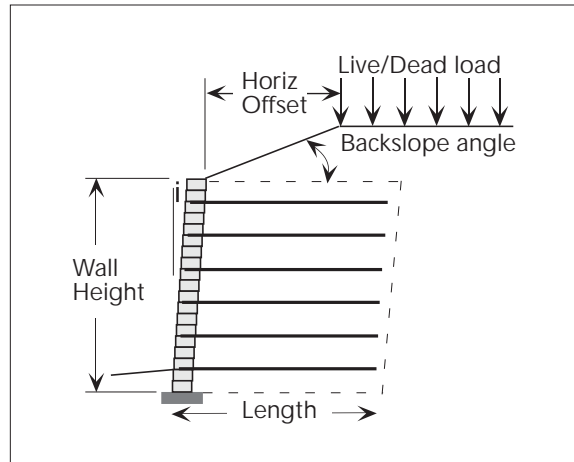


Figure 4:1 Design Section

WALL EMBEDMENT

For small Keystone gravity walls, a minimum 1-inch (25mm) of embedment is recommended for every unit of height (i.e., $H/8$) or 1 block minimum. For reinforced soil Keystone walls, the minimum depth of embedment as a ratio to wall height may be determined in the following table from the NCMA Design Manual (2009):

| Slope in Front of Wall | Min. Embedment |
|------------------------|----------------|
| Minimum Requirement | 0.5 ft (150mm) |
| Horizontal (walls) | $H/20$ |
| Horizontal (Abutments) | $H/10$ |
| 3H:1V | $H/10$ |
| 2H:1V | $H/7$ |

Note:
Project plans, specifications, and design codes may require minimum embedments that exceed the minimums recommended by NCMA.



SLOPING TOE

The minimum embedment required with a slope in front of the wall should be based on the establishment of a minimum 4 ft (1.2m) horizontal bench in front of the wall and establishing a minimum embedment from that point. Fill slopes usually have poor compaction near the edge of slope and all slopes are subject to erosion and surficial instability.

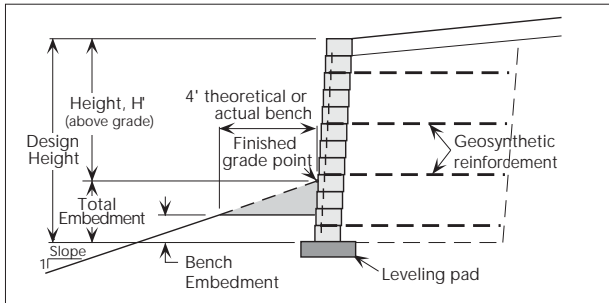


Figure 4:2 Sloping Toe

The depth of embedment should be increased when any of the following conditions occur:

- Weak bearing soils
- Potential scour of wall toe
- Submerged wall applications
- Significant shrink/swell/frost properties of foundation soils

SOIL PROPERTIES

The purpose of a retaining wall system is to safely hold a wedge of soil in place to make a grade change in the shortest possible distance. The angle of internal friction (ϕ), cohesion (c), and unit weight (γ) of the soils determine the force that will be exerted by the soil wedge on the wall structure. The figures 4:3 & 4:4 describe a simple shear test and test data plot that describes the soil strength properties. Some typical design ϕ and γ ranges for compacted or dense soils are shown in the Shear Strength and Weight Range Table.

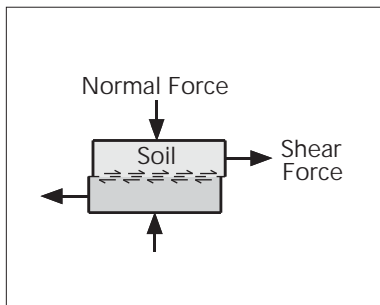


Figure 4:3 Soil Shear Test

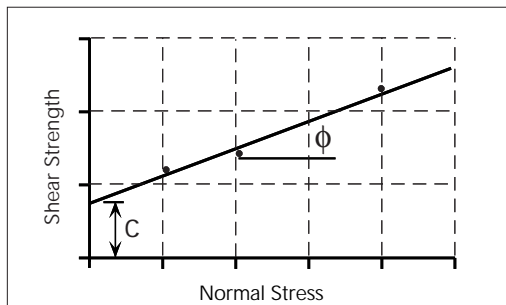


Figure 4:4 Soil Shear Plot

Note:

The required embedment depth for Keystone walls may become a controversial issue. The Uniform Building Code (UBC) recommends a 1' minimum or below prevailing frost depth, which ever is greater for foundations. AASHTO recommends a 2' minimum or below prevailing frost depth which ever is greater for retaining structures. These minimum recommended depths are based on rigid foundation systems and are not totally applicable to flexible systems, which function properly with significantly less embedment. The proper embedment depth is a function of the structure size and type, the underlying soils, and the site geometry, especially toe slopes. It is significantly more important to properly inspect the foundation area when excavated, determine the limits of removal and replacement of unsuitable materials, and then confirm the final embedment depth for stability and bearing given the site conditions.



SOIL PROPERTIES

| Soil Type | ϕ - Angle | γ - Weight |
|------------------------|-----------------------|--|
| Crushed Stone, Gravel | 34° + | 110-135 PCF (17-21 kN/m ³) |
| Sands | 30°-34° | 100-130 PCF (16-20 kN/m ³) |
| Silty Sands/Sandy Silt | 28°-30° | 100-125 PCF (16-20 kN/m ³) |
| Sandy Clay, Lean Clay | 26°-28° | 100-120 PCF (16-19 kN/m ³) |
| Other Clays | Determined by Testing | |

Shear Strength and Weight Range

A qualified geotechnical engineer should be consulted to establish the soil properties for a site. Reasonable design values can usually be estimated by a qualified engineer based upon visual observation and history of the soils encountered. Additional soil borings and laboratory testing may be required for taller walls or difficult site soil conditions.

SURCHARGE

A surcharge is a loading imposed on the soil behind the wall that exerts an additional force on the potential failure zone. For simplification in the KeyWall program, all surcharge loadings are assumed to be uniform live or dead loads and extend only on a horizontal surface. Line and point surcharge loads are not within the scope of this manual. Typical live load surcharge loadings are:

- Landscaping walls, -- 0 psf
- Pedestrian traffic, light storage, -- 50 psf (2.4 kPa)
- Light-traffic, auto parking -- 100 psf (4.8 kPa)
- Highway loading, heavy traffic -- 250 psf (12 kPa)

To model surcharge loading with sloping backfill conditions, KeyWall models the surcharge at the top of the slope on the horizontal surface (See Figure 4:1). If the surface is level, but the surcharge is a short distance back from the wall face, the designer may input a horizontal offset to move the load away from the back of wall units.

Surcharge live loads are used in the external stability analysis as driving forces, but are not included as resisting forces.

For heavy loadings due to equipment, railroads, footings, closely spaced tiers, etc., a Boussinesq stress distribution may be more applicable. The designer should analyze the wall with the appropriate uniform surcharging from imposed dead loads, then super-impose the earth pressure diagram for line or strip loads by hand or with the aid of a spread sheet analysis.



REINFORCEMENT TYPE & PROPERTIES

Geosynthetic reinforcement for retaining walls are generally geogrids specifically designed and tested for use as soil reinforcement. The basic design criteria is covered in part three of this Manual.

The selection of polymer type or manufacturer of the reinforcement is a subjective determination based upon the specific design considerations of a project. Each type of reinforcement can be used safely provided that the appropriate durability, installation damage, and long term creep factors are determined for a given application based on field and laboratory test data.

Each manufacturer should be able to provide test documentation for the recommended values contained in their product's technical literature. It is the designer's responsibility to evaluate the manufacturer's product test data and determine the grid types and design values appropriate for the project.

The strength level of reinforcement to be used in a wall design is a function of wall height and loading. Lower walls will generally use lower strength reinforcement, while taller walls will require stronger reinforcement. The designer may utilize lower strength reinforcement spaced closer together instead of higher strength reinforcement or different strengths within the same wall section to meet the design requirements. Construction and cost considerations typically govern this selection process.

SOIL REINFORCEMENT LENGTH

Irrespective of the design computations, minimum base to height proportions for MSE walls have been developed based on history and successful field performance.

In accordance with the NCMA Design Manual, the minimum reinforcement length shall be 0.6H for all wall applications. Per AASHTO, the minimum length shall be 0.7H or 8 foot minimum (2.44m), whichever is greater (AASHTO LRFD has provisions for 6' min. length (1.8m) under certain conditions). The minimum length shall be as stated or as required for external stability, whichever is greater. All lengths represent the depth of the reinforced mass to resist external forces; therefore, all lengths are measured from the front face of the wall system to the back of the reinforcement.

NCMA further recommends that the soil reinforcement extend beyond the Coulomb failure plane a minimum of 1 foot (300mm). AASHTO recommends that the reinforcement extend 3 feet (1m) past the Rankine failure plane.

The choice of 60% or 70% of height for minimum reinforcement length is one of design specification and the designer's preference. There is considerable evidence that walls experience greater deformation with shorter reinforcement "L/H" ratios ($L/H < 0.5H$).

AASHTO requires that reinforcement be uniform in length, while NCMA permits varying reinforcement lengths as required by the internal and external stability calculations. Keystone recommends providing uniform reinforcement lengths within a wall section as a general rule to avoid excessive detailing on the plans and installation confusion by the contractor.

Note:

The external stability analysis of a reinforced soil structure is similar for the Coulomb/NCMA and Rankine/AASHTO methodologies except that the Coulomb/NCMA method neglects the vertical component of the external forces whereas the Rankine/AASHTO methods include the stabilizing effects of the vertical component for sloping backfills, $\beta > 0$.

The Rankine and AASHTO methods use essentially the same external analysis method. The only differences relate to AASHTO design code recommendations such as minimum L/H ratio and minimum reinforcement length requirements.



EXTERNAL STABILITY ANALYSIS

External stability is the wall structure's ability to resist external sliding and overturning forces and the foundation's ability to support the structure. The wall system must be proportioned to provide adequate safety against applied soil and surcharge loads.

A typical external force analysis for a simple gravity wall is shown in Figure 4:5. Gravity walls rely solely on the mass of the facing units to resist external forces.

Note:

Live load surcharges are not typically applied on slopes. KeyWall only applies surcharges to the level surface behind walls.

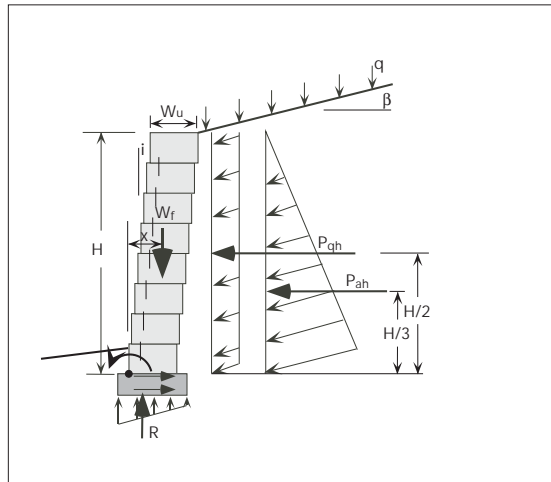


Figure 4:5 Gravity Wall Force Diagram

A typical external force analysis for a simple reinforced soil wall is shown in Figure 4:6. The reinforced soil section is treated as a coherent gravity mass and analyzed externally as a rigid body similar to the gravity wall.

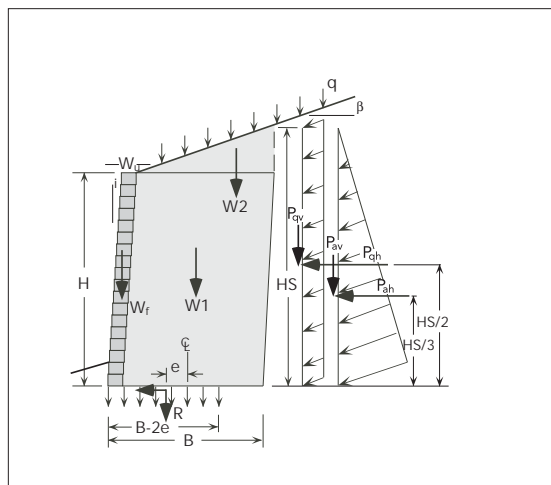


Figure 4:6 Reinforced Wall Force Diagram



BATTERED WALL DESIGN OPTIONS

Battered walls can create some problems in the design analysis since the geometry becomes more complex than vertical walls and the application of weights and loads is not as clear.

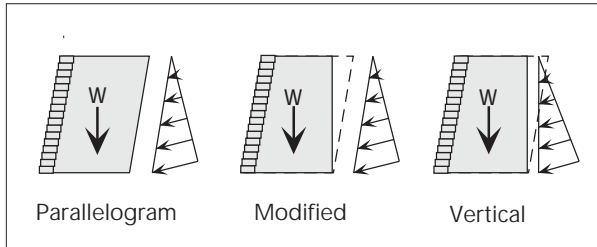


Figure 4:7 Design Options

Parallelogram

The parallelogram method calculates the weight of the battered reinforced mass over the base as indicated including the wedge of soil that is not over the base. Active earth pressure is determined based on the batter of the reinforced zone interface with the retained soil. This is the basis for NCMA Design Manual stability analysis.

Modified

The modified method calculates only the weight of the battered reinforced mass that is over the base as indicated. Active earth pressure is determined based on the batter of the reinforced zone interface with the retained soil. This is the basis for Rankine stability analysis.

Vertical

The vertical method calculates only the weight of the battered reinforced mass that is over the base as indicated. Active earth pressure is determined based on a vertical interface between the reinforced zone and the retained soil. This is the basis of FHWA software although external stability of battered MSE walls is not addressed in FHWA or AASHTO documents.

AASHTO LRFD

Load Resistance Factor Design (LRFD) is the next step in the evolution of US engineering design practice. MSE retaining walls have traditionally been designed (and still are) using allowable stress design (ASD) and factor of safety (FS) methods. Concrete structures have been designed using LRFD methods for many years followed by steel structure design adopting similar methods. The difference between LRFD and ASD/FS is in how the uncertainties of the design are handled. Allowable stress design uses a single variable, factor of safety, to handle uncertainties. The general form of ASD is shown below:

Equation (4b)
$$\frac{R}{P} \geq FS$$

where:

- R = resistance (stabilizing forces)
- P = load (destabilizing forces)
- FS = factor of safety



AASHTO LRFD

Load resistance factor design uses multiple variables to handle uncertainties. Load factors are applied to each of the different types of loads: live, dead, horizontal and vertical. Resistance factors are applied to the nominal resistance. The load factor and resistance factors are set based on statistical data and can better represent the uncertainties compared to ASD/FS design methods. The general form of LRFD is shown below.

$$\text{Equation (4c)} \quad \text{CDR} = \frac{\phi R}{\gamma_{t1} P_{t1} + \gamma_{t2} P_{t2}} > 1.0$$

where:

- ϕ = resistance factor
- R = resistance (stabilizing forces)
- γ_{t1}, γ_{t2} = load factor for a certain load type
- P_{t1}, P_{t2} = load of a certain type (destabilizing forces)
- CDR = capacity demand ratio

Given the nature of retaining wall design where certain loads contribute to the calculation of the resistance, R , load factors are also used to compute the nominal resistance. For example, in the sliding calculation the weight of the reinforced zone contributes to the resisting force. A resisting load factor is applied to this load. AASHTO lists both driving load factors (maximum) and resisting load factors (minimum) in chapter 3 of the 2010 Standard Specifications for Highway Bridges. The applicable load factors are shown in the table below:

Strength I

| Driving Load Factors | Resisting Load Factors |
|----------------------|------------------------|
| $EH_d = 1.50$ | $EH_r = 0.90$ |
| $EV_d = 1.35$ | $EV_r = 1.00$ |
| $ES_d = 1.50$ | $ES_r = 0.75$ |
| $LL_d = 1.75$ | |

AASHTO LRFD Load Factors

Extreme I (Seismic)

| Driving Load Factors | Resisting Load Factors |
|----------------------|------------------------|
| $EH_d = 1.50$ | $EH_r = 0.90$ |
| $EV_d = 1.35$ | $EV_r = 1.00$ |
| $ES_d = 1.50$ | $ES_r = 0.75$ |
| $LL_d = 0.50$ | $LL_r = 0.00$ |
| $EQ = 1.00$ | n/a |

Resistance factors for the external and internal failure mechanisms are listed below:

| Strength I | Extreme I |
|--------------------------|--------------------------|
| Sliding $RF_{sl} = 1.00$ | Sliding $RF_{sl} = 1.00$ |
| Overturning = NA | Overturning = NA |
| Bearing $RF_b = 0.65$ | Bearing $RF_b = 0.65$ |
| Tension $RF_t = 0.90$ | Tension $RF_t = 1.20$ |
| Pullout $RF_{po} = 0.90$ | Pullout $RF_{po} = 1.20$ |

AASHTO LRFD Resistance Factors



SLIDING ANALYSIS

The sliding resistance of a Keystone gravity wall is calculated by determining the sliding resistance between 1) the wall unit and the leveling pad material interface, 2) through the leveling pad material, and 3) unit to unit shear above the leveling pad as indicated in Figure 4:8.

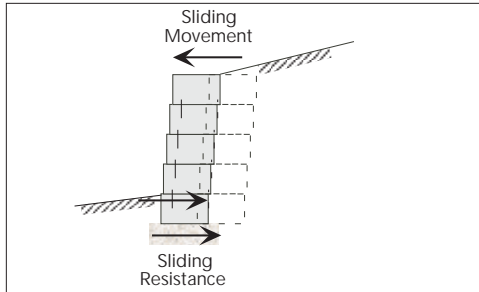


Figure 4:8 Gravity Wall Sliding

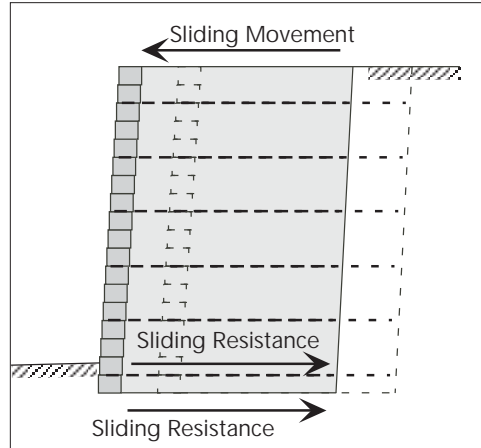


Figure 4:9 Reinforced Wall Sliding

The sliding resistance of a Keystone reinforced soil wall is calculated by determining the sliding resistance between 1) the reinforced soil zone and the foundation soil interface and 2) through the reinforced wall system along a reinforcement level as indicated in Figure 4:9.

For both gravity and reinforced walls, the driving force is calculated from Equations 3a and 3b for Coulomb active earth pressure and Equations 3e and 3f for Rankine active earth pressure.

The ratio of resisting forces to driving forces is calculated to determine a Factor of Safety against Sliding:

Equation (4d)
$$FS_{sl} = \frac{\Sigma \text{Resisting Forces}}{\Sigma \text{Driving Forces}}$$

Alternatively, AASHTO LRFD computes a sliding capacity demand ratio:

Equation (4e)
$$CDR_{sl} = \frac{\Sigma \text{Resisting Forces (factored)}}{\Sigma \text{Driving Forces (factored)}}$$

Note:

Gravity walls rarely fail in sliding as the overturning calculation generally controls the maximum design heights possible. On the other hand, reinforced soil structure design is typically proportioned based on sliding resistance and overturning rarely controls the design.



COULOMB - NCMA-SLIDING

Driving Forces

The horizontal earth pressure components, $(P_a + P_q) \cos(\delta)$, are the driving forces.

Resisting Forces

Gravity wall analysis calculates inter-unit shear, unit to leveling pad shear, leveling pad shear resistance based on the weight of wall, W_f .

Reinforced soil wall analysis calculates soil to soil sliding resistance $(W_f + W_1 + W_2) \tan \phi$ of the weaker soil (reinforced or foundation) as the resisting force. The 3rd edition of the NCMA Design Manual now permits the inclusion of vertical earth load components at the designer's option.

RANKINE - AASHTO-SLIDING

Driving Forces

The horizontal earth pressure components, $(P_a + P_q) \cos(\beta)$, are the driving forces in ASD/FS analysis.

The factored horizontal earth pressure components, $(EH_d P_a + ES_d \text{ or } LL_d P_q) \cos(\beta)$, are the driving forces in LRFD analysis.

Resisting Forces

Gravity wall analysis must calculate inter-unit shear, unit to leveling pad shear, and leveling pad shear resistance.

Reinforced soil wall analysis calculates sliding resistance as $(W_f + W_1 + W_2 + P_{av} + P_{qv}) \tan \phi$ of the weaker soil in ASD/FS analysis.

The factored sliding resistance is $RF_{sl} (EV_f W_f + EV_f W_1 + EV_f W_2 + EH_d P_{av} + ES_r P_{qv} \text{ (dead)}) \tan \phi$ of the weaker soil in LRFD analysis.

The reinforced soil wall sliding analysis becomes more complicated with geosynthetic sheet reinforcement because sliding must be checked along the lowest levels of reinforcement, as well as at the base of the mass (see Figure 4:10).

Note:

Live load does not contribute to resisting forces.



RANKINE - AASHTO-SLIDING

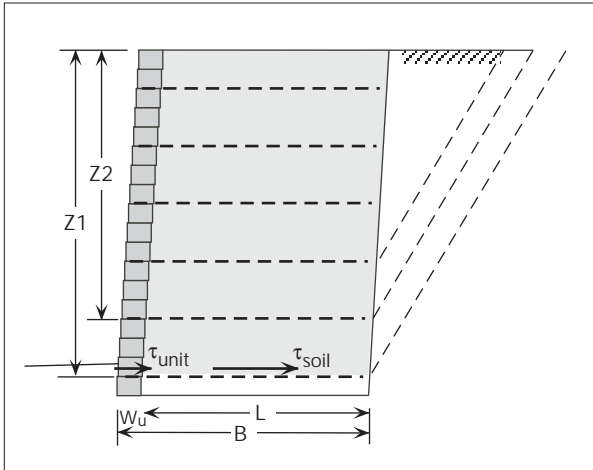


Figure 4:10 Internal Sliding Length

Driving forces are recalculated for each reinforcement level in the same manner as the external analysis.

The resisting force is calculated as the sum of the inter-unit shear and soil to geogrid interface shear:

$$\tau_{\text{unit}} = \text{from shear curve for unit with geogrid}$$

$$\tau_{\text{soil}} = \text{Weight} \times \tan \phi \times C_{ds}$$

For both gravity and soil reinforced structures, a minimum factor of safety of 1.5 against sliding is required in ASD analysis. A minimum CDR of 1.0 is required in LRFD analysis.



OVERTURNING ANALYSIS

Overturning is the wall's theoretical tendency to tip over due to lateral pressures exerted by the soil and any surcharge loading at the back of the wall system. In gravity wall design, overturning is a major design consideration since the units are rigid and have a small L/H ratio at relatively short heights.

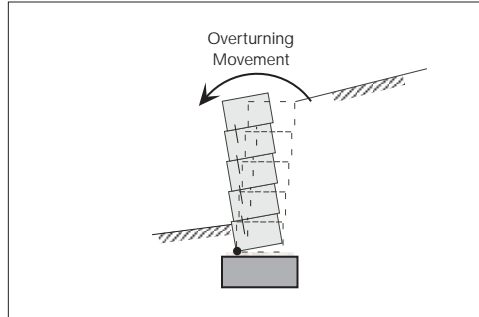


Figure 4:11 Gravity Wall Overturning

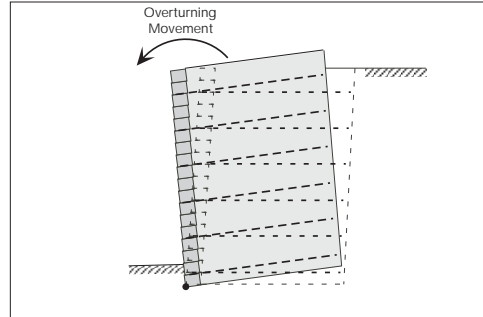


Figure 4:12 Reinforced Wall Overturning

In reinforced wall design, this “theoretical” overturning is not possible because the reinforcing is typically designed for a minimum L/H ratio of 60% or greater and the wall system is a flexible soil mass which cannot overturn.

The driving or overturning moment is the result of active earth pressure forces and surcharge forces pushing at the back of the wall system. Referring to Design Theory in Part Three, the active earth pressure force is a triangular pressure distribution with the maximum force at the base and the centroid is at $\frac{1}{3}$ of the height. The surcharge load is a rectangular pressure distribution against the back of the wall system and the centroid of the rectangle is at $\frac{1}{2}$ the height.

For both gravity and reinforced walls, the driving force is calculated from Equations 3a and 3b for Coulomb active earth pressure and Equations 3e and 3f for Rankine active earth pressure located in Part Three of this Manual.

The ratio of resisting moments to driving moments is calculated to determine a factor of safety against overturning.

Equation (4f)
$$FS_{OT} = \frac{\Sigma \text{Resisting Forces}}{\Sigma \text{Driving Forces}}$$

Alternatively, AASHTO LRFD computes an overturning capacity demand ratio:

Equation (4g)
$$CDR_{OT} = \frac{\Sigma \text{Resisting Forces (factored)}}{\Sigma \text{Driving Forces (factored)}}$$



COULOMB - NCMA OVERTURNING

Driving Moments

The horizontal earth pressure components, $(P_a + P_q) \cos(\delta)$, are the driving forces at their respective moment arms of $H/3$ or $HS/3$ and $H/2$ or $HS/2$ up from the toe.

Resisting Moments

Gravity wall analysis calculates the weight of the facing system, W_f , times the moment arm from toe to center of gravity of the facing column.

Reinforced soil walls calculate the weight of the entire system (W_f , W_1 , W_2) at their respective moment arm from the toe to each center of gravity as the resisting moment.

RANKINE - AASHTO OVERTURNING

Driving Moments

The horizontal earth pressure components, $(P_a + P_q) \cos(\beta)$, are the driving forces at their respective moment arms of $H/3$ or $HS/3$ and $H/2$ or $HS/2$ up from the toe in ASD analysis.

The factored components, $(EH_d P_a + ES_d \text{ or } LL_d P_q) \cos(\beta)$, are the driving forces in LRFD analysis.

Resisting Moments

Gravity wall analysis calculates the weight of the facing system, W_f , times the moment arm from toe to center of gravity of the facing column in ASD analysis. AASHTO LRFD uses $EV_r W_f$ for the resisting forces.

Reinforced soil walls calculate the weight of the entire system (W_f , W_1 , W_2) P_{av} , P_{qv} at their respective moment arm from the toe to each center of gravity as the resisting moment.

AASHTO LRFD uses $(EV_r W_f, EV_r W_1, EV_r W_2, EH_d P_{av}, ES_r P_{qv})$, for resisting forces.

Note:

Live load does not contribute to resisting forces.

OVERTURNING

For soil reinforced structures, a 2.0 minimum factor of safety against overturning is required in ASD analysis. For gravity walls, a 1.5 factor of safety against overturning is typically required. LRFD analysis may or may not look at overturning ($CDR > 1.0$) and rely on eccentricity criteria to limit overturning.

BEARING CAPACITY

Bearing capacity is the capacity of the foundation soil to support the load imposed by the wall system without shear failure or excessive settlement as depicted in Figures 4:13 and 4:14.

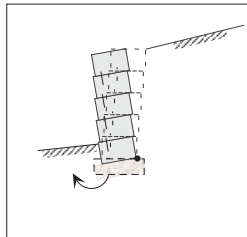


Figure 4:13 Gravity Wall Bearing Failure

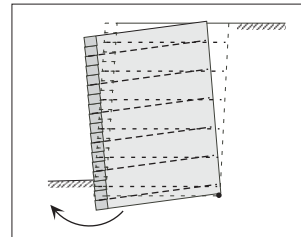


Figure 4:14 Reinforced Wall Bearing Failure

Note:

The live load surcharge is included as a driving force and not as a stabilizing force. Only permanent forces within the wall are included as stabilizing forces.



ULTIMATE BEARING CAPACITY

The ultimate bearing capacity of the foundation is a function of the soil shear strength (ϕ and c), the embedment depth below grade, and the bearing surface's effective width ($B-2e$) in accordance with the equation and factors discussed in Part Three.

In Figures 4:11 & 4:12, overturning, the resisting moments, and the driving moments are calculated for the section being analyzed. Those moments are analyzed here again. However, since the live load surcharge (if over the structure) is a destabilizing force, it is included in the driving moment term as one of the resisting reactions. The external driving moments remain the same. The equations previously provided in Part Three, and Figure 3:5 provide the calculated applied bearing pressure, σ_v , and the equivalent footing width, $B-2e$.

Note:

Rigid footing systems typically require a bearing capacity safety factor of 3.0 or lower resistance factors.

A minimum 2.0 (NCMA) or 2.5 (AASHTO) factor of safety is required for bearing capacity for reinforced soil wall systems in ASD analysis. A $CDR > 1.0$ is required in LRFD analysis based on a resistance factor of 0.65.

A second criteria for bearing capacity is settlement. Settlement, particularly differential settlement, should be evaluated by a qualified engineer. For reinforced soil systems, maximum allowable differential settlement is limited to 1% (NCMA) or 1/2% (FHWA). Settlement is evaluated on a service state basis (RF and $LF = 1.0$) in LRFD analysis.



INTERNAL STABILITY ANALYSIS

Internal stability is the ability of the reinforced mass to maintain its structure and resist the applied loads without deforming or failing. For a concrete cantilever wall, internal stability is provided by a combination of the stems bending and shear resistance at the footing and up the stem. In a crib or gabion system, internal stability is the dead weight and ability of each lift to resist sliding and overturning about the layer below. In soil reinforced wall system, it is the tensile and pullout capacity of the reinforcing elements and inter-unit shear/connection capacity that holds the potential wedge of soil in place. Sliding and shear are also evaluated internally to ensure the mass will not fail in internal shear.

The retaining wall mass, or structure, is composed of the Keystone units at the face combined with reinforcing elements extending back beyond the Coulomb or Rankine failure plane.

The Elements of Internal Design are to ensure:

- 1) The tensile elements do not exceed their working stress or factored resistance limits.
- 2) The tensile elements have adequate connection capacity to the Keystone units.
- 3) The tensile elements have adequate anchorage beyond the potential failure plane to hold the wedge of soil in place.
- 4) There is not a potential surface where the mass can shear internally.
- 5) The facing is stable against potential shear, bulging, and overturning.



Note:

Due to the spacing algorithm used in KeyWall, some of the reinforcing layers may be slightly over-stressed in the trial design section. It is our opinion that a slight overstress is acceptable as long as the layers above and below the layer in question are below the allowable stress level.

TENSILE CAPACITY

Tensile failure occurs when the long-term design strength of the reinforcement is exceeded and leads to tensile failure of the elements. A factor of safety or load resistance factor is incorporated into the design to keep the calculated applied stress safely below the rupture limit.

A factor of safety is typically applied as a reduction to the Long-Term Design Strength (LTDS) of the reinforcement in allowable stress design. In Limit State Design, load factors increase the applied loads and masses above their actual values and compare to the LTDS, similar to AASHTO LRFD. Both methods achieve essentially similar results. The load factor design method allows factoring of various applied loads (i.e., live load versus dead load) and materials by factors, depending on their variability and potential effect on the design.

Using the concept of “the sum of the parts equals the whole,” theoretical earth pressure stresses in each element can be isolated and calculated as an applied load. The “whole”, in this case, is the total internal stress within the reinforced zone. The internal pressure for earth pressure and surcharge are superimposed on the reinforcement levels as shown in Figure 4:15 using similar Coulomb and Rankine earth pressures as calculated for the reinforced soil type:

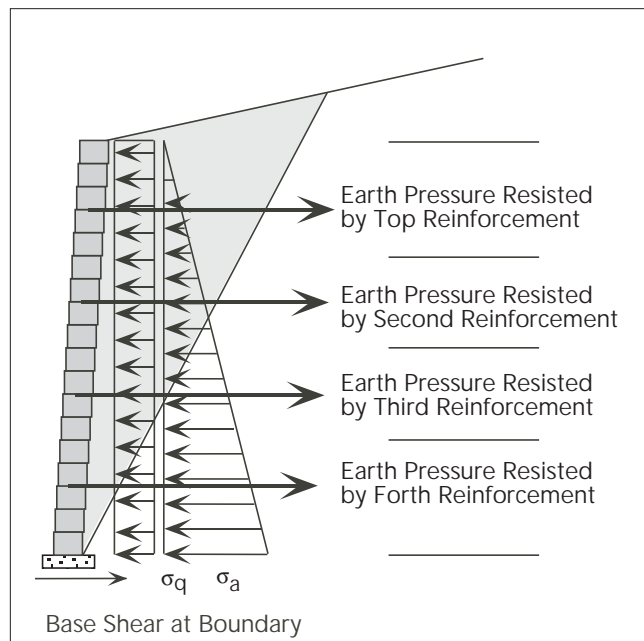


Figure 4:15 Internal Stress Distribution



TENSION LEVEL CALCULATION

Each individual reinforcement level can be broken down into respective tributary areas as shown in Figure 4:16.

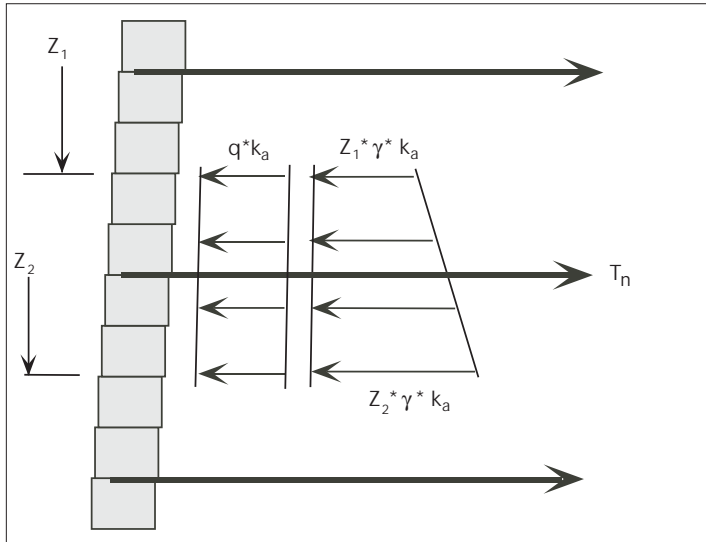


Figure 4:16 Tension Level Calculation

The internal horizontal pressure for surcharge and earth pressure are applied to the tributary area of each reinforcement. A simple equation can be setup that calculates the load per length of wall per reinforcement layer:

Equation (4h) $T_n = ((Z_1 + Z_2) / 2 * \gamma * k_a + q * k_a) * (Z_2 - Z_1)$ - Coulomb, Rankine, AASHTO

Equation (4i) $T_n = (EV_d (Z_1 + Z_2) / 2 * \gamma * k_a + EV_d * q * k_a) * (Z_2 - Z_1)$ - AASHTO LRFD

The calculated tension in each layer of reinforcement should be less than the maximum allowable design strength, T_{al} , of the specified reinforcement type at that level, allowable stress or LRFD.

Note:

KeyWall has an option to incorporate base friction at the foundation boundary as an additional stabilizing force which reduces the calculated load in the lowest reinforcement level. This analysis is consistent with model testing and finite element analysis which indicate a rapid reduction in load at the bottom of a structure. This “feature” is not recognized in the NCMA and AASHTO design specification so some caution must be used when applying this option in the Design Preferences.



AASHTO INTERNAL TENSION

The consensus in the present AASHTO codes is that extensible reinforcement permits enough strain (less stiffness) to permit simple active earth pressure design in accordance with Rankine Earth pressure theory.

AASHTO currently follows a “simplified” method for all reinforcement systems that still utilizes simple Rankine earth pressure methods, but treats a sloping backfill as an equivalent uniform surcharge on a level backfill per Figure 4:17.

The internal design for a sloping backfill calculates the internal earth pressure coefficient, k_a , for a level backfill condition and then adds the average equivalent surcharge for the sloping fill on top of the wall. This analysis tends to increase the load near the top of wall and reduces the load near the bottom. KeyWall permits three AASHTO design methods. AASHTO 96 uses the conventional Rankine analysis for sloping fills and AASHTO Simplified/AASHTO LRFD uses the equivalent surcharge method for slopes. External stability computations remain the same for all methods.

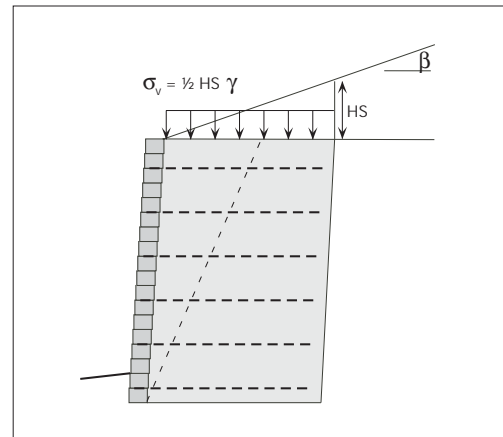


Figure 4:17 AASHTO Simplified Model

CONNECTION CAPACITY

As stated in Part Three, the tensile load in the reinforcing may be limited by, 1) Tensile capacity of the reinforcing based on material strength or 2) Connection capacity at peak connection load.

To check the connection capacity at any level, determine the capacity of the reinforcing-unit connection as a function of the normal force, N , as limited by the hinge height criteria. The normal force, N , is equal to:

Equation (4j)
$$N = h_i \gamma_{\text{unit}} W_u$$

where:

- h_i = depth to unit or hinge height, whichever is less
- γ = unit weight of the Keystone unit
- W_u = width of the Keystone unit

The designer must refer to the laboratory test curves to determine the connection capacity based on the unit and reinforcing type. All tensions should be below the connection capacity, T_{conn} , as determined from the curves and discussed in Part Three of this Manual.



PULLOUT CAPACITY

Pullout capacity is the amount of available reinforcing pullout force to withstand the outward forces of the soil wedge. The length of the reinforcing behind the Coulomb or Rankine failure plane is defined as the embedment length. As the failure plane approaches the top of the wall system, the embedment length beyond the failure plane is reduced to the point where the reinforcement may have to be longer than the lower levels to achieve adequate pullout resistance.

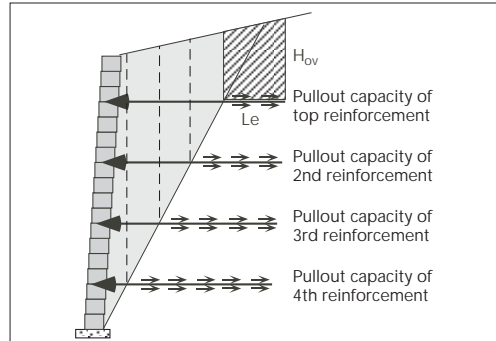


Figure 4:18 Pullout Diagram

Pullout capacity is checked at each reinforcing layer by calculating the average overburden height, H_{OV} , and the embedment length, L_e , per Figure 4:18 with the appropriate earth pressure theory. The formula for calculating the pullout resistance is defined as follows:

Equation (4k) $Pullout = (2L_e)(\gamma H_{OV})(\tan \phi C_i)$

$$Pullout = \alpha (2L_e)(E_v \gamma H_{OV})(\tan \phi C_i) \text{ AASHTO LRFD}$$

where:

- L_e = length of reinforcing beyond the Coulomb or Rankine failure plane
- γH_{OV} = average vertical pressure on the reinforcing in the pullout zone
- $\tan(\phi)$ = shear strength of soil
- C_i = interaction coefficient of the reinforcing
- α = scale effect correction factor

The “2” multiplier is included since the reinforcing is providing pullout resistance from both sides, (i.e., above and below) and is how the C_i coefficient is evaluated.

The pullout capacity is checked at all reinforcing layers. The factor of safety for pullout is given as:

Equation (4l) $FS_{pullout} = \frac{\text{Pullout Resistance}}{\text{Geogrid Load}}$

Alternatively, AASHTO LRFD computes a pullout capacity demand ratio:

Equation (4m) $CDR_{PO} = \frac{\text{Pullout Resistance (factored)}}{\text{Geogrid Load (factored)}}$



PULLOUT CAPACITY

The AASHTO LRFD pullout resistance factor, $RF_{po} = 0.90$. As is always the case in LRFD, the minimum pullout capacity demand ratio is 1.0.

If a layer of grid is failing in pullout, the designer may insert additional layers of reinforcement, increase the grid length(s), or lower the upper most layer of reinforcing to provide more overburden pressure.

NCMA recommends that all reinforcement extend a minimum of 1 foot (300mm) beyond the theoretical failure plane. AASHTO recommends a 3 foot (1m) minimum embedment length.

STABILITY OF FACING

Inter-unit shear capacity is discussed in Part One, as a resisting force when looking at sliding failures along the reinforcing planes.

In gravity wall design, inter-unit shear is the only resisting force holding the wall from sliding at any elevation above the base.

In reinforced structures, the stability of the facing must be checked for overturning above the top geogrid level, adequate shear resistance at the reinforcement levels, bulging/bending between reinforcement levels, and stability during construction. Figure 4:19 below shows the local stability loading condition that is analyzed by the KeyWall program. The inter-unit shear capacity of the units is a function of overburden height (N_1 or N_2) limited by the hinge height criteria for battered walls as discussed in Part Four. This resistance is compared to the maximum shear at each reinforcement connection, $(T_1 \text{ or } T_2)/2$.

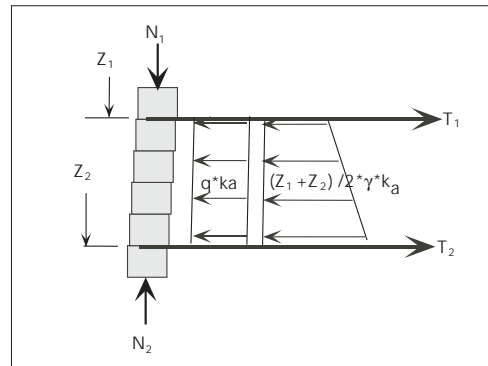


Figure 4:19 Shear & Bulging Stability

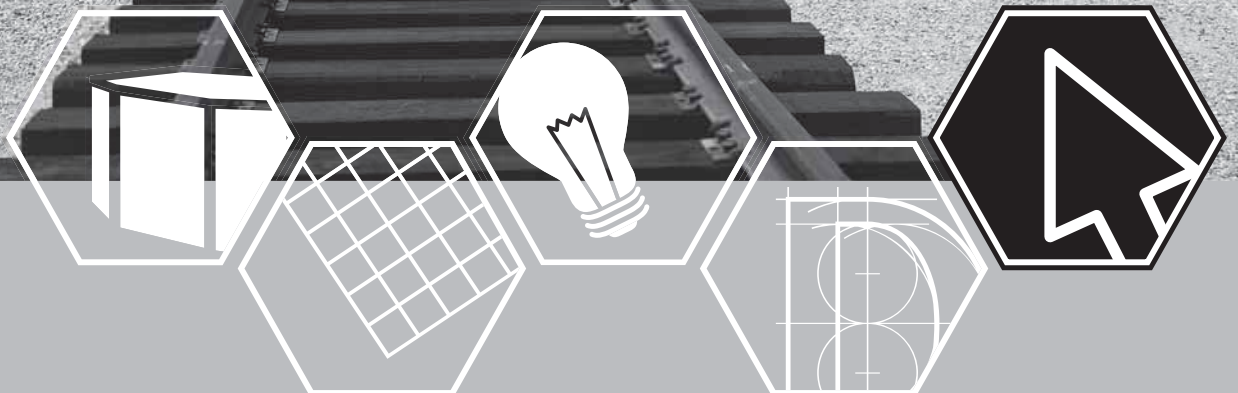
The Keystone unit resistance to bulging is based on the front of the wall not coming into tension as the earth pressure or span between reinforcement levels is increased. A simple moment formula, $M = WL^2/8$ for a continuous wall structure, is used to determine the moment couple that must be resisted by the facing in a vertical wall. The bulging moment is resisted by the compressive force in the wall section at any depth, N_1 and N_2 . Wall batter creates a less stable situation under this analysis as it introduces an additional moment couple into the facing load.

The factor of safety of shear at a unit interface and the resistance to bulging should be greater than 1.5 or a $CDR > 1.0$ in LRFD analysis.

PART FIVE

KEYWALL™ OPERATING INSTRUCTIONS

Vestas, Pueblo, Colorado; Keystone Compac





INSTALLATION & USER'S GUIDE

KeyWall design software is currently available as a Windows™ software program that will run under Windows XP, Windows VISTA, and Windows 7 operating system. Mac (Intel Processor) users should use PC software such as Parallels™ or VMware Fusion® to run KeyWall.

KeyWall takes advantage of the graphical interface and permits easy input and selection using a mouse or key commands. KeyWall will output correctly to most printers supported by the Windows operating system including network printers. Additionally, KeyWall gives the options to print an electronic PDF file.

REGISTRATION

KeyWall is provided on a demonstration basis until registered. KeyWall will not print in the demo mode, but the user will still be able to utilize all design functions. Registration is free and only requires contacting Keystone for a registration code. Note: KeyWall is programmed to expire each year and will require a new registration code. This feature limits the usage of old versions of the software with out-of-date data. See Keywall Registration later in these instructions.

KeyWall for Windows.





INSTALLATION

KeyWall is provided on the Keystone Technical CD, and is installed with a standard Windows Setup routine to simplify installation and initialize the program. The installation steps for the Microsoft Windows XP operating system are illustrated in the following instructions. The instructions assume familiarity with the Windows operating system.

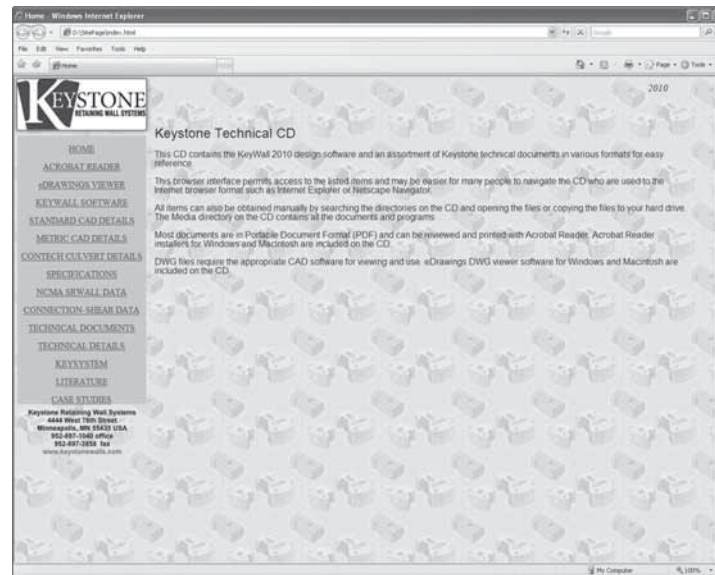


Figure 5:1 KeyWall Technical CD Homepage

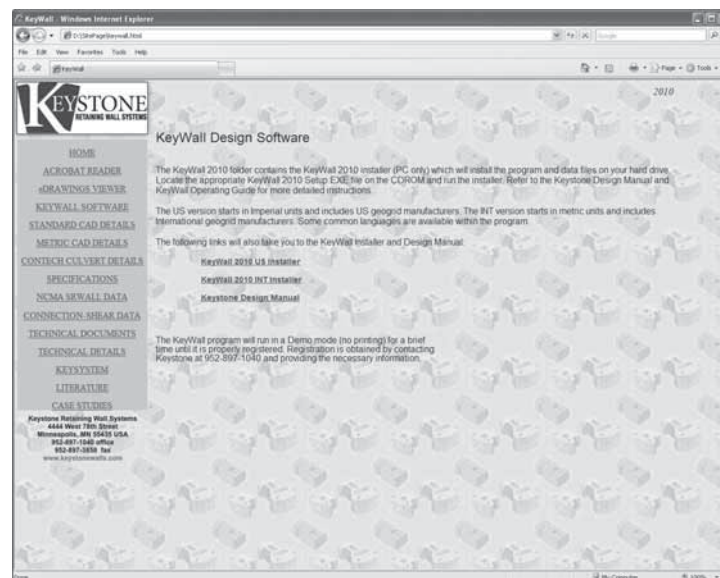


Figure 5:2 KeyWall for Windows



INSTRUCTIONS

Instructions - Keystone Technical CD

- 1 Insert **Keystone Technical CD** in CDROM.
- 2 CD will automatically start up **Microsoft Internet Explorer** (Auto Play must be on). This will provide a simple network browser interface to the files on the CD. Click the KeyWall Software link on the left.
- 3 Click the desired KeyWall installer: US or International. A file download window will open. Press run and the installation procedure will begin. If browser does not automatically start up, access the TechCD file through My Computer. Browser will open when you double-click the Tech CD file.

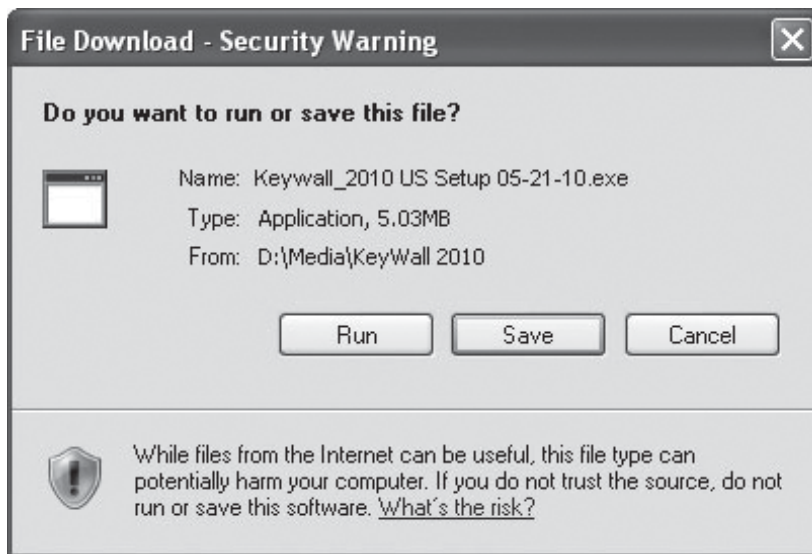
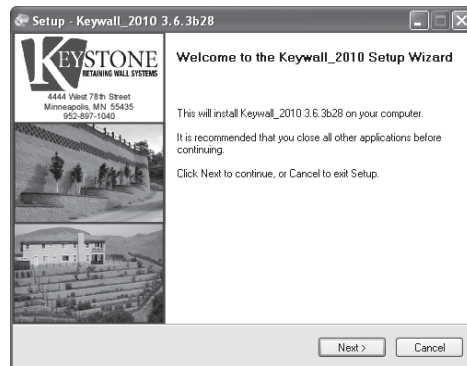


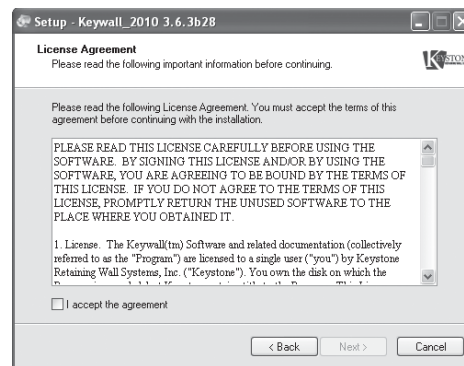
Figure 5:3 KeyWall For Windows Installing Software from Technical CD



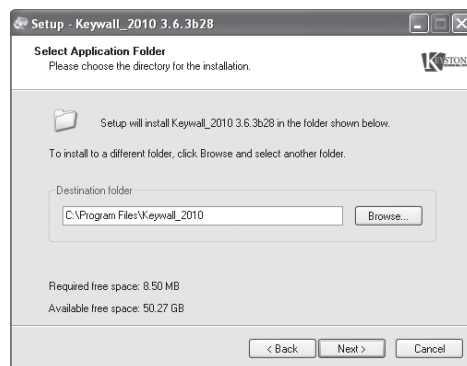
KEYWALL INSTALLATION & SET UP



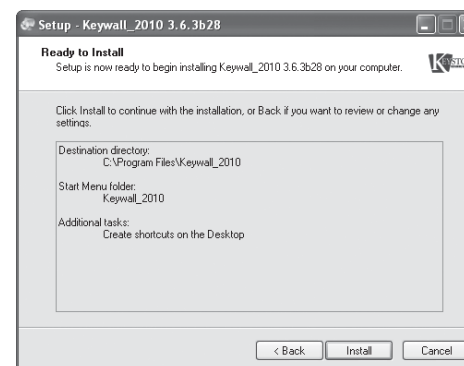
Step 1: Welcome Screen (press next).



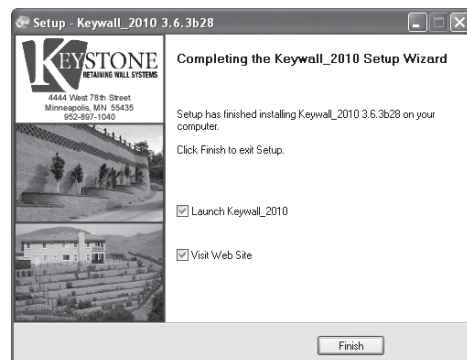
Step 2: License Agreement - Check "I accept the agreement" (press next).



Step 3: Destination Directory
(press next or browse if you wish to save in a different directory)



Step 4: Ready to Install (press install)



Step 5: Install Complete - Check "Launch KeyWall 2010" or "Visit Web Site" if desired (press finish).



LAUNCHING THE KEYWALL PROGRAM

The default KeyWall installation on the Windows operating system will place the KeyWall program and data files in a directory labeled “KeyWall 2010,” which resides in the Programs directory on the C: drive.

For easy access to the program, KeyWall will place a shortcut icon on the desktop that can be used to start the program. Additionally, KeyWall can be accessed from the Start Menu.



Figure 5:4 KeyWall for Windows Located on Desktop and Start Menu.

It is necessary to register KeyWall with Keystone Retaining Wall Systems to use the printing feature of the software. Registration is covered in a later section.



KEYWALL WINDOWS INTERFACE

The KeyWall interface follows standard Windows functions. KeyWall presents an operating window with pulldown menus and tabbed data input windows within the program window.

All pulldown menus, window tabs, and design cases can be accessed with the mouse in a conventional point-and-click manner or through “hot” key commands. Buttons and check box options can be set with a mouse-click. Data input fields can be individually selected with the mouse or the Tab key can be used as a method of quickly moving from field to field.

Design cases

This option permits the saving of multiple design sections under the same file name.

Add / Delete Cases

Add button creates a new subfile which copies all the settings from the design section that is being worked on until modified for the new section. Delete button deletes the selected case.

Move Up / Move Dn

Move Up and Move Dn buttons allow the cases to be rearranged by moving them up or down.

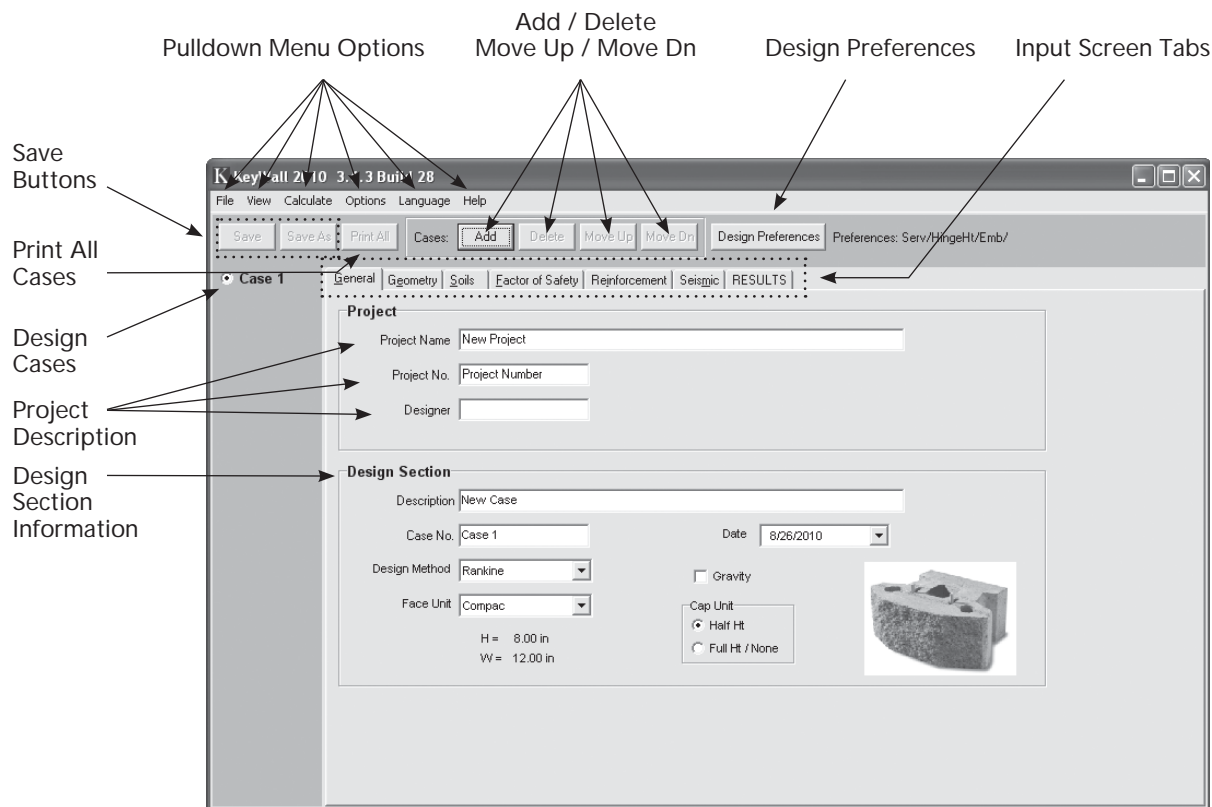


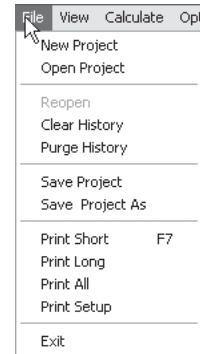
Figure 5:5 KeyWall for Windows Interface Options on the next page, includes comments



KEYWALL INTERFACE OPTIONS

File Menu

The File menu permits the creation of a new project and saving and opening of project files. Projects are saved with .kwp suffix for reference.

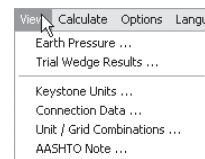


Print Setup and Print Short

Print Setup provides dialog boxes where different printers and printer features can be selected for use with the KeyWall program. Print Short prints the short printout for the selected case. Print Long provides a long-form printout that provides intermediate data and calculation steps used by the KeyWall program. Print All prints the short printout results for all of the cases. Exit shuts the program down.

View Menu

Earth Pressure permits quick access to the earth pressure and loads. Trial Wedge Results allows you to print the trial wedge iteration information. The print trial wedge box must be checked in the design preferences. Keystone Units will display the design characteristics of the unit selected. Unit/Grid Combinations display the possible block and grid combinations sorted by geogrid manufacturer or block. AASHTO Note provides a description how the active earth pressure coefficient, k_a , is handled for walls with batter less than and greater than or equal to 10° .



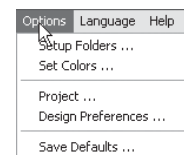
Calculate Menu

New Design will calculate trial results based on the input data provided.



Options

Setup Folders list two directories. The first is the data file folder where KeyWall accesses data files needed to complete the design. The second directory is the local work folder that shows where the KeyWall program is saved on the computer. Set Colors will permit changes to some of the display colors of the KeyWall program. Project will permit various KeyWall preference changes such as changing to metric units, printing the colors on the results, and removing disclaimers.



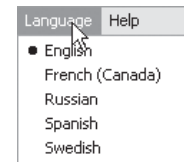
Design Preferences allows the setting of specific design parameters and methods of calculation for specific design elements. Save Defaults creates a default file that saves the KeyWall input settings and will reset the program data every time the program is restarted or a New Project is chosen.



KEYWALL INTERFACE OPTIONS

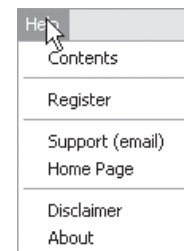
Language

A drop down menu allows you to choose the language of the program.



Help Menu

Contents provides access to a Help file which provides information about the KeyWall program. Register opens the registration window. Support opens a blank Microsoft Outlook e-mail message that can be used to e-mail Keystone questions. Home Page will direct you to the Keystone Web Site; www.keystonewalls.com.



Disclaimer is the Keystone disclaimer related to use of the KeyWall software. About provides a KeyWall information splash screen.

KEYWALL REGISTRATION DETAILS

KeyWall Registration

Version is registered, expiration is Mar 26, 2012

If you have a current registration, please insert it below:

Registration Code: 88CFD0C917C53B79

First Name: James

Last Name: Grams

Company: Keystone Retaining Wall Systems

Address: 4444 W. 78th Street

Address 2:

City: Minneapolis State: MN Postal Code: 55435 Country: USA

Telephone #: 952-897-1040 Facsimile: 952-897-3858

email: jgrams@keystonewalls.com

Ok email Reg. Print Cancel

Figure 5:6 KeyWall Registration

KeyWall Registration Steps

- 1 Enter Registration Information
Enter all registration information except the registration code.
- 2 E-mail Registration
Click on email Reg. An Outlook e-mail message will open with your registration information to be sent to Keystone. Click Send. The form can also be printed and sent via e-mail or fax.
- 3 Enter Registration Code
Keystone will reply with your registration code. Enter the registration code and press OK. A window stating Valid Code will open if the registration code was successful.



Note:

The Registration Code, First Name, Last Name and Company must be input exactly as shown on the registration information provided by Keystone Retaining Wall Systems. The letters are from A to F and the numbers are from 0 to 9. The spelling and capitalization has to be exactly as provided.



GENERAL INPUT

The Project name and number input fields accept project text information for the title block on the printouts. The Case input fields accept text data unique to each design case. The Date defaults to the present date unless manually changed.

Metric Units

KeyWall works in English units by default or Metric units when the metric option is selected under Options/Project.

Design Methodology

KeyWall permits analysis by eight different methods. The NCMA 2nd Edition method follows the 1997 Design Manual for Segmental Retaining Walls, Second Edition and is based on Coulomb active earth pressure analysis and failure planes.

The NCMA 3rd Edition method is based on the 2009 NCMA Design Manual for Segmental Retaining Walls. It is similar to the NCMA 2nd Edition with a few exceptions: hinge height and connection serviceability criteria have been eliminated, the designer has the option of including the vertical components in the external calculations, and NCMA 3rd Edition includes an additional design check called Internal Compound Stability(ICS). This option is not included in KeyWall 2010 and may be added at a later date (see description in KeyWall).

KeyWall 2010 3.6.3 Build 28

File View Calculate Options Language Help

Save Save As Print All Cases: Add Delete Move Up Move Dn Design Preferences Preferences: Serv/HingeHt/Emb/

Case 1 General Geometry Soils Factor of Safety Reinforcement Seismic RESULTS

Project

Project Name New Project

Project No. Project Number

Designer

Design Section

Description New Case

Case No. Case 1 Date 8/26/2010

Design Method Rankine

Face Unit Compac

H = 8.00 in

W = 12.00 in

☐ Gravity

Cap Unit

☒ Half Ht

☐ Full Ht / None

Figure 5:7 General Tab



GENERAL INPUT

The Rankine method is based on Rankine active earth pressure analysis and failure plane with an adjustment for the effects of wall batter. The 1996 AASHTO method is based on an earth pressure and failure plane analysis similar to the Rankine analysis. The AASHTO Simplified method is based on a Rankine earth pressure and failure plane analysis similar to the 1996 AASHTO analysis but treats backslope loadings in a different manner per the 2002 AASHTO specifications. Minimum grid length of 8' (2.4m), base/height ratio (0.7H) and other AASHTO requirements are included.

The Australian Method is based on the Standards Australia, AS4678-2002-Earth Retaining Structures. The method is explained in the joint publication "Segmental Concrete Reinforced Soil Retaining Walls - Design and Construction Guide", Concrete Masonry Association of Australia and Standards Australia, 2002.

The AASHTO-LRFD method is similar to the AASHTO Simplified method except with the application of load and resistance factors. It is based on the 2010 AASHTO Specifications.

The CAN-LRFD method is based on the Canadian Highway Bridge Design Code (CAN/CSA-56-00). The CAN-LRFD method follows the AASHTO-LRFD method except with Canadian load and resistance factors. The CAN-LRFD method also includes an overturning resistance factor.

Face Unit Type

Select the desired Keystone unit type. The typical units included are Keystone Compac, Compac II, Compac III, Country Manor, Keystone Century Wall, Standard 18", Standard 20", and Standard 21". Data files for other Keystone units are available. Selecting Gravity forces a gravity wall analysis regardless of parameters.

Cap Unit

A half block height cap or full block height cap/none may be selected.

GEOMETRY INPUT

The geometry input screen allows you to customize the project dimension information.

Wall Height

Enter the total wall height (feet or meters) as measured from top of leveling pad to finished grade at the top of wall.

Embedment

Enter the distance below grade that the top of leveling pad is located. This value is only used in the calculation of foundation bearing capacity based on level ground in front of the wall.

Face Batter

Enter the appropriate wall batter. Pressing the arrows will toggle through the typical Keystone unit batters (0°, 4°, 8°). Other batters are possible in straight walls by tilting the leveling pad slightly which can be entered directly.

Note:

AASHTO designs are specific to the owner and project requirements which can include special design considerations not typically included in the KeyWall options.



GEOMETRY INPUT

Backfill Slope

Enter the slope angle of backslope in the design section as applicable.

Horizontal Offset

Enter the horizontal offset as measured from the back of wall to the break in slope or the horizontal offset of live and dead load for a level surcharge design case.

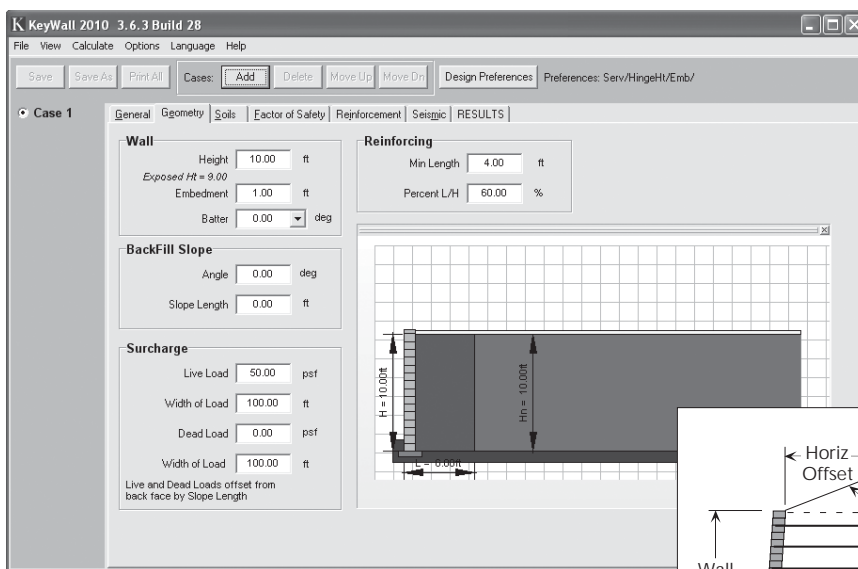


Figure 5:8 Geometry Tab

Surcharge

Enter the live or dead load surcharge (psf or kN/m²) as applicable.

Live and dead loads are only applied to the horizontal surface above the wall, not the sloping surface as shown.

Load Width

Enter width of the live or dead load when modeling a strip live load. This value should be left at 100ft or 30m for an infinite loading condition unless performing a special analysis.

Minimum Length

Enter a minimum geogrid length that may be required. The program defaults to 4' (1.2m) for NCMA / Rankine and 8' (2.44m) for AASHTO. KeyWall will use the minimum reinforcement length specified regardless if a shorter length will satisfy the design criteria.

Minimum L/H

Enter a minimum base to height percentage that may be required. The program defaults to minimums of 60% for NCMA and Rankine, and 70% for AASHTO.



SOIL PROPERTIES

Soil design properties must be established to include the phi angle (ϕ), and in-situ unit weight (γ), (pcf or kN/m³), for each soil zone material. The phi angle typically reflects the peak effective shear strength of the compacted or in-situ soil. The unit weight typically reflects the moist unit weight of the soil. These values may be estimated for small walls, but should be verified by testing or experience for larger structures. $\phi = 28^\circ$ and $\gamma = 120$ pcf (18.85kN/m³) is a good starting point if no other information exists but may be inadequate for steep back slopes. Cohesion (psf or kN/m²) is typically set to zero due to the difficulties of predicting long term soil strength properties of cohesive soils in the analysis. Cohesion can lead to artificially low earth pressures in small walls that could mathematically justify 10' (3m) gravity walls or no reinforcement required in the upper 5' (1.5m) of a wall which is not correct for a long-term design condition.

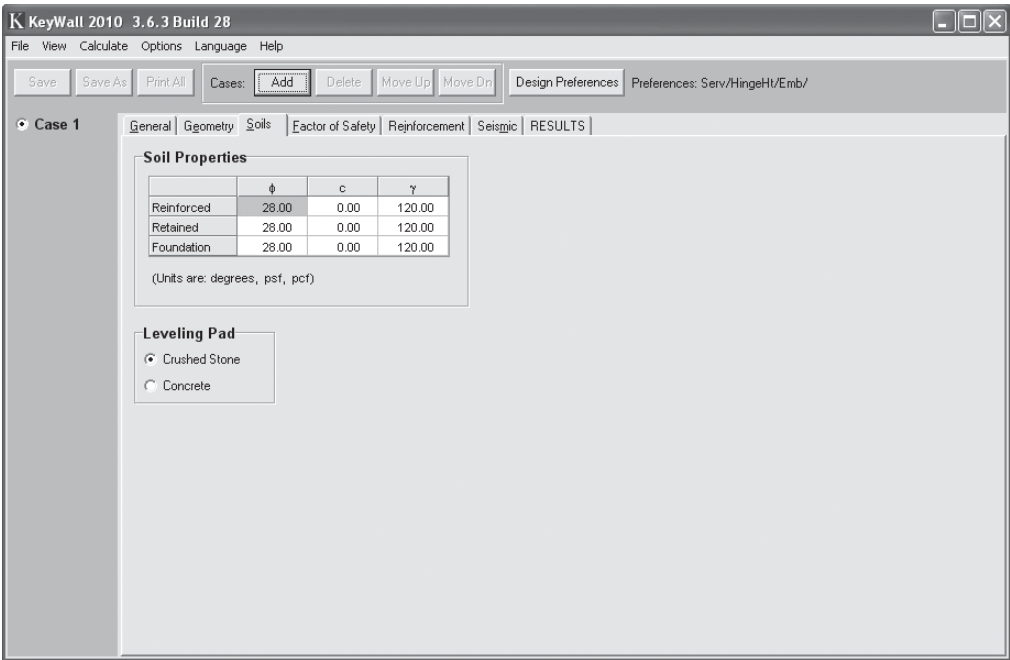


Figure 5:9 Soils Tab

LEVELING PAD

A leveling pad material must be chosen for gravity walls in order to determine the base friction for the specific unit type based on laboratory test data. This selection only affects the sliding resistance calculation of gravity walls.



FACTORS OF SAFETY

Default Values

Review default values and adjust as required for project specific requirements. Generally, a factor of safety of 1.5 is the minimum required on all essential elements of the wall design.

External Stability

The Factor of Safety in Overturning is usually set at 2.0 for reinforced soil structures even though this element of the design analysis rarely controls the reinforcement length. The Factor of Safety in sliding is usually is set at 1.5 for gravity and reinforced structures. The Factor of Safety in Bearing is usually set at 2.0 for reinforced soil structures per NCMA criteria. AASHTO requires a minimum factor of 2.5 whereas conventional concrete footings will use a factor of 3.0.

Uncertainties

The Factor of Safety for Uncertainties can change for some government and highway work. Sometimes, specific design factors or maximum allowable geogrid loads are dictated in construction documents and specifications that require the geogrid long term design strength to be manually adjusted (see Design Preferences).

Internal Connection Serviceability

The connection Serviceability (0.75" or 19mm deformation criteria on connection strength) is only active if a safety factor value of 1.0 or greater is input or enabled in the Design Preferences. When 0.0 is input (default for the Rankine method), the serviceability criteria is neglected and only the peak connection strength safety factor criteria governs the design. Sometimes, specific connection design factors are dictated in plans and specifications, which require that these factors be adjusted.

Note:

Connection serviceability is not part of current NCMA or AASHTO criteria but the option remains in KeyWall.

Figure 5:10 Factor of Safety Tab



LRFD FACTORS

Strength I

Strength I is the load combination for normal loading.

Extreme Event I

Extreme Event I is the load combination for earthquake loading.

Service I

Service I sets all load and resistance factors equal to 1.0. The Analyze Service I box must be checked for KeyWall to calculate the serviceability load case.

Max/Min

MAX load factors are applied to the driving forces and MIN load factor are applied to the resisting forces.

Load Factors

| | | |
|------------|---|--|
| DC | = | dead load of structural components |
| EH | = | horizontal earth pressure load |
| EV | = | vertical pressure from dead load of earth fill |
| ES | = | earth surcharge load |
| LL, PL, LS | = | vehicular live load, pedestrian live load, live load surcharge (LL is typically used to represent all live loads) |

The screenshot shows the KeyWall 2010 software interface. The title bar reads 'KeyWall 2010 3.6.3 Build 28'. The menu bar includes 'File', 'View', 'Calculate', 'Options', 'Language', and 'Help'. The toolbar contains buttons for 'Save', 'Save As', 'Print All', 'Cases', 'Add', 'Delete', 'Move Up', 'Move Down', 'Design Preferences', and 'Preferences: Serv/Hinge/Emb/'. The 'Cases' tab is selected, and the 'LRFD Factors' sub-tab is active. The 'Case 1' section is expanded, showing the 'AASHTO LRFD Load Factors for Load Combination Limit States' dialog box. This dialog box contains two tables: 'Load Factors' and 'Resistance Factors'. The 'Load Factors' table has columns for 'Case', 'Strength I (MAX, MIN)', 'Extreme Event I (MAX, MIN)', and 'Service I'. The 'Resistance Factors' table has columns for 'Strength I', 'Extreme I', and 'Service I'. A 'Reset to Defaults' button is located to the right of the 'Load Factors' table. A checkbox labeled 'Analyze Service I' is also present.

| Case | Strength I | | Extreme Event I | | Service I |
|--------------------------|------------|------|-----------------|------|-----------|
| | MAX | MIN | MAX | MIN | |
| Dead Load (DC) | 1.25 | 0.90 | 1.25 | 0.90 | 1.00 |
| Dead Load Driving (EH) | 1.50 | 0.90 | 1.50 | 0.90 | 1.00 |
| Vertical Earth Load (EV) | 1.35 | 1.00 | 1.35 | 1.00 | 1.00 |
| Dead Load Surcharge (ES) | 1.50 | 0.75 | 1.50 | 0.75 | 1.00 |
| Live Load (LL, PL, LS) | 1.75 | 0.00 | 0.50 | 0.00 | 1.00 |

Per AASHTO 11.10.6.2.1, Load Factor EV is used for both Live & Dead loads in Internal Tension calculation

| | Strength I | Extreme I | Service I |
|-------------------------------|------------|-----------|-----------|
| Bearing Resistance | 0.65 | 0.65 | 1.00 |
| Sliding Soil to Soil | 1.00 | 1.00 | 1.00 |
| Sliding Soil to Reinforcement | 1.00 | 1.00 | 1.00 |
| Tensile Resistance | 0.90 | 1.20 | 1.00 |
| Pullout Resistance | 0.90 | 1.20 | 1.00 |

Figure 5:11 Load and Resistance Factors



REINFORCEMENT SELECTION

Manufacturer

Click on arrow to view a pull-down menu that lists the reinforcement manufacturers contained in the KeyWall data files. Select the desired manufacturer.

Primary, Secondary & Third

Click on the arrow to view a pull-down menu that lists the soil reinforcement types in the data files for the selected manufacturer. KeyWall allows up to three geogrid types in the design. “Primary” is the stronger geogrid that KeyWall will use at the bottom of the wall and “Secondary” is the geogrid that KeyWall will use higher in the wall if two different grid types are selected. The same convention is true if three geogrid types are selected. Note that KeyWall may only use the secondary and third geogrid type if it is adequate for the design.

Interaction Coefficients

The default geogrid pullout and sliding coefficients, C_i and C_{ds} , are determined by the ϕ angle value of the reinforced zone material input in the Soils input screen. These values can be manually changed if required (see Design Preferences to change). The scale effect correction factor, α , is applied to the pullout strength calculation. α is unique to the AASHTO LRFD method and the 2010 AASHTO Specification sets α at 0.80 for geogrid if α has not been determined through testing.

KeyWall 2010 3.6.3 Build 28

File View Calculate Options Language Help

Save Save As Print All Cases: Add Delete Move Up Move Down Design Preferences Preferences: Serv/Hinge/HV/Emb/

Case 1 General Geometry Soils LRFD Factors Reinforcement Seismic RESULTS

Reinforcement Parameters

Manufacturer: Mirafi XT

Primary: 3XT

Secondary: 3XT

Third: 3XT

Reinforced Fill Type

☒ Sand, Silt or Clay "S"

☐ 0.75" minus Crushed Stone or Gravel "G"

☐ 2.5" minus Crushed Stone or Gravel "C"

Interaction Coefficients

Pullout: 0.80 α : 0.80

Direct Sliding: 0.80

Per AASHTO, Rf_d and Rf_{id} must not be less than 1.10
(All forces are expressed in ppi)

| | Tult | RFcr | RFd | RFid "S" | RFid "G" | RFid "C" | LTDS | RF _{Te} | Tal |
|------|------|------|------|----------|----------|----------|------|------------------|------|
| 2XT | 2000 | 1.58 | 1.10 | 1.10 | 1.10 | 1.50 | 1046 | 0.90 | 942 |
| 3XT | 3500 | 1.58 | 1.10 | 1.10 | 1.10 | 1.25 | 1831 | 0.90 | 1648 |
| 5XT | 4700 | 1.58 | 1.10 | 1.10 | 1.10 | 1.25 | 2458 | 0.90 | 2213 |
| 7XT | 5900 | 1.58 | 1.10 | 1.10 | 1.10 | 1.25 | 3086 | 0.90 | 2777 |
| 8XT | 7400 | 1.58 | 1.10 | 1.10 | 1.10 | 1.25 | 3871 | 0.90 | 3484 |
| 10XT | 9500 | 1.58 | 1.10 | 1.10 | 1.10 | 1.25 | 4969 | 0.90 | 4472 |

Figure 5:12 Reinforcement Selection Window



REINFORCEMENT SELECTION

Reinforcement Fill Type

Select the appropriate fill type that comprises the reinforced zone. This selection determines the installation damage value that is recommended by the geogrid manufacturer in the determination of the geogrid long term design strength. The three material types generally reflect finer grained soils, $\frac{3}{4}$ " (19mm) minus material, and 2" (50mm) minus material. Check with the geogrid manufacturer for appropriate damage values when larger backfill material is anticipated (See Design Preferences to change).

Data Table

The table displays the long term design strength calculation for the geogrid materials by the selected manufacturer (See Design Preferences to change). Note that each column can be widened with the mouse cursor. Not all reinforcement has been tested with all Keystone units. The table and choices will only present tested combinations for the chosen unit type.

DESIGN PREFERENCES

Preferences

Design Preferences can be found under the Options pull down menu or by clicking the Design Preferences button located above the results tab.

Embedment

Select to include wall embedment in the bearing capacity calculation. (Default)

Serviceability

Select to include or not include connection serviceability in NCMA and AASHTO options as the default setting. Rankine method does not include serviceability by default.

Hinge Height

Select to include "hinge height" limitation in connection strength evaluation. (Default)

Base Friction

Select to not include base friction reduction in bottom reinforcement tension. (Default)

Change Ci/Cds

Select to permit changes to the default factors in Reinforcement Input Screen.

Durability at Connection

Select to include the durability reduction factor on the connection if required. Changes can be made to the default factor of 1.1 on the Reinforcement Tab.

Connection Creep

Select to include the long term creep reduction factor on the connection if required. Changes can be made to the default factor of 2.0 on the Reinforcement Tab.



DESIGN PREFERENCES

Seismic Connection Reduction

Select to include the seismic frictional connection reduction factor, equal to 0.80, in accordance with AASHTO LRFD and simplified design methods.

LL and ES for LRFD Internal Tension Calculation

The 2010 AASHTO Specifications apply the load factor ES to the live load and dead load for the internal tension calculation. Select this option if you choose to use the LL load factor on the live load for the internal tension calculation instead.

Vertical Design

Select to include the vertical components in the external stability calculation when designing with the NCMA 3rd Edition method.

Minimum Defaults

Select to override manufacturer's minimum values with greater reduction factors as required. KeyWall will use the greater value of the two when option is employed.

Unit Height

Select the default height of unit for English and Metric systems (ie: 203mm vs 200mm).

Professional Mode

Select professional mode to disable design limitations intended to discourage misuse of the KeyWall program.

Print Trial Wedge

Select to allow the trial wedge results to be printed. The trial wedge results can be printed from the view menu.

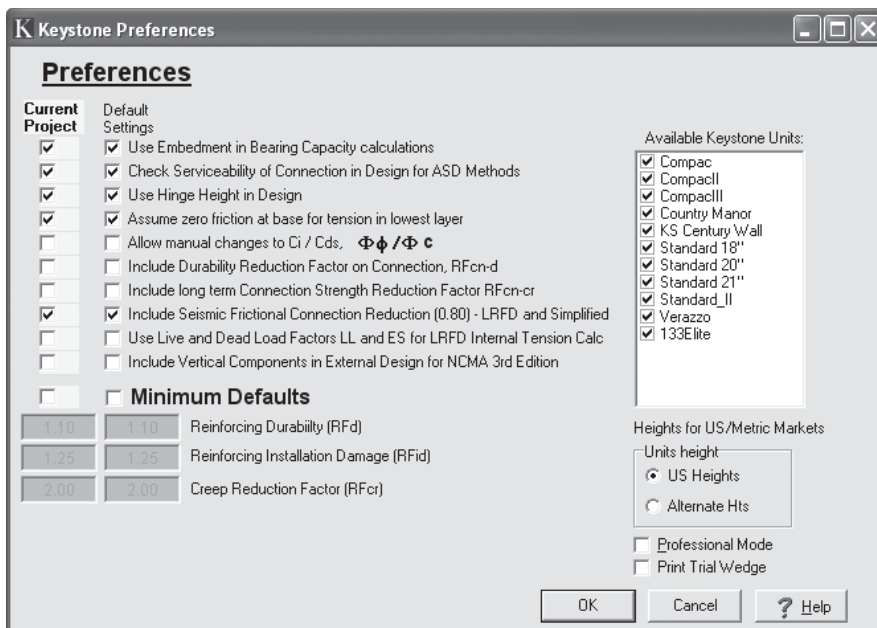


Figure 5:13 Design Preferences



RESULTS SCREEN

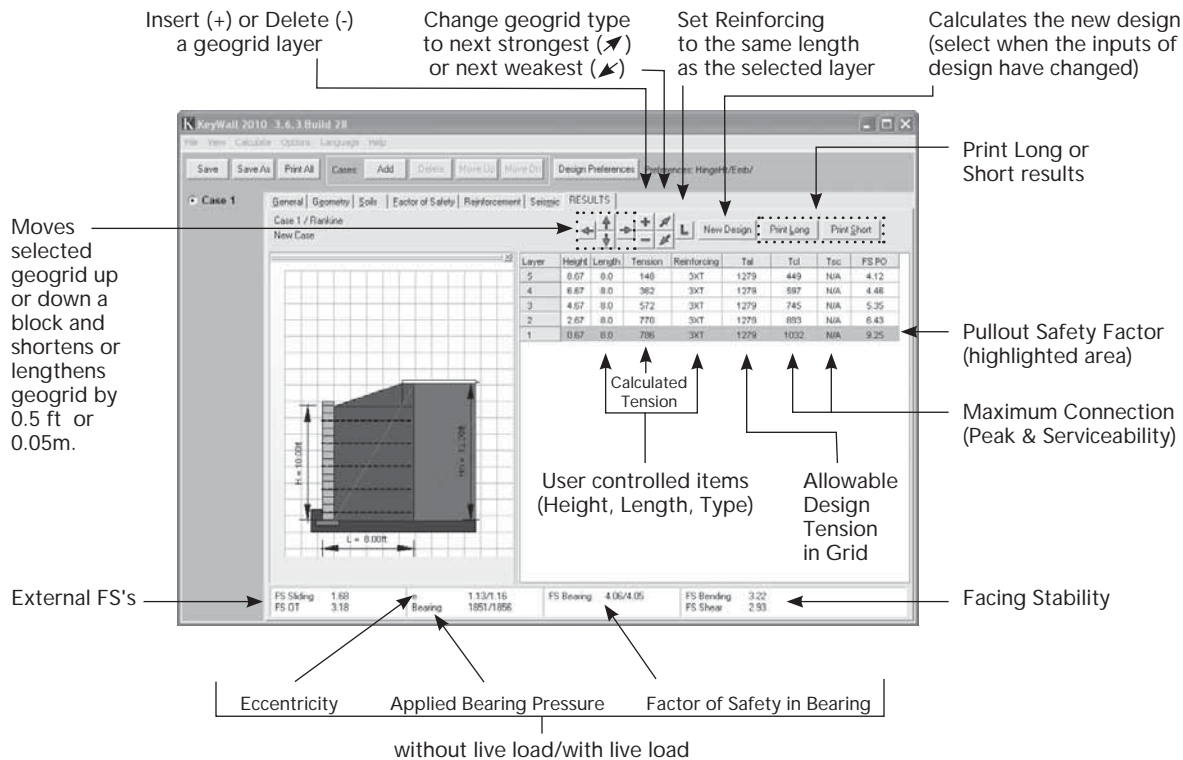


Figure 5:14 Soil Geogrid Result screen Input Data

Note:
Results will recalculate automatically after every audit. See Help file for explanation of flags and notations on display.

EDIT GEOGRID DATA IN RESULTS SCREEN

Prior to editing, select a level of geogrid in the results screen first to begin any changes.

| | |
|-----------------------|---|
| <Tab> | Move forward through the geogrid height, length and type fields |
| <Shift-Tab> | Move backward through the geogrid height, length, and type fields |
| <Arrow Up> | Move up one geogrid level |
| <Arrow Down> | Move down one geogrid level |
| <Control-Arrow Up> | Move selected grid layer up one unit |
| <Control-Arrow Down> | Move selected grid layer down one unit |
| <Control-Arrow Left> | Shorten grid length by 0.5' or 0.05m |
| <Control-Arrow Right> | Lengthen grid length by 0.5' 0.05m |
| <Space Bar> | Change geogrid type at a specific layer |
| <Control-Insert> | Insert additional reinforcement layer above selected layer |
| <Control-Delete> | Delete selected reinforcement layer |
| <Control-L> | Set all reinforcing to the same length as selected grid layer |



SEISMIC INPUT

A

Input peak ground acceleration for project location. This may come from site specific recommendation, local codes, or seismic maps such as those contained in AASHTO. Site class and site factors may have to be considered when determining the appropriate A value.

A_m

A_m , peak structure acceleration, is calculated from A in accordance with the appropriate design methodology chosen.

K_h

K_h , horizontal seismic coefficient, is calculated from A and A_m in accordance with the appropriate design methodology chosen.

K_v

K_v , vertical seismic coefficient, is input when required but generally left at $K_v = 0$.

Enable Seismic

Check this box to enable the pseudo-static seismic calculation and press Results. The Results will now display two values in each field representing the static and static+dynamic values for comparison (534/845). KeyWall will only trial design the static case so the user must manually adjust the design to satisfy both the static + dynamic case.

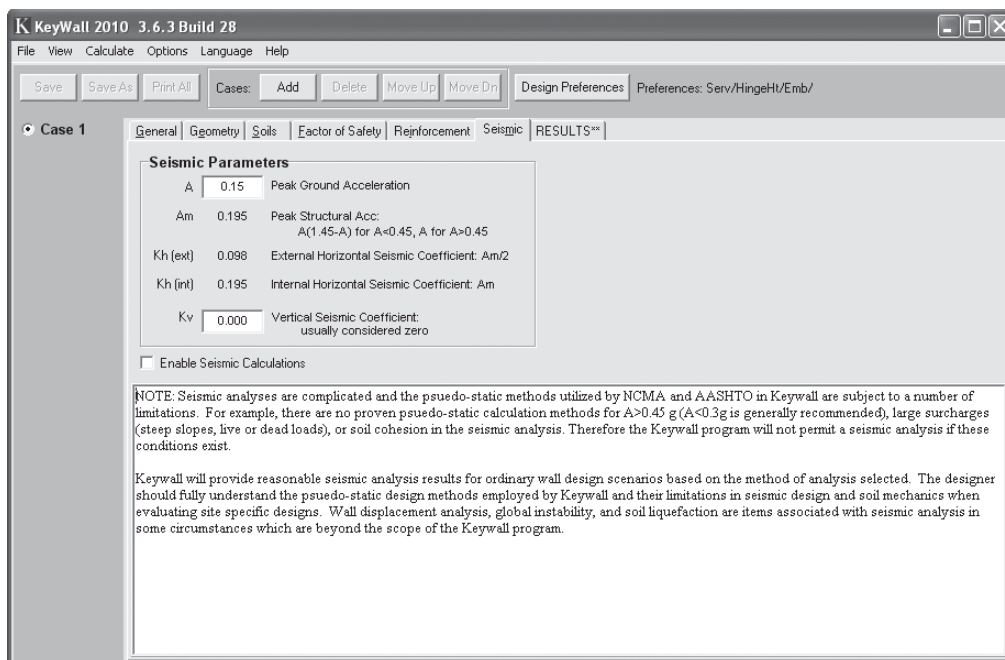


Figure 5:15 Dynamic Results Screen



SEISMIC INPUT

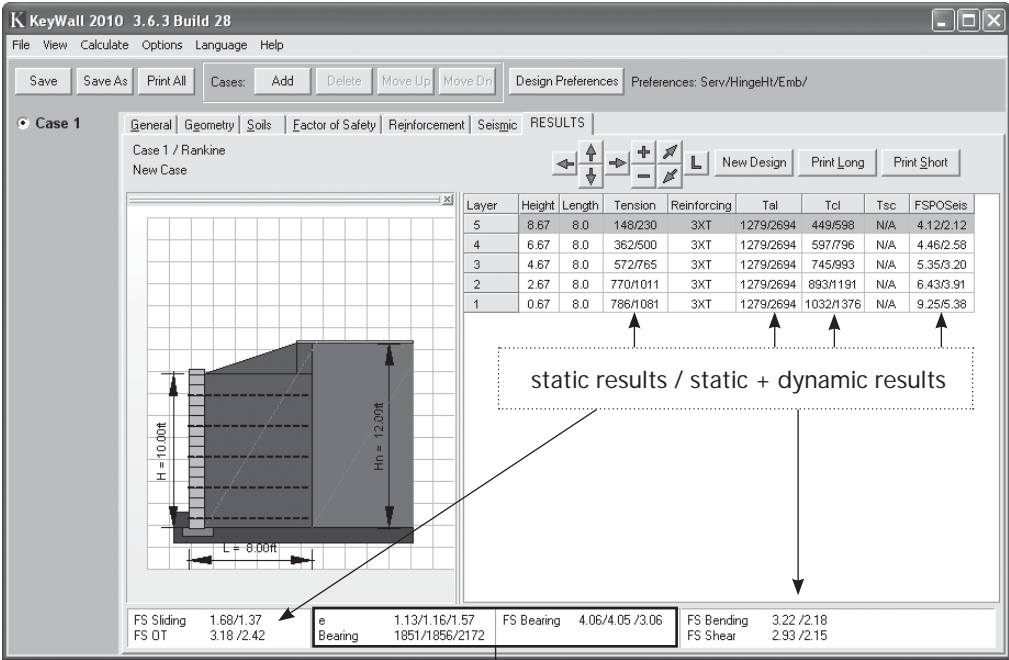


Figure 5:16 Seismic Input Screen

static without live load /
static with live load /
static + dynamic results

PART SIX



APPENDIX A

Turtle Mountain, Vernon, British Columbia; Keystone Century Wall®





INFINITE SLOPE SURCHARGE NCMA 2ND EDITION - COULOMB METHODOLOGY

This set of calculations is intended to verify the KeyWall program output of a typical gravity wall design section. The design follows the procedure outlined in the 1997 NCMA Design Manual for Segmental Retaining Walls. The pertinent design information is summarized below:

1) General Design Data

Compac Keystone Units (120 pcf with unit drainage fill and $W_u = 1.0'$)
 Crushed Stone Leveling Pad ($\phi = 40^\circ$ and $\gamma = 130\text{pcf}$)
 Wall Batter (ι) = 8 degrees (1"-1.25" per 8" unit)
 Design Height = 3.0' (2.5' exposed + 0.5' embedment)
 Backslope, (β) = 1V:4H (14.0 degrees)
 Length of Backslope = 50' (infinite for design purposes)
 Surcharge = 0 psf (not applied to infinite slope design)
 Gravity wall design ($FS_{ot} > 1.5$)

2) Soil Parameters (degrees, psf, pcf)

| SOIL PARAMETERS | ϕ | c | γ |
|-----------------|--------|---|----------|
| Retained Soil | 30 | 0 | 120 |
| Foundation Soil | 30 | 0 | 120 |

3) Geometric Parameters

ϕ = 30 degrees
 δ = $\frac{2}{3}\phi = 20$ degrees (concrete to soil)
 α = 98 degrees ($90^\circ + 8^\circ$)
 β = 14 degrees (4H:1V slope)

4) Coulomb Earth Pressure Calculation

Equation (3c)

$$k_a = \frac{\sin^2 (98 + 30)}{\sin^2 98 \sin (98 - 20) \left[1 + \sqrt{\frac{\sin(30 + 20) \sin(30 - 14)}{\sin(98 - 20) \sin(98 + 14)}} \right]^2}$$



INFINITE SLOPE SURCHARGE NCMA 2ND EDITION - COULOMB METHODOLOGY

Equation (3c) $k_a = 0.295$ for above parameters - Coulomb

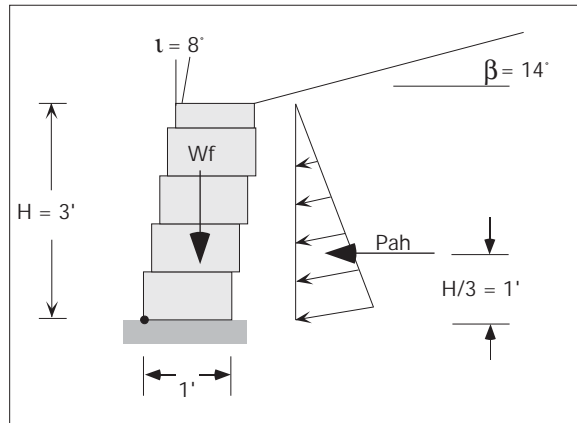
Equation (3a) $P_a = \frac{1}{2} \gamma H^2 k_a$

$$P_{ah} = \frac{1}{2} \gamma H^2 k_a \cos(\delta - \alpha)$$

$$P_{ah} = (0.5) (120 \text{ pcf}) (3')^2 (0.295) \cos(20 - 8)$$

$$P_{ah} = \mathbf{156 \text{ lbs/lf}}$$

External Stability Diagram



5) Overturning

Overturning Moment

$$\begin{aligned} M_o &= P_{ah} (H/3) + P_{qh} (H/2) \\ &= 156 \text{ lbs} (3'/3) + 0 \\ &= \mathbf{156 \text{ ft-lbs}} \end{aligned}$$

Resisting Moment

$$\begin{aligned} M_r &= W_f \times X = (H W_u \gamma) (W_u/2 + (H/2) \tan(\alpha)) \\ &= (3' \times 1' \times 120 \text{ pcf}) (1'/2 + (3'/2) \tan(8)) \\ &= \mathbf{256 \text{ ft-lbs}} \end{aligned}$$

$$FS_{ot} = M_r / M_o = 256/156 = \mathbf{1.64} > 1.5 \text{ OK}$$



INFINITE SLOPE SURCHARGE NCMA 2ND EDITION - COULOMB METHODOLOGY

6) Sliding

Sliding Resistance (*at interface or through pad*)

$$F_v = 0.92R_v = 0.92(3' \times 120\text{pcf}) = 331 \text{ lbs/ft (from shear curves)}$$

$$F_v = R_v \tan 40^\circ = (3' \times 1' \times 120\text{pcf}) (\tan 40^\circ) = 302 \text{ lbs/ft (through pad)}$$

$$= \mathbf{302 \text{ lbs/ft (lower of two)}}$$

$$FS_{sl} = F_v / P_{ah} = 302/156 = \mathbf{1.94} > 1.5 \text{ OK}$$

7) Bearing Pressure (*under crushed stone leveling pad*)

Equation (3i) $e = B/2 - (M_r - M_o)/R_v$

$$= 1' / 2 - (256 - 156) / (3' \times 1' \times 120\text{pcf})$$

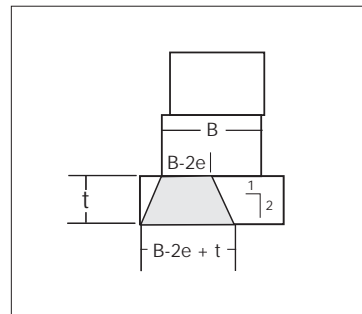
$$= \mathbf{0.222'}$$

Applied Bearing Pressure (*under 6" pad*)

Equation (3j) $\sigma_v = R_v / ((B-2e) + 0.5') + 0.5'\gamma$

(modified) $= (3 \times 120\text{pcf}) / ((1' - 2 \times 0.222') + 0.5') + 0.5'(130\text{pcf})$

$$= \mathbf{406 \text{ lbs/sf}}$$



8) Bearing Capacity (*under crushed stone leveling pad*)

Equation (3k) $Q_{ult} = cN_c + \gamma DN_q + 0.5\gamma(B-2e)N_\gamma$

where:

$$N_c = 30.14, N_q = 18.4, N_\gamma = 22.40$$

$$B = (B-2e) + 0.5 = (1' - 2 \times 0.222') + .5 = 1.06'$$

$$D = .5 + .5 = 1.0' \text{ (to bottom of pad)}$$

$$c = 0$$

$$Q_{ult} = 0 + (120)(1)(18.4) + (0.5)(120)(1.06)(22.40)$$

$$= \mathbf{3633 \text{ psf}}$$

$$FS_{br} = 3633/406 = \mathbf{8.95} > 2.0 \text{ OK}$$

Note:

Factor of safety in bearing without 6" crushed stone base is only 3.3.



INFINITE SLOPE SURCHARGE NCMA 2ND EDITION - COULOMB METHODOLOGY

9) Inter-Unit Shear

The inter-unit shear of Compac units is over 700 lbs/lf plus overburden pressure friction. The maximum driving force is 156 lbs/lf, so by inspection, inter-unit shear is more than adequate with the Keystone Compac units.

10) General Comments

Gravity wall design is very sensitive to wall batter, backslope, surcharge, assumed soil properties, and foundation stability. Small variations can result in unacceptable safety factors and potential wall movement. Inadequate surface drainage can permit saturation of the retained soils and foundation soils, which can also cause wall instability and movement.



INFINITE SLOPE SURCHARGE NCMA 2ND EDITION - COULOMB METHODOLOGY



RETAINING WALL DESIGN

KeyWall_2010 Version 3.7.1 Build 1

Project: Part 6; Design Examples

Project No: NA

Case: Appendix A

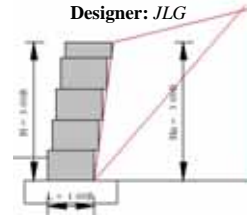
Design Method: NCMA 2nd Edition (parallelogram soil interface)

Date: 8/31/2010

Designer: JLG

Design Parameters

| Soil Parameters: | ϕ | c psf | γ pcf |
|------------------|-----------------------------|---------|--------------|
| Retained Zone | 30 | 0 | 120 |
| Foundation Soil | 30 | 0 | 120 |
| Unit Fill: | Crushed Stone, 1 inch minus | | |



Minimum Design Factors of Safety

| | | | | | |
|--------------|------|----------|------|-----------------|------|
| sliding: | 1.50 | pullout: | 1.50 | uncertainties: | 1.50 |
| overturning: | 1.50 | shear: | 1.50 | connection: | 1.50 |
| bearing: | 2.00 | bending: | 1.50 | Serviceability: | 1.00 |

Analysis:

Case: Appendix A

NCMA 2nd Edition - Infinite Slope

| | | | |
|---------------|-----------------------------|-----------------------------|-----------|
| Unit Type: | Compac / 120.00 pcf | Wall Batter: | 8.00 deg. |
| Leveling Pad: | Crushed Stone | embedment: | 0.50 ft |
| Wall Ht: | 3.00 ft | 50.00 ft long | |
| Back Slope: | 14.00 deg. slope, | DL: 0 psf uniform surcharge | |
| Surcharge: | LL: 0 psf uniform surcharge | Load Width: 100.00 ft | |
| | Load Width: 100.00 ft | | |

Results:

| | <u>Sliding</u> | <u>Overturning</u> | <u>Bearing</u> | <u>Shear</u> | <u>Bending</u> |
|------------------------------|----------------|--------------------|----------------|--------------|----------------|
| Factors of Safety: | 1.94 | 1.64 | 8.95 | N/A | N/A |
| Calculated Bearing Pressure: | 405 / 405 psf | | | | |
| Eccentricity at base: | 0.22 ft | | | | |

NOTE: THESE CALCULATIONS ARE FOR PRELIMINARY DESIGN ONLY AND SHOULD NOT BE USED FOR CONSTRUCTION WITHOUT REVIEW BY A QUALIFIED ENGINEER

PART SIX



APPENDIX B

Lakeside Development, Chanhassen, Minnesota; Keystone 133Elite®





NCMA 2ND EDITION - COULOMB METHODOLOGY LEVEL SURCHARGE - 250 PSF

This set of calculations is intended to verify the KeyWall program output of a typical reinforced soil wall design section. The design follows the procedure outlined in the 1997 NCMA Design Manual for Segmental Retaining Walls.

The pertinent design information is summarized below:

1) General Design Data

Standard 21 Keystone Units (120 pcf with drainage fill and $W_u = 1.75'$)
 Stratagrid SG200 Polyester Geogrid
 Wall Batter (ι) = 0°, near-vertical orientation
 Design Height = 10' (9' exposed + 1' embedment)
 Base Length, B = 9.0' (uniform lengths chosen for simplicity)
 Backslope, β = 0, level backslope
 Surcharge = 250 psf (typical roadway surcharge)

2) Soil Parameters (*Degrees, psf, pcf*)

| SOIL PARAMETERS | ϕ | C | γ |
|-----------------|--------|---|----------|
| Reinforced Soil | 34 | 0 | 120 |
| Retained Soil | 30 | 0 | 120 |
| Foundation Soil | 30 | 0 | 120 |

3) Geogrid Design Parameters (*plf*)

| Geogrid | T_{ult} | RF_{cr} | RF_d | RF_{id} | LTDS | FS | T_{al} |
|--------------|-----------|-----------|--------|-----------|------|-----|----------|
| Strata SG200 | 3600 | 1.55 | 1.10 | 1.10 | 1919 | 1.5 | 1280plf |

(C_i & C_d = 0.90 for select backfill)

4) Geometric Parameters - Coulomb

Internal

ϕ = 34 degrees
 δ = $\frac{2}{3} \phi$ = 22.67 degrees (concrete/soil)
 α = 90 degrees (90° + no batter)
 β = 0 degrees (level)

External

ϕ = 30 degrees
 δ = ϕ = 30 degrees (soil to soil)
 α = 90 degrees (90° + no batter)
 β = 0 degrees (level)



NCMA 2ND EDITION - COULOMB METHODOLOGY LEVEL SURCHARGE - 250 PSF

5) Coulomb Earth Pressure Calculation

Internal

$$K_a = \frac{\sin^2(90 + 34)}{\sin^2 90 \sin(90 - 22.67) \left[1 + \sqrt{\frac{\sin(34 + 22.67) \sin(34 - 0)}{\sin(90 - 22.67) \sin(90 + 0)}} \right]^2}$$

Equation (3c) $k_a = 0.254$ for above parameters - Coulomb

Equation (3d) $\rho = 58.4^\circ$ for above parameters - Coulomb

External

$$K_a = \frac{\sin^2(90 + 30)}{\sin^2 90 \sin(90 - 30) \left[1 + \sqrt{\frac{\sin(30 + 30) \sin(30 - 0)}{\sin(90 - 30) \sin(90 + 0)}} \right]^2}$$

Equation (3c) $k_a = 0.297$ for above parameters - Coulomb

External Forces

Equation (3a)

$$P_a = \frac{1}{2} \gamma H^2 k_a$$

$$P_{ah} = \frac{1}{2} \gamma H^2 k_a \cos(\delta - \iota) \text{ (Horz.)}$$

$$P_{ah} = (0.5) (120 \text{ pcf}) (10')^2 (0.297) \cos(30 - 0)$$

$$P_{ah} = 1543 \text{ lbs/lf}$$

External Masses

$$W_f = W_u H \gamma = (1.75')(10')(120 \text{ pcf}) = 2100 \text{ lbs/lf}$$

$$W_1 = (B - W_u) H \gamma = (9.0' - 1.75')(10')(120 \text{ pcf}) = 8700 \text{ lbs/lf}$$

$$W_q = q(B - W_u) = (250 \text{ psf})(9.0' - 1.75') = 1813 \text{ lbs/lf}$$

Equation (3b)

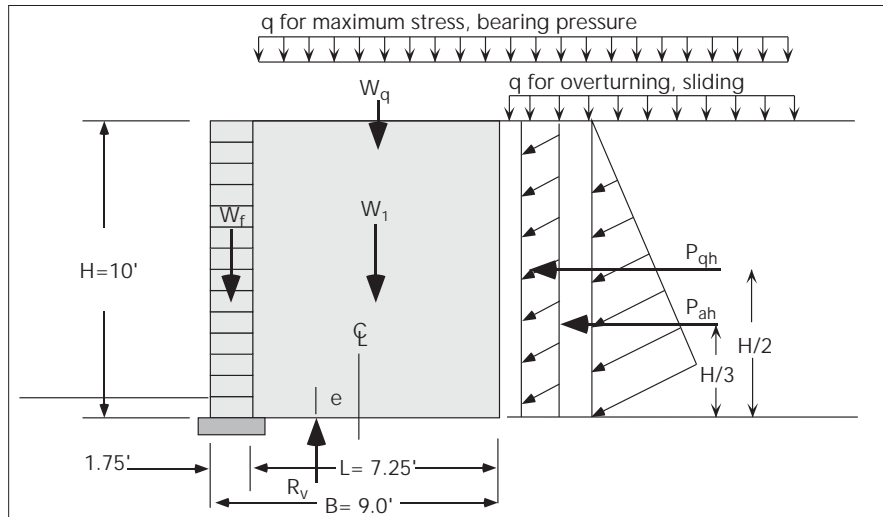
$$P_q = qH k_a$$

$$P_{qh} = qH k_a \cos(\delta - \iota) \text{ (Horizontal Component)}$$

$$P_{qh} = (250 \text{ psf})(10')(0.297) \cos(30 - 0)$$

$$P_{qh} = 643 \text{ lbs/lf}$$

External Stability Diagram





NCMA 2ND EDITION - COULOMB METHODOLOGY LEVEL SURCHARGE - 250 PSF

6) Overturning

Overturning Moment

$$\begin{aligned} M_O &= P_{ah} (H/3) + P_{qh} (H/2) \\ &= 1543 \text{ lbs} (10' / 3) + 643 \text{ lbs} (10' / 2) \\ &= \mathbf{8358 \text{ ft-lbs}} \end{aligned}$$

Resisting Moment

$$\begin{aligned} M_r &= W_f \times W_u / 2 + W_1 \times (W_u + L / 2) \\ &= (2100 \times 1.75' / 2) + 8700(1.75' + 7.25' / 2) \\ &= \mathbf{48600 \text{ ft-lbs}} \end{aligned}$$

$$FS_{ot} = M_r / M_O = 48600 / 8358 = \mathbf{5.81 > 1.5 \text{ OK}}$$

7) Base Sliding

Lateral Driving Forces

$$\begin{aligned} R_d &= P_{ah} + P_{qh} \\ &= 1543 \text{ lbs} + 643 \text{ lbs} \\ &= \mathbf{2186 \text{ lbs/ft}} \end{aligned}$$

Lateral Resisting Forces

$$\begin{aligned} R_r &= (W_f + W_1) \times \tan \phi \text{ of foundation} \\ &= (2148 + 8652) \times \tan 30 \\ &= 6235 \text{ lbs/ft} \end{aligned}$$

$$FS_{sl} = R_r / R_d = 6235 / 2186 = \mathbf{2.85 > 1.5 \text{ OK}}$$

8) Sliding at Lowest Reinforcement Level

Lateral Driving Forces *(at depth of 9.33')*

$$\begin{aligned} R_d &= P_{ah} + P_{qh} \\ &= 1344 \text{ lbs} + 600 \text{ lbs} \\ &= \mathbf{1944 \text{ lbs/ft}} \end{aligned}$$

Lateral Resisting Forces *(at depth of 9.33')*

$$\begin{aligned} \tau_{unit} &= 1550 + N \times \tan 17.4 \\ &= 1550 \text{ plf} + (9.33' \times 1.75' \times 120 \text{ pcf}) \tan 17.4 \\ &= \mathbf{2164 \text{ plf}} \end{aligned}$$

$$\begin{aligned} \tau_{soil} &= (\gamma H (B - W_u) \times \tan \phi \text{ (of reinforced material)}) \times C_{ds} \\ &= 120 \text{ pcf} \times 9.33' \times 7.25' \times \tan 34 \times 0.90 \\ &= \mathbf{4928 \text{ lbs/ft}} \end{aligned}$$

$$FS_{sl} = R_r / R_d = (2164 + 4928) / 1944 = \mathbf{3.65 > 1.5 \text{ OK}}$$



PART SIX

Appendix B

NCMA 2ND EDITION - COULOMB METHODOLOGY LEVEL SURCHARGE - 250 PSF

9) Bearing Pressure *(Note: Live load is added for e and Max Bearing Pressure)*

Eccentricity

Equation (3i)

$$\begin{aligned}
 e &= B/2 - (M_r - M_o)/R_v \\
 M_r &= M_r + W_q \times (W_u + L/2) \\
 &= 48600 + 1813 \times (1.75' + 7.25'/2) = 58345 \text{ ft-lbs} \\
 R_v &= W_f + W_1 + W_q \\
 &= (2100 + 8700 + 1813) = 12613 \text{ lbs/ft} \\
 e &= 9.0'/2 - (58345 - 8358)/(12613) \\
 &= 0.54'
 \end{aligned}$$

Applied Bearing Pressure

Equation (3j)

$$\begin{aligned}
 \sigma_v &= R_v / (B - 2e) \\
 &= (12613) / (9.0' - 2 \times 0.54') \\
 &= 1593 \text{ lbs/sf}
 \end{aligned}$$

Note:

The external analysis above is limited to simple overturning, sliding, applied bearing pressure and bearing capacity for the reinforced mass based on a level toe. No attempt has been made to evaluate the more complicated geotechnical concerns of settlement and global stability. Geotechnical site and soils evaluation is a site specific art and cannot be programmed.

10) Bearing Capacity

Equation (3k)

$$Q_{ult} = cN_c + \gamma D N_q + 0.5 \gamma BN \gamma$$

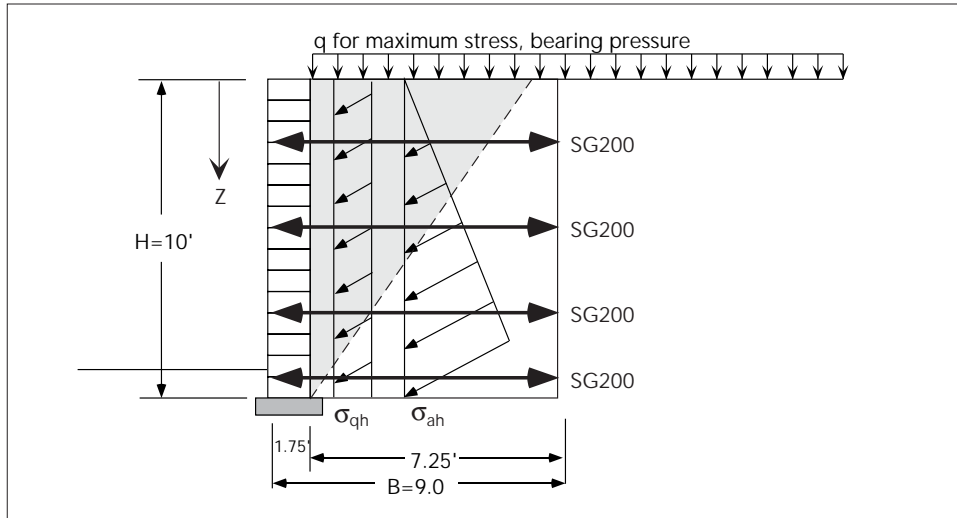
where:

$$\begin{aligned}
 N_c &= 30.14, N_q = 18.4, N\gamma = 22.40 \\
 B &= (B - 2e) = (9.0' - 2 \times 0.54') = 7.92' \\
 D &= 1.0' \text{ level embedment} \\
 c &= 0 \\
 Q_{ult} &= 0 + (120)(1)(18.4) + (0.5)(120)(7.92)(22.40) \\
 &= 12852 \text{ psf} \\
 F_{Sbr} &= 12852/1593 = 8.07 > 2.0 \text{ OK}
 \end{aligned}$$



NCMA 2ND EDITION - COULOMB METHODOLOGY LEVEL SURCHARGE - 250 PSF

Internal Stability - The internal analysis must look at the maximum loads at each grid level, connection strength, pullout resistance, and local stability concerns:



INTERNAL STABILITY ANALYSIS - LEVEL, 250 PSF SURCHARGE

The internal earth pressure at any level is calculated as follows:

$$\begin{aligned}\sigma_{ah} &= \gamma Z k_a \cos(\delta - \alpha) \\ &= (120\text{pcf})(Z)(0.254) \cos(22.67^\circ - 0^\circ) \\ &= 28.1(Z) \text{ psf/lf} \\ \sigma_{qh} &= qk_a \cos(\delta - \alpha) \\ &= (250\text{psf})(0.254) \cos(22.67^\circ - 0^\circ) \\ &= 58.6 \text{ psf/lf}\end{aligned}$$

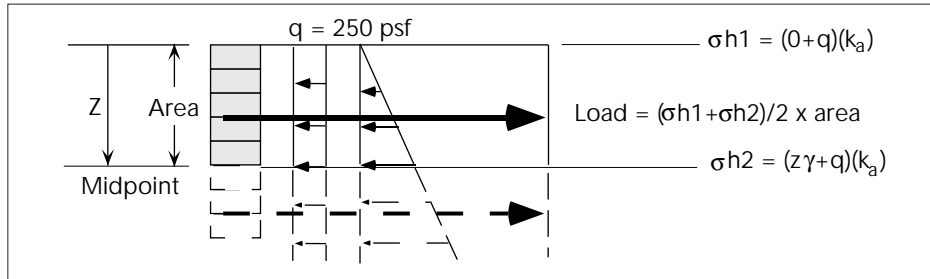
The calculated pressure is applied to the tributary area of each reinforcement level that determines the tensile load in the geogrid reinforcement.



NCMA 2ND EDITION - COULOMB METHODOLOGY LEVEL SURCHARGE - 250 PSF

11) Maximum Grid Tension

The calculated grid tensions (plf) are tabulated below:



| | GRID | DEPTH | z | σ_{ah} | σ_{qh} | σ_{tot} | Ave | Area | Load |
|----|-------|--------|-------|---------------|---------------|----------------|-----|------|------|
| | | TOP | 0.0 | 0.0 | 58.6 | 59 | | | |
| 4) | SG200 | 2.00' | | | | | 106 | 3.33 | 353 |
| | | MID | 3.33 | 94 | 58.6 | 152 | | | |
| 3) | SG200 | 4.67' | | | | | 190 | 2.67 | 507 |
| | | MID | 6.00 | 169 | 58.6 | 228 | | | |
| 2) | SG200 | 7.33' | | | | | 261 | 2.33 | 608 |
| | | MID | 8.33 | 234 | 58.6 | 293 | | | |
| 1) | SG200 | 9.33' | | | | | 316 | 1.67 | 528 |
| | | BOTTOM | 10.00 | 281 | 58.6 | 340 | | | |

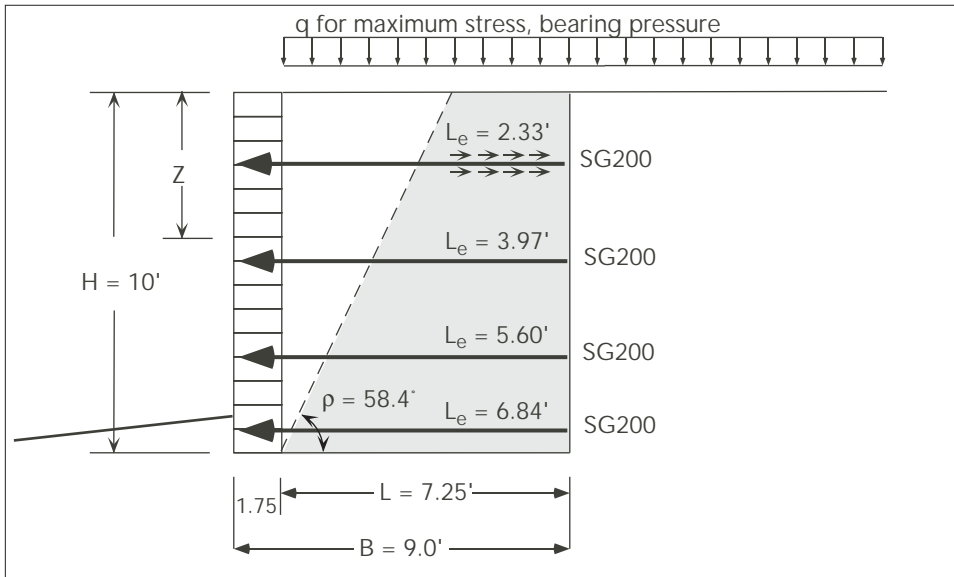
Strata SG200 has an allowable design capacity of 1280 plf from the first page which is greater than the calculated value at each level. Therefore, Strata SG200 is OK for all four levels in tension.



NCMA 2ND EDITION - COULOMB METHODOLOGY LEVEL SURCHARGE - 250 PSF

12) Pullout Resistance

Pullout safety factors are determined on a level by level basis. The effective lengths and calculated pullout is determined at each level and compared to a safety factor of 1.5.



LEVEL BY LEVEL PULLOUT ANALYSIS

Check each grid level for available pullout resistance against previously calculated tensile loads. Surcharge is not considered as a resisting force under NCMA guidelines:

Pullout Resistance = $(\gamma H_{ov}) (2L_e) (\tan(\phi) C_i)$ with H_{ov} = average height of over burden.

$L_e = (L - \text{Height} / \tan \rho)$

| | Height | Grid | Hov | γ | L_e | Tan34 | C_i | Pullout Load | | FS_{po} |
|----|--------|-------|------|----------|-------|-------|-------|--------------|-----|-----------|
| 4) | 8.00' | SG200 | 2.00 | 120 | 2.33 | .674 | 0.90 | 679 | 353 | 1.92 |
| 3) | 5.33' | SG200 | 4.67 | 120 | 3.97 | .674 | 0.90 | 2701 | 507 | 5.33 |
| 2) | 2.67' | SG200 | 7.33 | 120 | 5.60 | .674 | 0.90 | 5980 | 608 | 9.84 |
| 1) | 0.67' | SG200 | 9.33 | 120 | 6.84 | .674 | 0.90 | 9298 | 528 | 17.61 |

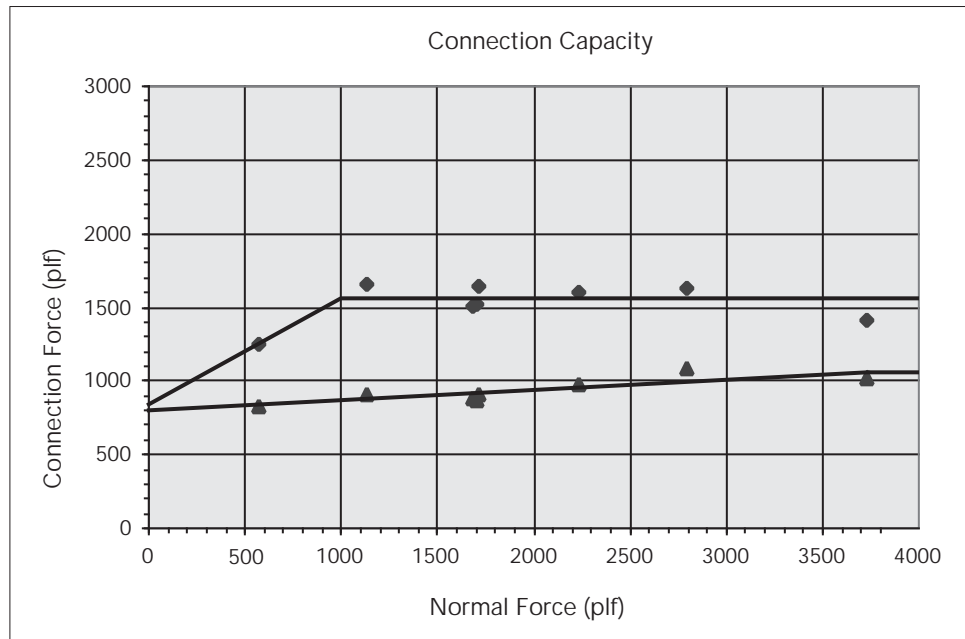
OK - All pullout safety factors are greater than 1.5.



NCMA 2ND EDITION - COULOMB METHODOLOGY LEVEL SURCHARGE - 250 PSF

13) Connection Strength

The last major item to check is the geogrid connection strength. KeyWall incorporates the laboratory connection test data for all Keystone unit types connected to different geogrid types. The following chart is applicable for Standard units and Stratagrid SG200 geogrid in this example:



The equations for these connection curves are:

$$\text{Peak Connection} = 834 \text{ plf} + N \tan 35.8^\circ < 1567 \text{ plf Max} / 1.5 \text{ Factor of Safety}$$

$$\frac{3}{4}'' \text{ Serviceability} = 795 \text{ plf} + N \tan 4.1^\circ < 1062 \text{ plf Max}$$

| | Height | Grid | Depth | N | Tpeak | TServ | Load |
|----|--------|-------|-------|------|-------|-------|------|
| 4) | 8.00' | SG200 | 2.00' | 420 | 758 | 825 | 353 |
| 3) | 5.33' | SG200 | 4.67' | 981 | 1028 | 865 | 507 |
| 2) | 2.67' | SG200 | 7.33' | 1539 | 1045 | 905 | 608 |
| 1) | 0.67' | SG200 | 9.33' | 1959 | 1045 | 935 | 528 |

OK - Calculated loads are less than the maximum allowable for Peak and Serviceability connection criteria.



NCMA 2ND EDITION - COULOMB METHODOLOGY LEVEL SURCHARGE - 250 PSF

14) Other Design Checks

The KeyWall program also checks the spacing between geogrid levels and the cantilever at the top of wall against the stability of the facing units. Standard Keystone units are typically spaced no greater than 4 blocks between geogrid levels to remain stable during construction and eliminate concerns over local stability. The cantilever at the top of wall is also checked against the final loading condition as a small gravity wall. By inspection, the three unit vertical cantilever is ok with the 250 psf surcharge and the four block maximum spacing between geogrids will be stable during construction and in the final design condition.

Summary

The hand calculations verify the attached computer output. The data and methods conform to the NCMA, 1997.



PART SIX
Appendix B

NCMA 2ND EDITION - COULOMB METHODOLOGY LEVEL SURCHARGE - 250 PSF



RETAINING WALL DESIGN

KeyWall_2010 Version 3.7.1 Build 1

Project: Part 6; Design Examples
Project No: NA
Case: Appendix B
Design Method: NCMA 2nd Edition (parallelogram soil interface)

Date: 8/31/2010
Designer: JLG

Design Parameters

| Soil Parameters: | ϕ | c psf | γ pcf |
|------------------------------|-----------------------------|-------|--------------|
| Reinforced Fill | 34 | 0 | 120 |
| Retained Zone | 30 | 0 | 120 |
| Foundation Soil | 30 | 0 | 120 |
| Reinforced Fill Type: | Sand, Silt or Clay | | |
| Unit Fill: | Crushed Stone, 1 inch minus | | |



Minimum Design Factors of Safety

| | | | | | |
|--------------|------|----------|------|-----------------|------|
| sliding: | 1.50 | pullout: | 1.50 | uncertainties: | 1.50 |
| overturning: | 2.00 | shear: | 1.50 | connection: | 1.50 |
| bearing: | 2.00 | bending: | 1.50 | Serviceability: | 1.00 |

Reinforcing Parameters: Strata-Grid Geogrids

| | T_{ult} | R_{Fcr} | R_{Fd} | R_{Fid} | $LTDS$ | FS | T_{al} | C_i | C_{ds} |
|-------|-----------|-----------|----------|-----------|--------|------|----------|-------|----------|
| SG200 | 3600 | 1.55 | 1.10 | 1.10 | 1919 | 1.50 | 1280 | 0.90 | 0.90 |

Analysis: Case: Appendix B

NCMA 2nd Edition - Level Surcharge 250 psf

| | | | |
|----------------|-------------------------------|--------------|-------------------------|
| Unit Type: | Standard 21" / 120.00 pcf | Wall Batter: | 0.00 deg. |
| Leveling Pad: | Crushed Stone | embedment: | 1.00 ft |
| Wall Hr: | 10.00 ft | | |
| Level Backfill | Offset: 0.00 ft | | |
| Surcharge: | LL: 250 psf uniform surcharge | DL: | 0 psf uniform surcharge |
| | Load Width: 100.00 ft | | Load Width: 100.00 ft |

| Results: | <u>Sliding</u> | <u>Overturning</u> | <u>Bearing</u> | <u>Shear</u> | <u>Bending</u> |
|--------------------|----------------|--------------------|----------------|--------------|----------------|
| Factors of Safety: | 2.85 | 5.81 | 8.08 | 6.19 | 2.37 |

Calculated Bearing Pressure: 1591 / 1449 psf

Eccentricity at base: 0.54 ft

Reinforcing: (ft & lbs/ft)

| Layer | Height | Length | Calc. Tension | Reinf. Type | Allow Ten Tal | Pk Conn Tcl | Serv Conn Tsc | Pullout FS |
|-------|--------|--------|---------------|-------------|---------------|-------------|---------------|------------|
| 4 | 8.00 | 9.0 | 352 | SG200 | 1280 ok | 758 ok | 825 ok | 1.92 ok |
| 3 | 5.33 | 9.0 | 507 | SG200 | 1280 ok | 1027 ok | 865 ok | 5.32 ok |
| 2 | 2.67 | 9.0 | 608 | SG200 | 1280 ok | 1045 ok | 905 ok | 9.86 ok |
| 1 | 0.67 | 9.0 | 528 | SG200 | 1280 ok | 1045 ok | 935 ok | >10 ok |

Reinforcing Quantities (no waste included):

SG200 4.00 sy/ft

NOTE: THESE CALCULATIONS ARE FOR PRELIMINARY DESIGN ONLY AND SHOULD NOT BE USED FOR CONSTRUCTION WITHOUT REVIEW BY A QUALIFIED ENGINEER

PART SIX



APPENDIX C

Clay Terrace, Carmel, Indiana; Keystone Standard





RANKINE METHODOLOGY LEVEL SURCHARGE - 250 PSF

This set of calculations is intended to verify the KeyWall program output of a typical reinforced soil wall design section. The design follows the Rankine procedure outlined previously in the Keystone Design Manual. The pertinent design information is summarized below:

1) General Design Data

Standard Keystone Units (120 pcf with drainage fill and $W_u = 1.75'$)
 Stratagrid SG200 Polyester Geogrid
 Wall Batter(ι) = 0° , near-vertical orientation
 Design Height = 10' (9' exposed + 1' embedment)
 Base Length, B = 8.5' (uniform lengths chosen for simplicity)
 Backslope, $\beta = 0$, level backslope
 Surcharge = 250 psf (typical roadway surcharge)

2) Soil Parameters (*degrees, psf, pcf*)

| SOIL PARAMETERS | ϕ | C | γ |
|-----------------|--------|---|----------|
| Reinforced Soil | 34 | 0 | 120 |
| Retained Soil | 30 | 0 | 120 |
| Foundation Soil | 30 | 0 | 120 |

3) Geogrid Design Parameters (*plf*)

| Geogrid | T_{ult} | RF_{cr} | RF_d | RF_{id} | LTDS | FS | T_{al} |
|--------------|-----------|-----------|--------|-----------|------|-----|----------|
| Strata SG200 | 3600 | 1.55 | 1.10 | 1.10 | 1919 | 1.5 | 1280plf |

C_i & $C_d = 0.90$ for select backfill

4) Geometric Parameters - Rankine

Internal

$\phi = 34$ degrees
 $\delta = \beta = 0$ degrees (no backslope)
 $\alpha = 90$ degrees ($90^\circ +$ no batter)
 $\beta = 0$ degrees (level)

External

$\phi = 30$ degrees
 $\delta = \beta = 0$ degrees (no backslope)
 $\alpha = 90$ degrees ($90^\circ +$ no batter)
 $\beta = 0$ degrees (level)



RANKINE METHODOLOGY LEVEL SURCHARGE - 250 PSF

5) Rankine Earth Pressure Calculation

Internal

Equation (3g)

$$k_a = \tan^2 (45 - \phi/2)$$

$$k_a = \mathbf{0.283} \text{ for above parameters - Rankine}$$

Equation (3h)

$$\rho = 45 + \phi/2$$

$$\rho = \mathbf{62.0^\circ} \text{ for above parameters - Rankine}$$

External

Equation (3g)

$$k_a = \tan^2 (45 - \phi/2)$$

$$k_a = \mathbf{0.333} \text{ for above parameters - Rankine}$$

External Forces

Equation. (3e)

$$P_a = \frac{1}{2} \gamma H^2 k_a$$

$$P_{ah} = \frac{1}{2} \gamma H^2 k_a \cos(\beta) - \text{Horizontal Component}$$

$$P_{ah} = (0.5)(120 \text{ pcf})(10')^2 (0.333) \cos(0)$$

$$P_{ah} = \mathbf{1998 \text{ lbs/lf}}$$

Equation. (3f)

$$P_q = qH k_a$$

$$P_{qh} = qH k_a \cos(\beta) - \text{Horizontal Component}$$

$$P_{qh} = (250 \text{ psf})(10')(0.333) \cos(0)$$

$$P_{qh} = \mathbf{833 \text{ lbs/lf}}$$

External Masses

$$W_f = W_u H \gamma = (1.75')(10')(120 \text{ pcf}) = \mathbf{2100 \text{ lbs/lf}}$$

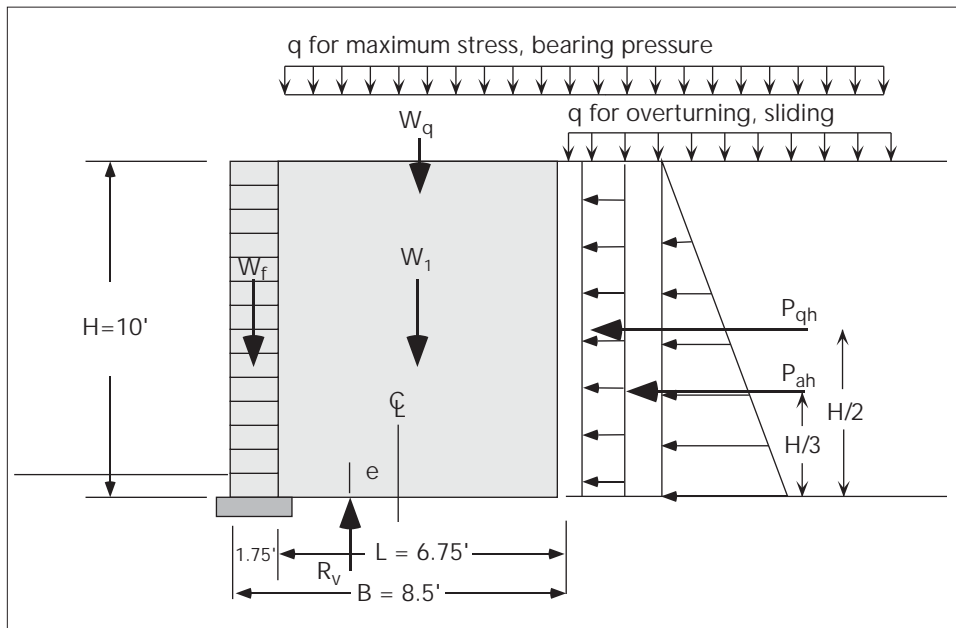
$$W_l = (B - W_u) H \gamma = (8.5' - 1.75')(10')(120 \text{ pcf}) = \mathbf{8100 \text{ lbs/lf}}$$

$$W_q = q (B - W_u) = (250 \text{ psf})(8.5' - 1.75') = \mathbf{1688 \text{ lbs/lf}}$$



RANKINE METHODOLOGY LEVEL SURCHARGE - 250 PSF

External Stability Diagram



6) Overturning

Overturning Moment

$$\begin{aligned} M_o &= P_{ah} (H/3) + P_{qh} (H/2) \\ &= 1998 \text{ lbs} (10'/3) + 833 \text{ lbs} (10'/2) \\ &= \mathbf{10825 \text{ ft-lbs}} \end{aligned}$$

Resisting Moment

$$\begin{aligned} M_r &= W_f \times W_u/2 + W_1 \times (W_u + L/2) \\ &= (2100 \times 1.75'/2) + 8100(1.75' + 6.75'/2) \\ &= \mathbf{43613 \text{ ft-lbs}} \\ FS_{ot} &= M_r / M_o = 43613/10825 = \mathbf{4.03} > 1.5 \text{ OK} \end{aligned}$$



RANKINE METHODOLOGY LEVEL SURCHARGE - 250 PSF

7) Base Sliding

Lateral Driving Forces

$$\begin{aligned} R_d &= P_{ah} + P_{qh} \\ &= 1998 \text{ lbs} + 833 \text{ lbs} \\ &= \mathbf{2831 \text{ lbs/ft}} \end{aligned}$$

Lateral Resisting Forces

$$\begin{aligned} R_r &= (W_f + W_1) \times \tan \phi \text{ of foundation} \\ &= (2100 + 8100) \times 0.577 \\ &= \mathbf{5885 \text{ lbs/ft}} \\ FS_{sl} &= R_r / R_d = 5885 / 2831 = \mathbf{2.08} > 1.5 \text{ OK} \end{aligned}$$

8) Sliding at Lowest Reinforcement Level

Lateral Driving Forces *(at depth of 9.33')*

$$\begin{aligned} R_d &= P_{ah} + P_{qh} \\ &= 1739 \text{ lbs} + 777 \text{ lbs} \\ &= \mathbf{2516 \text{ lbs/ft}} \end{aligned}$$

Lateral Resisting Forces *(at depth of 9.33')*

$$\begin{aligned} \tau_{unit} &= 1550 + N \tan 17.4 \\ &= 1550 \text{ plf} + (9.33' \times 1.75' \times 120 \text{ pcf}) \tan 17.4 \\ &= \mathbf{2164 \text{ plf}} \\ \tau_{soil} &= (\gamma H (B - W_u)) \times \tan \phi \text{ (of reinforced material)} \times C_{ds} \\ &= 120 \text{ pcf} \times 9.33' \times 6.75' \times 0.675 \times 0.90 \\ &= \mathbf{4591 \text{ lbs/ft}} \\ FS_{sl} &= R_r / R_d = (2164 + 4591) / 2516 = \mathbf{2.68} > 1.5 \text{ OK} \end{aligned}$$



RANKINE METHODOLOGY LEVEL SURCHARGE - 250 PSF

9) Bearing Pressure *(Note: Live load is added for “e” and Max Bearing Pressure)*

Eccentricity

Equation (3i)

$$\begin{aligned} e &= B/2 - (M_r - M_o)/R_v \\ M_r &= M_r + W_q \times (W_u + L/2) \\ &= 43613 + (1688 \times (1.75' + 6.75'/2)) = 52001 \text{ ft-lbs} \\ R_v &= W_f + W_1 + W_q \\ &= (2100 + 8100 + 1688) = 11888 \text{ lbs/ft} \\ e &= 8.5'/2 - (52001 - 10825)/(11888) \\ &= \mathbf{0.79'} \end{aligned}$$

Applied Bearing Pressure

Equation (3j)

$$\begin{aligned} \sigma_v &= R_v / (L - 2e) \\ &= (11888) / (8.5' - 2 \times 0.79') \\ &= \mathbf{1718 \text{ lbs/sf}} \end{aligned}$$

10) Bearing Capacity

Equation (3k)

$$\begin{aligned} Q_{ult} &= cN_c + \gamma DN_q + 0.5\gamma(B-2)N_\gamma \\ \text{where:} \\ N_c &= 30.14, N_q = 18.4, N_\gamma = 22.40 \\ B &= (B-2e) = (8.5' - 2 \times 0.79) = 6.92' \\ D &= 1.0' \text{ level embedment} \\ c &= 0 \\ Q_{ult} &= 0 + (120)(1)(18.4) + (0.5)(120)(6.92)(22.40) \\ &= \mathbf{11508 \text{ psf}} \\ FS_{br} &= 11508/1716 = \mathbf{6.71} > 2.0 \text{ OK} \end{aligned}$$

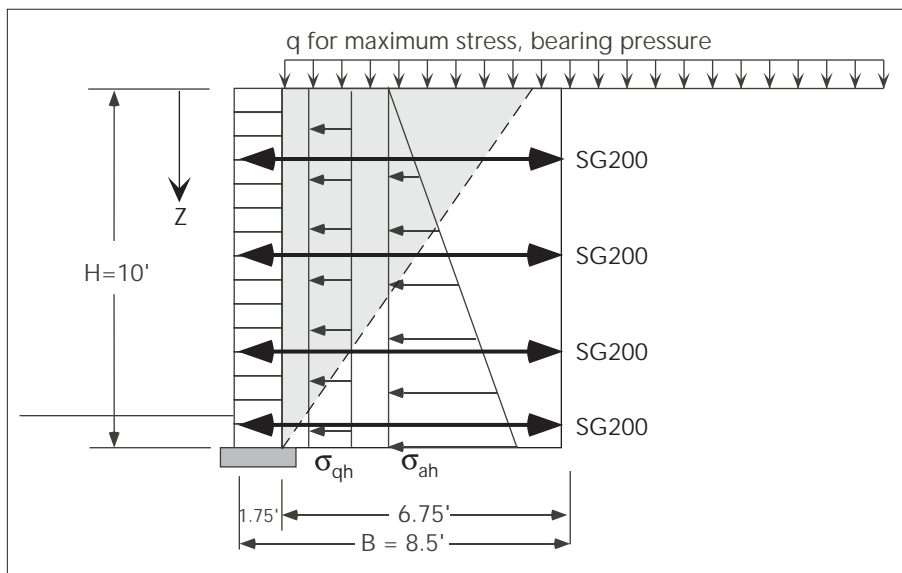
Note:

The external analysis above is limited to simple overturning, sliding, applied bearing pressure and bearing capacity for the reinforced mass based on a level toe. No attempt has been made to evaluate the more complicated geotechnical concerns of settlement and global stability. Geotechnical site and soils evaluation is a site specific art and can not be programmed.



RANKINE METHODOLOGY LEVEL SURCHARGE - 250 PSF

Internal Stability - The internal analysis must look at the maximum loads at each grid level, connection strength, pullout resistance, and local stability concerns:



The internal earth pressure at any level is calculated as follows:

$$\begin{aligned}\sigma_{ah} &= \gamma Z k_a \cos(\beta) \\ &= (120\text{pcf})(Z)(0.283) \cos(0) \\ &= \mathbf{34.0 (Z) \text{ plf}} \\ \sigma_{qh} &= q k_a \cos(\beta) \\ &= (250\text{psf})(0.283) \cos(0) \\ &= \mathbf{70.8 \text{ plf}}\end{aligned}$$

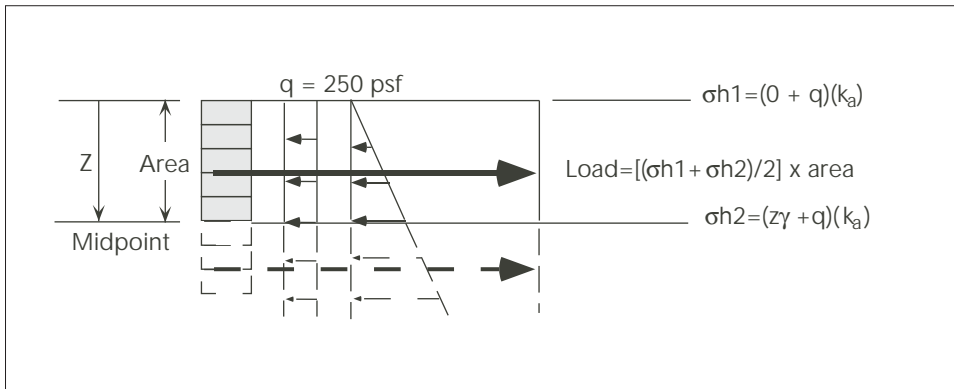
The calculated pressure is applied to the tributary area of each reinforcement level which determines the tensile load in the geogrid reinforcement.



RANKINE METHODOLOGY LEVEL SURCHARGE - 250 PSF

11) Maximum Grid Tension

The calculated grid tensions (plf) are tabulated below:



| | GRID | DEPTH | z | σ_{ah} | σ_{qh} | σ_{tot} | Ave | Area | Load |
|----|-------|--------|-------|---------------|---------------|----------------|-----|------|------|
| | | TOP | 0.0 | 0.0 | 70.8 | 71 | | | |
| 4) | SG200 | 2.00' | | | | | 127 | 3.33 | 425 |
| | | MID | 3.33 | 113 | 70.8 | 184 | | | |
| 3) | SG200 | 4.67' | | | | | 229 | 2.67 | 611 |
| | | MID | 6.00 | 204 | 70.8 | 275 | | | |
| 2) | SG200 | 7.33' | | | | | 314 | 2.33 | 732 |
| | | MID | 8.33 | 283 | 70.8 | 354 | | | |
| 1) | SG200 | 9.33' | | | | | 382 | 1.67 | 637 |
| | | BOTTOM | 10.00 | 340 | 70.8 | 411 | | | |

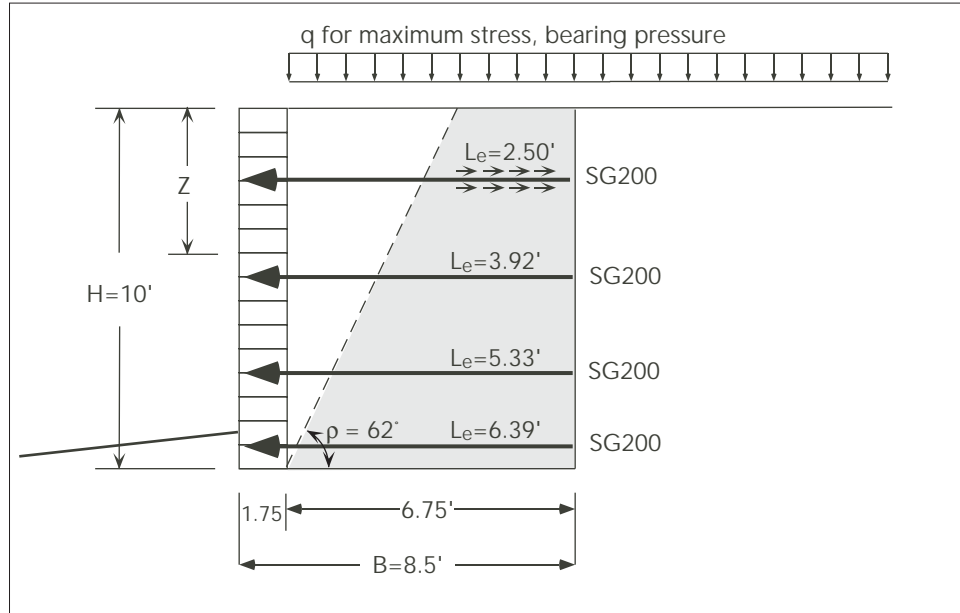
Strata SG200 has an allowable design capacity of 1280 plf from the first page which is greater than the calculated value at each level. Therefore, Strata SG200 is OK for all four levels in tension.



RANKINE METHODOLOGY LEVEL SURCHARGE - 250 PSF

12) Pullout Resistance

Pullout safety factors are determined on a level by level basis. The effective lengths and calculated pullout is determined at each level and compared to a safety factor of 1.5.



Check each grid level for available pullout resistance against previously calculated tensile loads, a live load surcharge is not considered as a resisting force:

Pullout Resistance = $(\gamma H_{ov}) (2L_e) (\tan(\phi) C_i)$ with H_{ov} = average height of over burden.

$L_e = (L - \text{Height} / \tan \rho)$

| | Height | Grid | H _{ov} | γ | L _e | Tan34 | C _i | Pullout Load | FS _{po} |
|----|--------|-------|-----------------|-----|----------------|-------|----------------|--------------|------------------|
| 4) | 8.00' | SG200 | 2.00 | 120 | 2.50 | .674 | 0.90 | 728 425 | 1.71 |
| 3) | 5.33' | SG200 | 4.67 | 120 | 3.93 | .674 | 0.90 | 2665 611 | 4.36 |
| 2) | 2.67' | SG200 | 7.33 | 120 | 5.33 | .674 | 0.90 | 5688 732 | 7.77 |
| 1) | 0.67' | SG200 | 9.33 | 120 | 6.39 | .674 | 0.90 | 8680 637 | 13.63 |

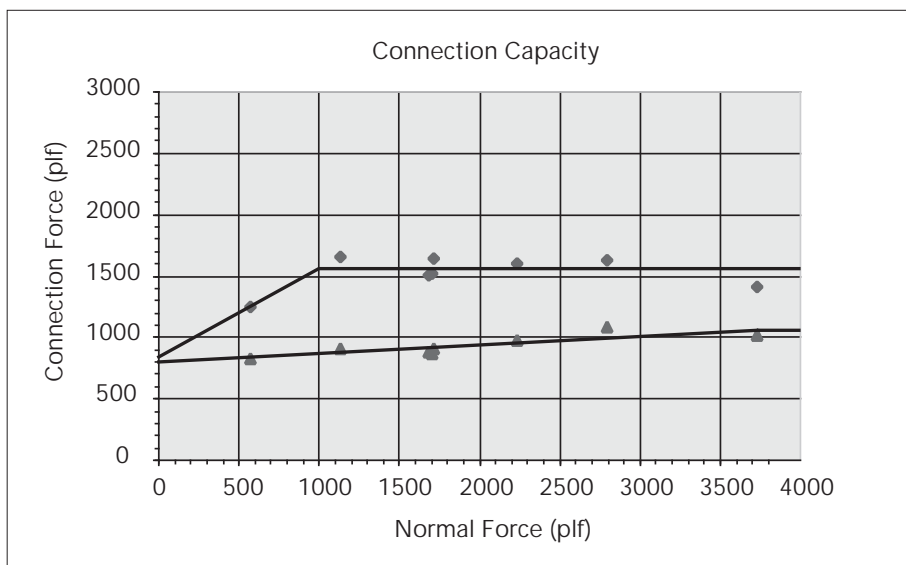
OK - All pullout safety factors are greater than 1.5



RANKINE METHODOLOGY LEVEL SURCHARGE - 250 PSF

13) Connection Strength

The last major item to check is the geogrid connection strength. KeyWall incorporates the laboratory connection test data for all Keystone unit types connected to different geogrid types. The following chart is applicable for Standard units and Stratagrid SG200 geogrid in this example:



The equations for these connection curves are:

$$\text{Peak Connection} = 834 \text{ plf} + N \tan 35.80^\circ < 1567 \text{ plf Max} / 1.5 \text{ Factor of Safety}$$

$$\frac{3}{4}'' \text{ Serviceability} = 795 \text{ plf} + N \tan 4.10^\circ < 1062 \text{ plf Max}$$

| | Height | Grid | Depth | N | T _{peak} | T _{Serv} | Load |
|----|--------|-------|-------|------|-------------------|-------------------|------|
| 4) | 8.00' | SG200 | 2.00' | 420 | 758 | 825 | 425 |
| 3) | 5.33' | SG200 | 4.67' | 981 | 1027 | 865 | 611 |
| 2) | 2.67' | SG200 | 7.33' | 1539 | 1045 | 905 | 732 |
| 1) | 0.67' | SG200 | 9.33' | 1959 | 1045 | 935 | 637 |

OK - Calculated loads are less than the maximum allowable for Peak connection criteria.
(Rankine method does not check serviceability as the default setting)



RANKINE METHODOLOGY LEVEL SURCHARGE - 250 PSF

14) Other Design Checks

The KeyWall program also checks the spacing between geogrid levels and the cantilever at the top of wall against the stability of the facing units. Standard Keystone units are typically spaced no greater than 4 blocks between geogrid levels to remain stable during construction and eliminate concerns over local stability. The cantilever at the top of wall is also checked against the final loading condition as a small gravity wall. By inspection, the three unit vertical cantilever is ok with the 250 psf surcharge and the four block maximum spacing between geogrids will be stable during construction and in the final design condition.

Summary

The hand calculations verify the attached computer output. The data and methods conform to the Rankine design method as outlined in the Keystone Design Manual.



RANKINE METHODOLOGY LEVEL SURCHARGE - 250 PSF



RETAINING WALL DESIGN

KeyWall_2010 Version 3.7.1 Build 1

Project: Part 6; Design Examples

Project No: NA

Case: Appendix C

Design Method: Rankine-w/Batter (modified soil interface)

Date: 8/31/2010

Designer: JLG

Design Parameters

| Soil Parameters: | ϕ | c psf | γ pcf |
|------------------------------|-----------------------------|---------|--------------|
| Reinforced Fill | 34 | 0 | 120 |
| Retained Zone | 30 | 0 | 120 |
| Foundation Soil | 30 | 0 | 120 |
| Reinforced Fill Type: | Sand, Silt or Clay | | |
| Unit Fill: | Crushed Stone, 1 inch minus | | |



Minimum Design Factors of Safety

| | | | | | |
|--------------|------|----------|------|----------------|------|
| sliding: | 1.50 | pullout: | 1.50 | uncertainties: | 1.50 |
| overturning: | 2.00 | shear: | 1.50 | connection: | 1.50 |
| bearing: | 2.00 | bending: | 1.50 | | |

Reinforcing Parameters: Strata-Grid Geogrids

| | T_{ult} | RF_{cr} | RF_d | RF_{id} | $LTDS$ | FS | T_{al} | C_i | C_{ds} |
|-------|-----------|-----------|--------|-----------|--------|------|----------|-------|----------|
| SG200 | 3600 | 1.55 | 1.10 | 1.10 | 1919 | 1.50 | 1280 | 0.90 | 0.90 |

Analysis:

Case: Appendix C

Rankine Methodology - Level Surcharge 250 psf

Unit Type: Standard 21" / 120.00 pcf

Leveling Pad: Crushed Stone

Wall Ht: 10.00 ft

Level Backfill: Offset: 0.00 ft

Surcharge: LL: 250 psf uniform surcharge

Load Width: 100.00 ft

Wall Batter: 0.00 deg.

embedment: 1.00 ft

DL: 0 psf uniform surcharge

Load Width: 100.00 ft

Results:

| | <u>Sliding</u> | <u>Overturning</u> | <u>Bearing</u> | <u>Shear</u> | <u>Bending</u> |
|--------------------|----------------|--------------------|----------------|--------------|----------------|
| Factors of Safety: | 2.08 | 4.00 | 6.71 | 5.14 | 1.97 |

Calculated Bearing Pressure: 1716 / 1600 psf

Eccentricity at base: 0.79 ft

Reinforcing: (ft & lbs/ft)

| Laver | Height | Length | Calc. Tension | Reinf. Type | Allow Ten Tal | Pk Conn Tcl | Serv Conn Tsc | Pullout FS |
|-------|--------|--------|---------------|-------------|---------------|-------------|---------------|------------|
| 4 | 8.00 | 8.5 | 424 | SG200 | 1280 ok | 758 ok | N/A | 1.72 ok |
| 3 | 5.33 | 8.5 | 611 | SG200 | 1280 ok | 1027 ok | N/A | 4.36 ok |
| 2 | 2.67 | 8.5 | 732 | SG200 | 1280 ok | 1045 ok | N/A | 7.78 ok |
| 1 | 0.67 | 8.5 | 636 | SG200 | 1280 ok | 1045 ok | N/A | >10 ok |

Reinforcing Quantities (no waste included):

SG200 3.78 sy/ft

NOTE: THESE CALCULATIONS ARE FOR PRELIMINARY DESIGN ONLY AND SHOULD NOT BE USED FOR CONSTRUCTION WITHOUT REVIEW BY A QUALIFIED ENGINEER

PART SIX



APPENDIX D

Glasshoughton Railway Station, Glasshoughton, Yorkshire, UK; Keystone Compac





3H:1V SLOPING SURCHARGE AASHTO METHODOLOGIES

This set of calculations is intended to verify the KeyWall program output of a typical reinforced soil wall design section. The design follows the basic AASHTO procedures outlined previously in the Keystone Design Manual and compares the internal difference of AASHTO 96 vs AASHTO Simplified. The pertinent design information is summarized below:

1) General Design Data

Keystone Compac II Units (120 pcf with drainage fill and $W_u = 1.00'$)
 Tensor UXK1400 HDPE Geogrid
 Wall Batter (ι) = 0° , near-vertical orientation
 Design Height = 10' (9' exposed + 1' embedment)
 Base Length, B = 9.0' (70% min B/H or 8' minimum)
 Backslope, β = 18.4° , 3H:1V backslope
 Surcharge = slope only

2) Soil Parameters (*degrees, psf, pcf*)

| SOIL PARAMETERS | ϕ | c | γ |
|-----------------|--------|---|----------|
| Reinforced Soil | 34 | 0 | 120 |
| Retained Soil | 30 | 0 | 120 |
| Foundation Soil | 30 | 0 | 120 |

3) Geogrid Design Parameters (*plf*)

| Geogrid | T_{ult} | FS_{cr} | RF_d | RF_{id} | LTDS | FS | T_{al} |
|----------------|-----------|-----------|--------|-----------|------|-----|----------------|
| Tensor UXK1400 | 4800 | 2.60 | 1.10 | 1.10 | 1526 | 1.5 | 1017plf |

C_i & $C_d = 0.90$ for select backfill

4) Geometric Parameters - AASHTO

| Internal | | External | |
|----------|--|----------|--|
| ϕ | = 34 degrees | ϕ | = 30 degrees |
| δ | = $\beta = 18.4$ degrees | δ | = $\beta = 18.4$ degrees |
| α | = 90 degrees (90° + no batter) | α | = 90 degrees (90° + no batter) |
| β | = 18.4 degrees (Infinite slope) | β | = 18.4 degrees (Infinite slope) |



3H:1V SLOPING SURCHARGE AASHTO METHODOLOGIES

5) Rankine Earth Pressure Calculation

Internal - AASHTO 96

Equation (3g)

$$k_a = \cos \beta \frac{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \phi}}{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi}}$$

$$k_a = 0.328 \text{ for above parameters - Rankine}$$

Equation (3h)

$$\rho = 45 + \phi/2$$

$$\rho = 62.0^\circ \text{ for above parameters - Rankine}$$

Internal - AASHTO Simplified

Equation (3g)

$$k_a = \tan^2 (45 - \phi/2)$$

$$k_a = 0.283 \text{ for above parameters - Rankine}$$

Equation (3h)

$$\rho = 45 + \phi/2$$

$$\rho = 62.0^\circ \text{ for above parameters - Rankine}$$

External

Equation (3g)

$$k_a = \cos \beta \frac{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \phi}}{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi}}$$

$$k_a = 0.398 \text{ for above parameters - Rankine}$$

External Forces

Equation (3e)

$$P_a = \frac{1}{2} \gamma H S^2 k_a$$

$$HS = H + (B - W_u) \tan \beta$$

$$HS = 10' + (9' - 1.0') \tan 18.4^\circ$$

$$HS = 12.66'$$

$$P_{ah} = \frac{1}{2} \gamma H S^2 k_a \cos(\beta) - \text{Horizontal Component}$$

$$P_{ah} = (0.5)(120 \text{ pcf})(12.66')^2(0.398) \cos(18.4)$$

$$P_{ah} = 3632 \text{ lbs/lf}$$

$$P_{av} = \frac{1}{2} \gamma H S^2 k_a \sin(\beta) - \text{Vertical Component}$$

$$P_{av} = (0.5)(120 \text{ pcf})(12.66')^2(0.398) \sin(18.4)$$

$$P_{av} = 1208 \text{ lbs/lf}$$

External Masses

$$W_f = W_u H \gamma = (1.00')(10')(120 \text{ pcf}) = 1200 \text{ lbs/lf}$$

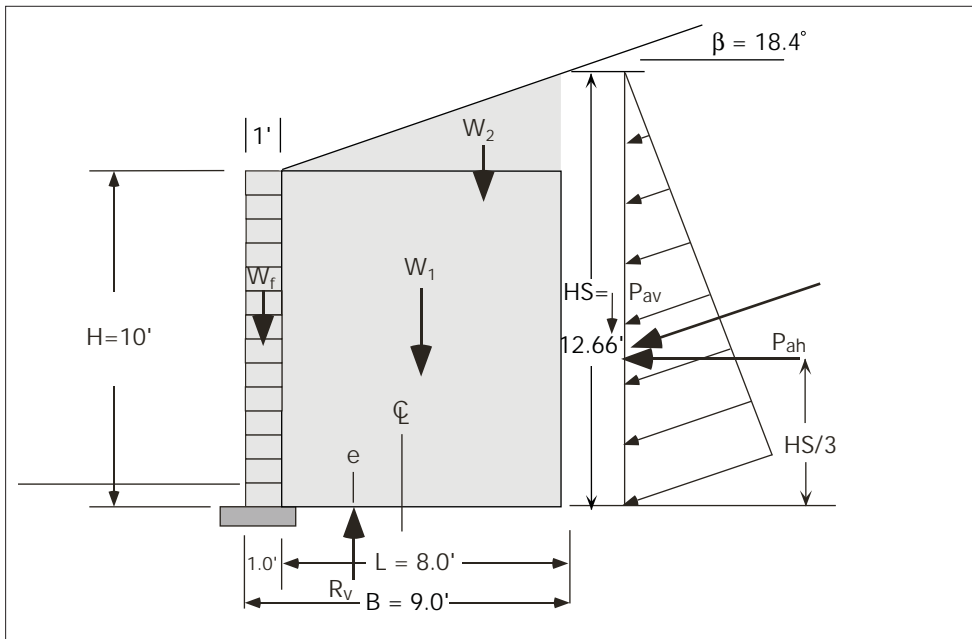
$$W_1 = (B - W_u) H \gamma = (9.0' - 1.0')(10')(120 \text{ pcf}) = 9600 \text{ lbs/lf}$$

$$W_2 = \frac{1}{2}(B - W_u)(HS - H) \gamma = \frac{1}{2}(9' - 1.0')(12.66' - 10') 120 \text{ pcf} = 1277 \text{ lbs/lf}$$



3H:1V SLOPING SURCHARGE AASHTO METHODOLOGIES

External Stability Diagram



6) Overturning

Overturning Moment

$$\begin{aligned} M_o &= P_{ah} (HS/3) \\ &= 3632 \text{ lbs} (12.66'/3) \\ &= \mathbf{15327 \text{ ft-lbs}} \end{aligned}$$

Resisting Moment

$$\begin{aligned} M_r &= W_f \times W_u/2 + W_1 \times (W_u + L/2) + W_2 \times (W_u + 2/3L) + (P_{av} \times B) \\ &= (1200 \times 1.0'/2) + 9600(1.0' + 8.0'/2) + 1277(1.0' + 5.33') + (1208 \times 9') \\ &= \mathbf{67555 \text{ ft-lbs}} \end{aligned}$$

$$FS_{ot} = M_r/M_o = 67555/15327 = \mathbf{4.41} > 1.5 \text{ OK}$$



3H:1V SLOPING SURCHARGE AASHTO METHODOLOGIES

7) Base Sliding

Lateral Driving Forces

$$\begin{aligned} R_d &= P_{ah} \\ &= 3632 \text{ lbs} \\ &= \mathbf{3632 \text{ lbs/ft}} \end{aligned}$$

Lateral Resisting Forces

$$\begin{aligned} R_r &= (W_f + W_1 + W_2 + P_{av}) \times \tan \phi \text{ of foundation} \\ &= (1200 + 9600 + 1277 + 1208) \times 0.577 \\ &= \mathbf{7665 \text{ lbs/ft}} \\ FS_{sl} &= R_r / R_d = 7665 / 3632 = \mathbf{2.11} > 1.5 \text{ OK} \end{aligned}$$

8) Sliding at Lowest Reinforcement Level

Lateral Driving Forces (*at depth of 9.33'*)

$$\begin{aligned} R_d &= P_{ah} @ 9.33' \\ &= \frac{1}{2}(120)(9.33' + 2.66')^2 (0.398) \cos 18.4^\circ \\ &= \mathbf{3257 \text{ lbs/ft}} \end{aligned}$$

Lateral Resisting Forces (*at depth of 9.33'*)

$$\begin{aligned} \tau_{unit} &= 1250 + N \tan 29^\circ \\ &= 1250 \text{ plf} + (9.33' \times 1.00' \times 120 \text{ pcf}) \tan 29^\circ \\ &= \mathbf{1871 \text{ plf}} \\ \tau_{soil} &= ((\gamma H (B - W_u)) + W_2 + P_{av}) \times \tan \phi \text{ (of reinforced material)} \times C_{ds} \\ &= ((120 \text{ pcf} \times 9.33 \times 8.0') + 1277 + 1208) \times 0.675 \times 0.90 \\ &= \mathbf{6951 \text{ lbs/ft}} \\ FS_{sl} &= R_r / R_d = (1871 + 6951) / 3257 = \mathbf{2.71} > 1.5 \text{ OK} \end{aligned}$$



3H:1V SLOPING SURCHARGE AASHTO METHODOLOGIES

9) Bearing Pressure *(Note: Live load is added for e and Max Bearing Pressure)*

Eccentricity

Equation (3i)

$$\begin{aligned} e &= B/2 - (M_r - M_o)/R_v \\ M_r &= M_r \text{ (no live load)} \\ &= 67555 \text{ ft-lbs} \\ R_v &= W_f + W_1 + W_2 + P_{av} \\ &= (1200 + 9600 + 1277 + 1208) = 13285 \text{ lbs/ft} \\ e &= 9.0'/2 - (67555 - 15327)/(13285) \\ &= 0.57' \end{aligned}$$

Applied Bearing Pressure

Equation (3j)

$$\begin{aligned} \sigma_v &= R_v / (B - 2e) \\ &= (13285) / ((9.0' - 2 \times 0.57')) \\ &= 1690 \text{ lbs/sf} \end{aligned}$$

10) Bearing Capacity

Equation (3k) $Q_{ult} = cN_c + \gamma DN_q + 0.5\gamma BN_\gamma$

where:

$$\begin{aligned} N_c &= 30.14, N_q=18.4, N_\gamma=22.40 \\ B &= (B - 2e) = (9.0' - 2 \times 0.57') = 7.86' \\ D &= 1.0' \text{ level embedment} \\ c &= 0 \\ Q_{ult} &= 0 + (120)(1)(18.4) + (0.5)(120)(7.86)(22.40) \\ &= 12772 \text{ psf} \\ FS_{br} &= 12772/1690 = 7.56 > 2.0 \text{ OK} \end{aligned}$$

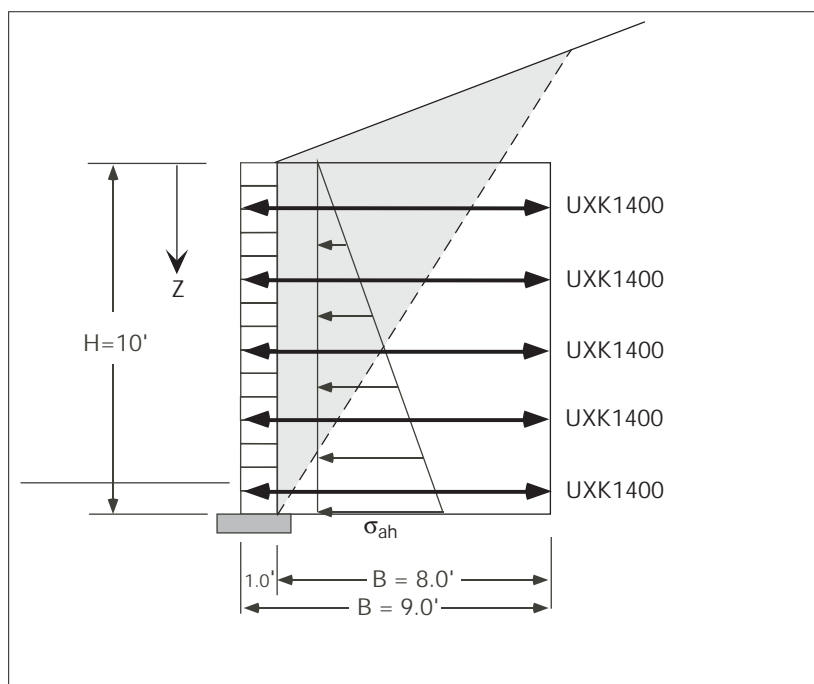
Note:

The external analysis above is limited to simple overturning, sliding, applied bearing pressure and bearing capacity for the reinforced mass based on a level toe. No attempt has been made to evaluate the more complicated geotechnical concerns of settlement and global stability. Geotechnical site and soils evaluation is a site specific art and can not be programmed.



3H:1V SLOPING SURCHARGE AASHTO METHODOLOGIES

Internal Stability - The internal analysis must look at the maximum loads at each grid level, connection strength, pullout resistance, and local stability concerns.



The internal earth pressure at any level is calculated as follows:

$$\begin{aligned}\sigma_{ah} &= \gamma Z k_a \cos(\beta) \\ &= (120\text{pcf})(Z)(0.328)\cos(18.4) \\ &= 37.3 (Z) \text{ plf}\end{aligned}$$

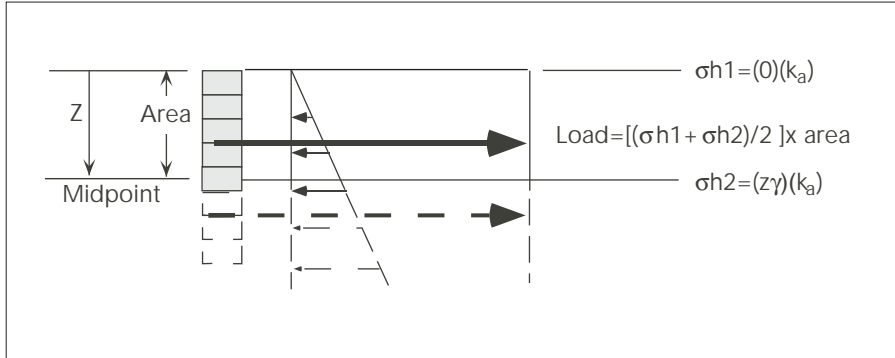
The calculated pressure is applied to the tributary area of each reinforcement level which determines the tensile load in the geogrid reinforcement. This AASHTO method calculates the internal active earth pressure coefficient based on a sloping backfill in accordance with the Rankine earth pressure formula for sloping backfill.



3H:1V SLOPING SURCHARGE AASHTO METHODOLOGIES

11a) Maximum Grid Tension (AASHTO 96 Method)

The calculated grid tensions (plf) are tabulated below:



Tensor UXK1400 = UXK1400 has an allowable design capacity of 1017 plf from the first page which is greater than the calculated tension value at each level.

| | GRID | DEPTH | z | σ_{ah} | Ave | Area | Load |
|----|---------|--------|-------|---------------|-----|------|------|
| | | TOP | 0.0 | 0.0 | | | |
| 5) | UXK1400 | 1.33' | | | 44 | 2.33 | 103 |
| | | MID | 2.33 | 87 | | | |
| 4) | UXK1400 | 3.33' | | | 125 | 2.00 | 250 |
| | | MID | 4.33 | 162 | | | |
| 3) | UXK1400 | 5.33' | | | 199 | 2.00 | 398 |
| | | MID | 6.33 | 236 | | | |
| 2) | UXK1400 | 7.33' | | | 274 | 2.00 | 548 |
| | | MID | 8.33 | 311 | | | |
| 1) | UXK1400 | 9.33' | | | 342 | 1.67 | 571 |
| | | BOTTOM | 10.00 | 373 | | | |

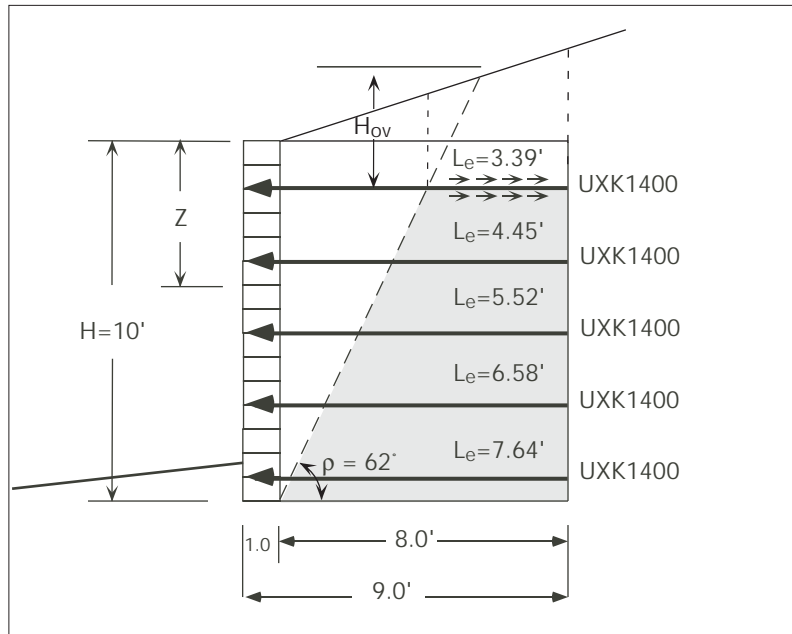
Therefore, Tensor UXK1400 is OK for all five levels in tension.



3H:1V SLOPING SURCHARGE AASHTO METHODOLOGIES

12) Pullout Resistance

Pullout safety factors are determined on a level by level basis. The effective lengths and calculated pullout is determined at each level and compared to a safety factor of 1.5.



Check each grid level for available pullout resistance against previously calculated tensile loads.

Pullout Resistance = $(\gamma H_{ov}) (2L_e) (\tan(\phi) C_i)$ with H_{ov} = average height of over burden.

$$L_e = (L - \text{Height} / \tan \rho)$$

$$H_{ov} = z + \left(\frac{\text{Height}}{\tan \rho} + 0.5 L_e \right) \tan \beta$$

| | Height | Grid | z | H _{ov} | γ | L _e | Tan34 | C _i | Pullout Load | | FS _{po} |
|----|--------|---------|------|-----------------|-----|----------------|-------|----------------|--------------|-----|------------------|
| 5) | 8.67' | UXK1400 | 1.33 | 3.43 | 120 | 3.39 | .674 | 0.90 | 1693 | 103 | 16.44 |
| 4) | 6.67' | UXK1400 | 3.33 | 5.25 | 120 | 4.45 | .674 | 0.90 | 3401 | 250 | 13.60 |
| 3) | 4.67' | UXK1400 | 5.33 | 7.07 | 120 | 5.52 | .674 | 0.90 | 5682 | 398 | 14.28 |
| 2) | 2.67' | UXK1400 | 7.33 | 8.90 | 120 | 6.58 | .674 | 0.90 | 8526 | 548 | 15.56 |
| 1) | 0.67' | UXK1400 | 9.33 | 10.71 | 120 | 7.64 | .674 | 0.90 | 11912 | 571 | 20.86 |

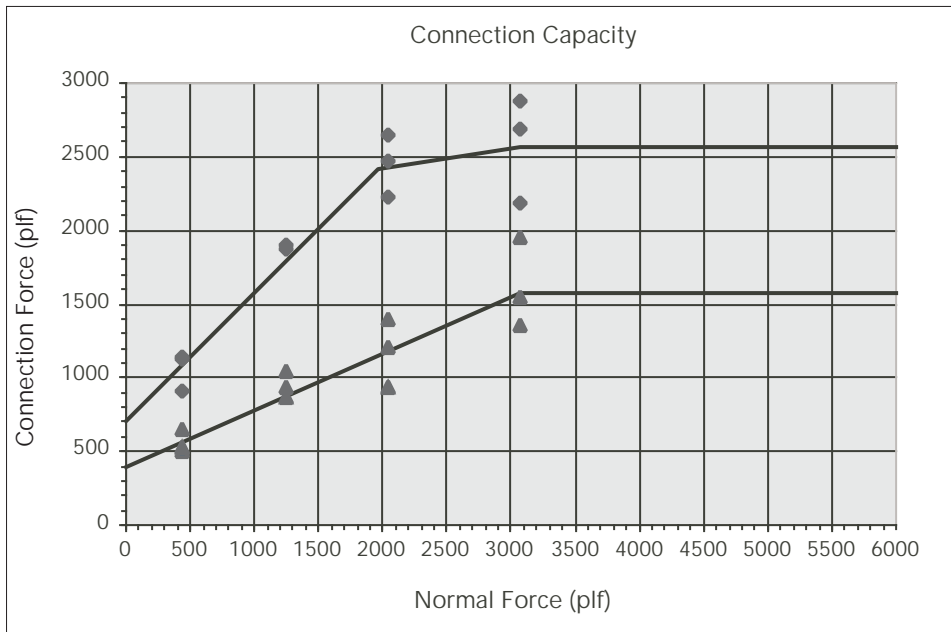
OK - All pullout safety factors are greater than 1.5.



3H:1V SLOPING SURCHARGE AASHTO METHODOLOGIES

13) Connection Strength

The last major item to check is the geogrid connection strength. KeyWall incorporates the laboratory connection test data for all Keystone unit types connected to different geogrid types. The following chart is applicable for Compac II units and Tensar UXK1400 geogrid in this example. The equations for these connection curves are:



Peak Connection = $1178 \text{ plf} + N \tan 25.1^\circ$ up to $N = 1878 \text{ plf}$
 $N > 1878 \text{ plf}$ = $1884 \text{ plf} + N \tan 5.3^\circ < 2198 \text{ plf Max} / 1.5 \text{ Factor of Safety}$
 $\frac{3}{4}" \text{ Serviceability}$ = $775 \text{ plf} + N \tan 8.9^\circ < 1306 \text{ plf Max}$

| | Height | Grid | Depth | N | T _{peak} | T _{serv} | Load |
|----|--------|---------|-------|------|-------------------|-------------------|------|
| 5) | 8.67' | UXK1400 | 1.33' | 160 | 835 | 800 | 103 |
| 4) | 6.67' | UXK1400 | 3.33' | 400 | 910 | 838 | 250 |
| 3) | 4.67' | UXK1400 | 5.33' | 640 | 985 | 875 | 398 |
| 2) | 2.67' | UXK1400 | 7.33' | 880 | 1060 | 913 | 548 |
| 1) | 0.67' | UXK1400 | 9.33' | 1120 | 1135 | 950 | 571 |

OK - Calculated loads are less than the maximum allowable for Peak and Serviceability connection criteria.



3H:1V SLOPING SURCHARGE AASHTO METHODOLOGIES

14) Other Design Checks

The KeyWall program also checks the spacing between geogrid levels and the cantilever at the top of wall against the stability of the facing units. Keystone Compac II units are typically spaced no greater than 3 blocks between geogrid levels to remain stable during construction and eliminate concerns over local stability. The cantilever at the top of wall is also checked against the final loading condition as a small gravity wall. By inspection, the two unit vertical cantilever is ok with the 3:1 sloping surcharge and the three block maximum spacing between geogrids will be stable during construction and in the final design condition.

Summary

The hand calculations verify the attached computer output. The data and methods conform to the AASHTO design methods as outlined in this manual.

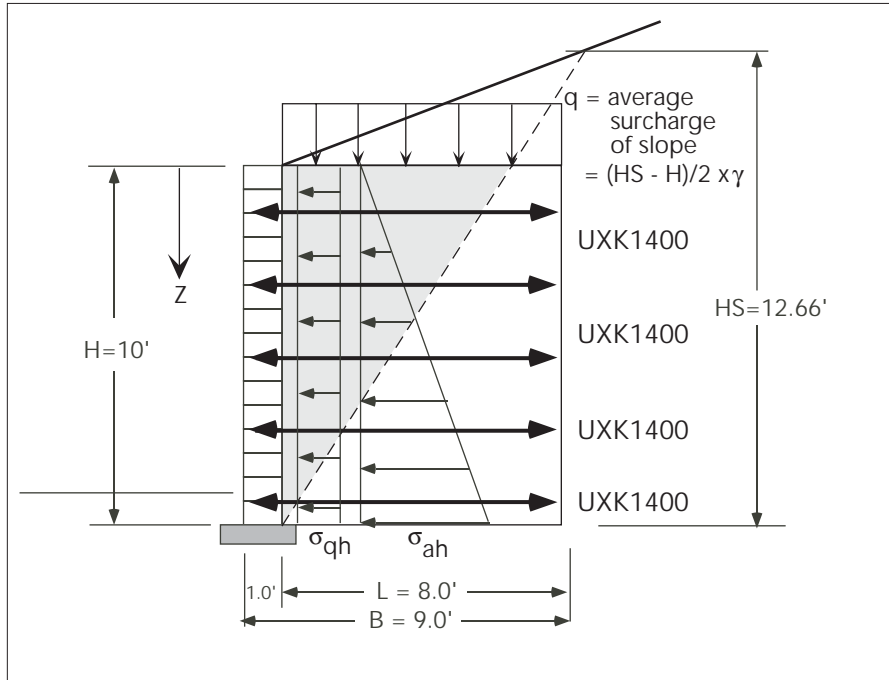
The internal stress analysis for a sloping backfill was calculated using a sloping backfill earth pressure coefficient similar to the Rankine method.

In the next section, the AASHTO Simplified method is used to recalculate the internal stresses and contrast the results. This method uses a level earth pressure coefficient and adds the sloping fill as an equivalent uniform surcharge. All other items of the design remain the same.



3H:1V SLOPING SURCHARGE AASHTO METHODOLOGIES

Internal Stability - The internal analysis must look at the maximum loads at each grid level, connection strength, pullout resistance, and local stability concerns:



The internal earth pressure at any level is calculated as follows:

$$\begin{aligned}\sigma_{ah} &= \gamma Z k_a \\ &= (120\text{pcf})(Z)(0.283) \\ &= 34.0 (Z) \text{ plf} \\ \sigma_{qh} &= qk_a \\ &= (12.66' - 10')/2 \times 120 \text{ pcf} (0.283) \\ &= 45.2 \text{ plf}\end{aligned}$$

The calculated pressure is applied to the tributary area of each reinforcement level which determines the tensile load in the geogrid reinforcement.

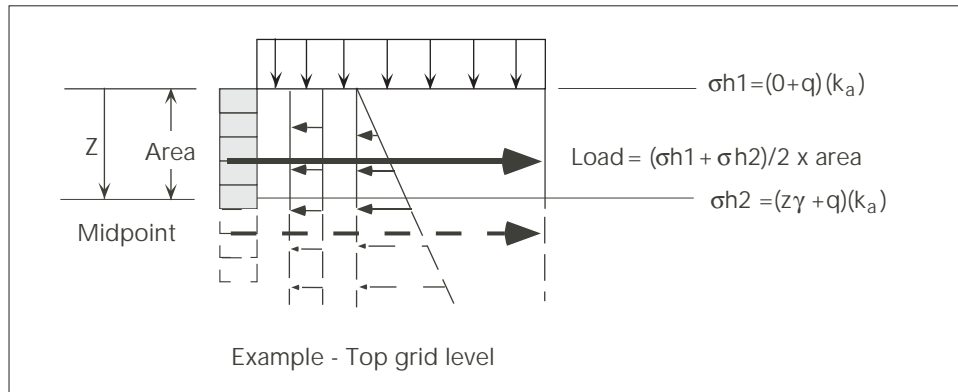
The AASHTO Simplified method calculates the internal active earth pressure coefficient based on a level backfill in accordance with the Rankine earth pressure formula and applies the slope as an average surcharge on a level backfill.



3H:1V SLOPING SURCHARGE AASHTO METHODOLOGIES

11b) Maximum Grid Tension (*AASHTO 96 simplified Method*)

The calculated grid tensions (plf) are tabulated below:



Tensar UXK1400 has an allowable design capacity of 1017 plf from the first page which is greater than the calculated tension value at each level.

| | GRID | DEPTH | z | σah | σqh | σtot | Ave | Area | Load |
|----|---------|--------|-------|-----|------|------|-----|------|------|
| | | TOP | 0.0 | 0.0 | 45.2 | 45 | | | |
| 5) | UXK1400 | 1.33' | | | | | 85 | 2.33 | 198 |
| | | MID | 2.33' | 79 | 45.2 | 124 | | | |
| 4) | UXK1400 | 3.33' | | | | | 158 | 2.00 | 316 |
| | | MID | 4.33' | 147 | 45.2 | 192 | | | |
| 3) | UXK1400 | 5.33' | | | | | 226 | 2.00 | 452 |
| | | MID | 6.33' | 215 | 45.2 | 260 | | | |
| 2) | UXK1400 | 7.33' | | | | | 294 | 2.00 | 588 |
| | | MID | 8.33' | 283 | 45.2 | 328 | | | |
| 1) | UXK1400 | 9.33' | | | | | 357 | 1.67 | 596 |
| | | BOTTOM | 10.00 | 340 | 45.2 | 385 | | | |

Therefore, **UXK1400** is **OK** for all five levels in tension.

The connection capacity, pullout calculations, and local stability are checked in a similar manner based on this load distribution. The “simplified” equivalent surcharge method distributes the loads differently and is weighted towards the upper wall section. A drawback of this method is that the calculated internal loads increase as the reinforcement lengths increase which is not consistent with earth pressure theory.



3H:1V SLOPING SURCHARGE AASHTO METHODOLOGIES

Summary

The hand calculations verify the attached computer output. The data and methods conform to the AASHTO design methods as outlined in the Keystone Design Manual.

The internal stress analysis for a sloping backfill was calculated two ways: 1) AASHTO 96 uses a sloping backfill earth pressure coefficient and 2) AASHTO Simplified uses a level earth pressure coefficient and adds the slope as an equivalent surcharge. All other items of the design remain the same.

Caution: AASHTO designs can be very tricky since the design code has changed almost every year since 1992. Each highway department has different state specification requirements and modifications to the various AASHTO editions. Many items are controversial and not fully addressed. Items such as battered wall design, connection strength evaluation, minimum reduction factors, barrier loadings and various other design and performance constraints must be reviewed for each project. KeyWall can not comply with all these AASHTO variations without the User properly determining and selecting the parameters and design constraints within the software for each project.



PART SIX
Appendix D

3H:1V SLOPING SURCHARGE AASHTO METHODOLOGIES, AASHTO-96



RETAINING WALL DESIGN

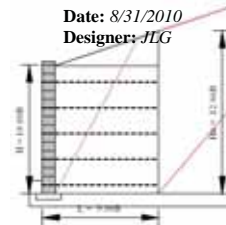
KeyWall_2010 Version 3.7.1 Build 1

Project: Part 6; Design Examples
Project No: NA
Case: Appendix D-1
Design Method: AASHTO-96 (modified soil interface)

Date: 8/31/2010
Designer: JLG

Design Parameters

| Soil Parameters: | ϕ | c psf | γ pcf |
|-----------------------|-----------------------------|-------|--------------|
| Reinforced Fill | 34 | 0 | 120 |
| Retained Zone | 30 | 0 | 120 |
| Foundation Soil | 30 | 0 | 120 |
| Reinforced Fill Type: | Sand, Silt or Clay | | |
| Unit Fill: | Crushed Stone, 1 inch minus | | |



Minimum Design Factors of Safety

| | | | | | |
|--------------|------|----------|------|-----------------|------|
| sliding: | 1.50 | pullout: | 1.50 | uncertainties: | 1.50 |
| overturning: | 2.00 | shear: | 1.50 | connection: | 1.50 |
| bearing: | 2.00 | bending: | 1.50 | Serviceability: | 1.00 |

Reinforcing Parameters: Tensar-UXK Geogrids

| | T_{ult} | R_{Fcr} | R_{Fd} | R_{Fid} | $LTDS$ | FS | T_{al} | C_i | C_{ds} |
|---------|-----------|-----------|----------|-----------|--------|------|----------|-------|----------|
| UXK1400 | 4800 | 2.60 | 1.10 | 1.10 | 1526 | 1.50 | 1017 | 0.90 | 0.90 |

Analysis: Case: Appendix D-1

AASHTO Methodology - Infinite Slope

| | | | |
|---------------|-----------------------------|-----------------------------|-----------|
| Unit Type: | CompacII / 120.00 pcf | Wall Batter: | 0.00 deg. |
| Leveling Pad: | Crushed Stone | embedment: | 1.00 ft |
| Wall Ht: | 10.00 ft | 100.00 ft long | |
| BackSlope: | 18.40 deg. slope, | DL: 0 psf uniform surcharge | |
| Surcharge: | LL: 0 psf uniform surcharge | Load Width: 100.00 ft | |
| | Load Width: 100.00 ft | | |

| Results: | <u>Sliding</u> | <u>Overturning</u> | <u>Bearing</u> | <u>Shear</u> | <u>Bending</u> |
|--------------------|----------------|--------------------|----------------|--------------|----------------|
| Factors of Safety: | 2.11 | 4.40 | 7.56 | 5.67 | 4.49 |

Calculated Bearing Pressure: 1690 / 1690 psf

Eccentricity at base: 0.57 ft

Reinforcing: (ft & lbs/ft)

| Layer | Height | Length | Calc. Tension | Reinf. Type | Allow Ten Tal | Pk Conn Tcl | Serv Conn Tsc | Pullout FS |
|-------|--------|--------|---------------|-------------|---------------|-------------|---------------|------------|
| 5 | 8.67 | 9.0 | 102 | UXK1400 | 1017 ok | 835 ok | 800 ok | >10 ok |
| 4 | 6.67 | 9.0 | 249 | UXK1400 | 1017 ok | 910 ok | 838 ok | >10 ok |
| 3 | 4.67 | 9.0 | 398 | UXK1400 | 1017 ok | 985 ok | 875 ok | >10 ok |
| 2 | 2.67 | 9.0 | 547 | UXK1400 | 1017 ok | 1060 ok | 913 ok | >10 ok |
| 1 | 0.67 | 9.0 | 570 | UXK1400 | 1017 ok | 1135 ok | 950 ok | >10 ok |

Reinforcing Quantities (no waste included):

UXK1400 5.00 sy/ft

NOTE: THESE CALCULATIONS ARE FOR PRELIMINARY DESIGN ONLY AND SHOULD NOT BE USED FOR CONSTRUCTION WITHOUT REVIEW BY A QUALIFIED ENGINEER



3H:1V SLOPING SURCHARGE AASHTO METHODOLOGIES, AASHTO SIMPLIFIED



RETAINING WALL DESIGN

KeyWall_2010 Version 3.7.1 Build 1

Project: Part 6; Design Examples

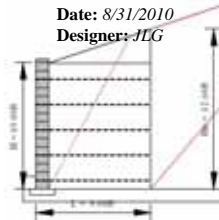
Project No: NA

Case: Appendix D-2

Design Method: AASHTO-Simplified (vertical soil interface)

Date: 8/31/2010

Designer: JLG



Design Parameters

| Soil Parameters: | ϕ | c psf | γ pcf |
|------------------------------|-----------------------------|-------|--------------|
| Reinforced Fill | 34 | 0 | 120 |
| Retained Zone | 30 | 0 | 120 |
| Foundation Soil | 30 | 0 | 120 |
| Reinforced Fill Type: | Sand, Silt or Clay | | |
| Unit Fill: | Crushed Stone, 1 inch minus | | |

Minimum Design Factors of Safety

| | | | | | |
|--------------|------|----------|------|-----------------|------|
| sliding: | 1.50 | pullout: | 1.50 | uncertainties: | 1.50 |
| overturning: | 2.00 | shear: | 1.50 | connection: | 1.50 |
| bearing: | 2.00 | bending: | 1.50 | Serviceability: | 1.00 |

Reinforcing Parameters: Tensar-UXK Geogrids

| | T_{ult} | R_{Fcr} | R_{Fd} | R_{Fid} | $LTDS$ | FS | T_{al} | C_i | C_{ds} |
|---------|-----------|-----------|----------|-----------|--------|------|----------|-------|----------|
| UXK1400 | 4800 | 2.60 | 1.10 | 1.10 | 1526 | 1.50 | 1017 | 0.90 | 0.90 |

Analysis:

Case: Appendix D-2

AASHTO Methodology - Infinite Slope

Unit Type: CompacII / 120.00 pcf

Wall Batter: 0.00 deg.

Leveling Pad: Crushed Stone

Wall Ht: 10.00 ft

embedment: 1.00 ft

BackSlope: 18.40 deg. slope,

100.00 ft long

Surcharge: LL: 0 psf uniform surcharge

DL: 0 psf uniform surcharge

Load Width: 100.00 ft

Load Width: 100.00 ft

Results:

| Factors of Safety: | <u>Sliding</u> | <u>Overturning</u> | <u>Bearing</u> | <u>Shear</u> | <u>Bending</u> |
|--------------------|----------------|--------------------|----------------|--------------|----------------|
| | 2.11 | 4.40 | 7.56 | 5.43 | 3.64 |

Calculated Bearing Pressure: 1690 / 1690 psf

Eccentricity at base: 0.57 ft

Reinforcing: (ft & lbs/ft)

| Laver | Height | Length | Calc. Tension | Reinf. Type | Allow Ten Tal | Pk Conn Tcl | Serv Conn Tsc | Pullout FS |
|-------|--------|--------|---------------|-------------|---------------|-------------|---------------|------------|
| 5 | 8.67 | 9.0 | 198 | UXK1400 | 1017 ok | 835 ok | 800 ok | 8.58 ok |
| 4 | 6.67 | 9.0 | 316 | UXK1400 | 1017 ok | 910 ok | 838 ok | >10 ok |
| 3 | 4.67 | 9.0 | 452 | UXK1400 | 1017 ok | 985 ok | 875 ok | >10 ok |
| 2 | 2.67 | 9.0 | 588 | UXK1400 | 1017 ok | 1060 ok | 913 ok | >10 ok |
| 1 | 0.67 | 9.0 | 594 | UXK1400 | 1017 ok | 1135 ok | 950 ok | >10 ok |

Reinforcing Quantities (no waste included):

UXK1400 5.00 sy/ft

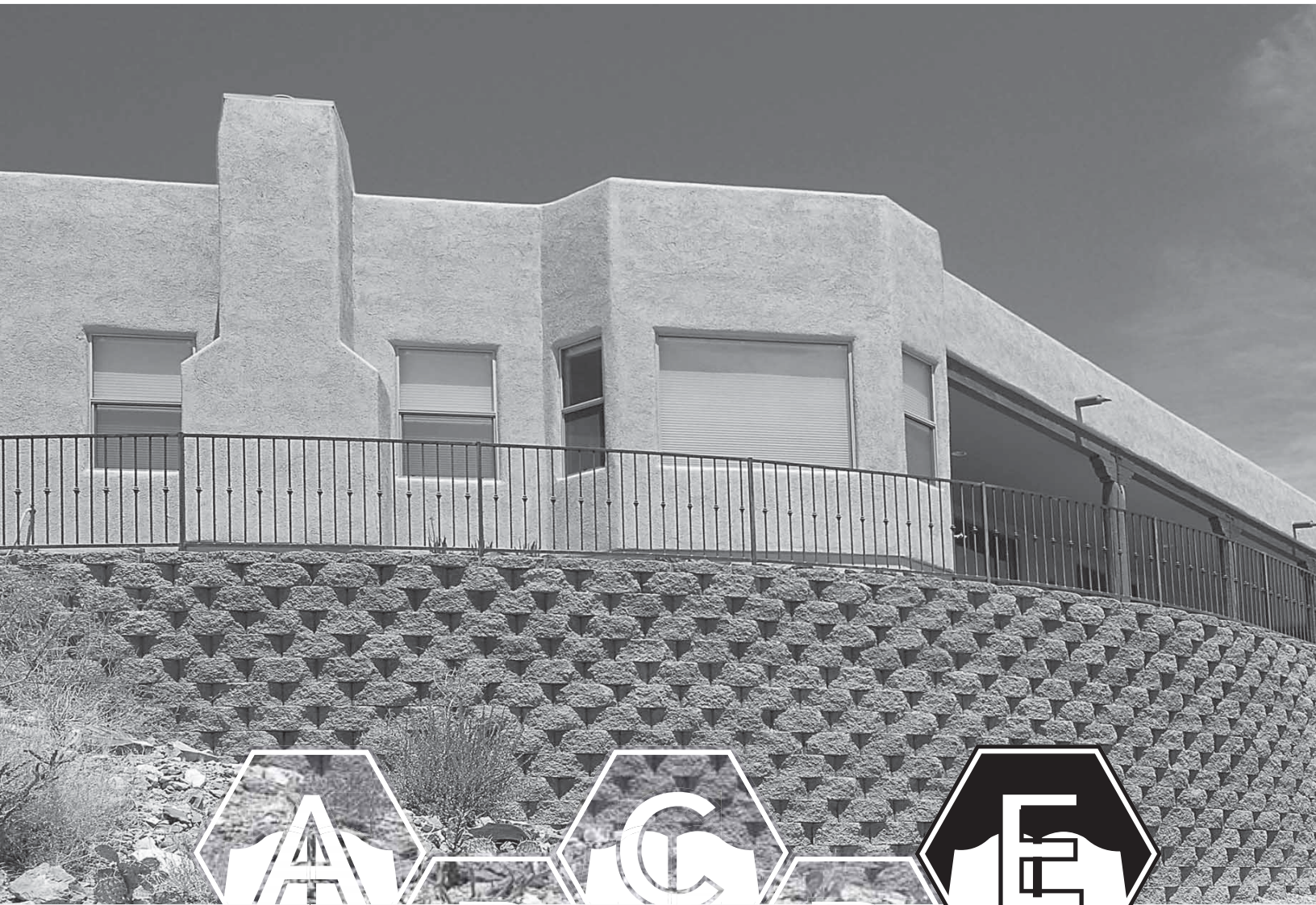
NOTE: THESE CALCULATIONS ARE FOR PRELIMINARY DESIGN ONLY AND SHOULD NOT BE USED FOR CONSTRUCTION WITHOUT REVIEW BY A QUALIFIED ENGINEER

PART SIX



APPENDIX E

Paige Residence, Tucson, Arizona; Keystone Compac





AASHTO LRFD METHODOLOGY LEVEL SURCHARGE - 250 PSF

This set of calculations is intended to verify the KeyWall program output of a typical reinforced soil wall design section. The design follows the AASHTO LRFD procedure outlined previously in the Keystone Design Manual. AASHTO LRFD follows the simplified method. The pertinent design information is summarized below:

1) General Design Data

Keystone Compac II Units (120 pcf with drainage fill and $W_u = 1.00'$)
 Mirafi 3XT Polyester Geogrid
 Wall Batter(ι) = 0° , near-vertical orientation
 Design Height = 10' (9' exposed + 1' embedment)
 Base Length, B = 9' (uniform lengths chosen for simplicity)
 Backslope, $\beta = 0$, level backslope
 Surcharge = 250 psf (typical roadway surcharge)

2) Soil Parameters (*degrees, psf, pcf*)

| SOIL PARAMETERS | ϕ | C | γ |
|-----------------|--------|---|----------|
| Reinforced Soil | 34 | 0 | 120 |
| Retained Soil | 30 | 0 | 120 |
| Foundation Soil | 30 | 0 | 120 |

3) Geogrid Design Parameters (*plf*)

| Geogrid | T_{ult} | RF_{cr} | RF_d | RF_{id} | LTDS | ϕ_{GEO} | T_{al} |
|------------|-----------|-----------|--------|-----------|------|--------------|----------------|
| Mirafi 3XT | 3500 | 1.58 | 1.10 | 1.10 | 1831 | 0.90 | 1648plf |

C_i & $C_d = 0.90$ for select backfill

4) Load and Resistance Factors - Strength I

| Driving Load Factors | Resisting Load Factors |
|----------------------|------------------------|
| $EH_d = 1.50$ | $EH_r = 0.90$ |
| $EV_d = 1.35$ | $EV_r = 1.00$ |
| $ES_d = 1.50$ | $ES_r = 0.75$ |
| $LL_d = 1.75$ | |

AASHTO LRFD Load Factors

| Resistance Factors |
|--------------------------|
| Sliding $RF_{sl} = 1.00$ |
| Bearing $RF_b = 0.65$ |
| Tension $RF_t = 0.90$ |
| Pullout $RF_{po} = 0.90$ |

AASHTO LRFD Resistance Factors



AASHTO LRFD METHODOLOGY LEVEL SURCHARGE - 250 PSF

5) Geometric Parameters - AASHTO LRFD

Internal

$$\begin{aligned}\phi &= 34 \text{ degrees} \\ \delta &= \beta = 0 \text{ degrees (no backslope)} \\ \alpha &= 90 \text{ degrees (90° + no batter)} \\ \beta &= 0 \text{ degrees (level)}\end{aligned}$$

External

$$\begin{aligned}\phi &= 30 \text{ degrees} \\ \delta &= \beta = 0 \text{ degrees (no backslope)} \\ \alpha &= 90 \text{ degrees (90° + no batter)} \\ \beta &= 0 \text{ degrees (level)}\end{aligned}$$

6) Rankine Earth Pressure Calculation

Internal

Equation (3g)

$$\begin{aligned}k_a &= \tan^2 (45 - \phi/2) \\ k_a &= \mathbf{0.283} \text{ for above parameters - Rankine}\end{aligned}$$

Equation (3h)

$$\begin{aligned}\rho &= 45 + \phi/2 \\ \rho &= \mathbf{62.0^\circ} \text{ for above parameters - Rankine}\end{aligned}$$

External

Equation (3g)

$$\begin{aligned}k_a &= \tan^2 (45 - \phi/2) \\ k_a &= \mathbf{0.333} \text{ for above parameters - Rankine}\end{aligned}$$

External Forces

Equation. (3e)

$$\begin{aligned}P_a &= \frac{1}{2} \gamma H^2 k_a \\ P_{ah} &= \frac{1}{2} \gamma H^2 k_a \cos(\beta) - \text{Horizontal Component} \\ P_{ah} &= (0.5)(120 \text{ pcf})(10')^2 (0.333) \cos(0) \\ P_{ah} &= \mathbf{1998 \text{ lbs/lf}}\end{aligned}$$

Equation. (3f)

$$\begin{aligned}P_q &= q H k_a \\ P_{qh} &= q H k_a \cos(\beta) - \text{Horizontal Component} \\ P_{qh} &= (250 \text{ psf})(10')(0.333) \cos(0) \\ P_{qh} &= \mathbf{833 \text{ lbs/lf}}\end{aligned}$$

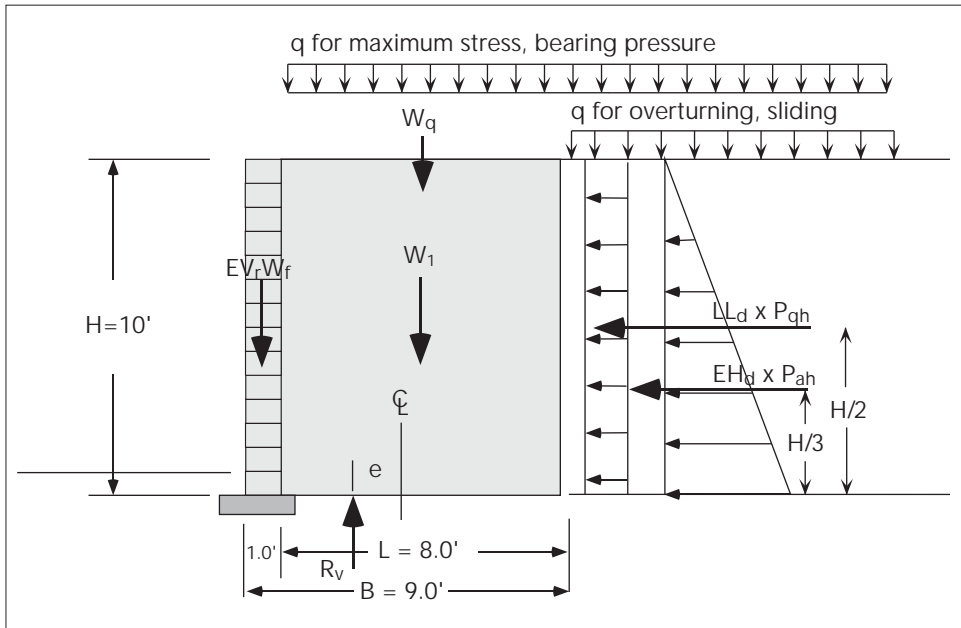
External Masses

$$\begin{aligned}W_f &= W_u H \gamma = (1.00')(10')(120 \text{ pcf}) = \mathbf{1200 \text{ lbs/lf}} \\ W_1 &= (B - W_u) H \gamma = (9' - 1.0')(10')(120 \text{ pcf}) = \mathbf{9600 \text{ lbs/lf}} \\ W_q &= q (B - W_u) = (250 \text{ psf})(9' - 1.0') = \mathbf{2000 \text{ lbs/lf}}\end{aligned}$$



AASHTO LRFD METHODOLOGY LEVEL SURCHARGE - 250 PSF

External Stability Diagram



7) Overturning

Overturning Moment

$$\begin{aligned}
 M_o &= EH_d \times P_{ah} (H/3) + LL_d \times P_{qh} (H/2) \\
 &= 1.50 \times 1998 \text{ lbs} (10'/3) + 1.75 \times 833 \text{ lbs} (10'/2) \\
 &= \mathbf{17279 \text{ ft-lbs}}
 \end{aligned}$$

Resisting Moment (*live load Wq does not contribute to resisting moment*)

$$\begin{aligned}
 M_r &= EV_r \times W_f (W_u/2) + EV_r \times W_1 (W_u + L/2) \\
 &= 1.00 \times 1200 (1.00'/2) + 1.00 \times 9600 (1.0' + 8.0'/2) \\
 &= \mathbf{48600 \text{ ft-lbs}}
 \end{aligned}$$

$$CDR_{ot} = M_r / M_o = 48600 / 17279 = \mathbf{2.81 > 1.0 \text{ OK}}$$



AASHTO LRFD METHODOLOGY LEVEL SURCHARGE - 250 PSF

8) Base Sliding

Lateral Driving Forces

$$\begin{aligned} R_d &= E H_d \times P_{ah} + L L_d \times P_{qh} \\ &= 1.50 \times 1998 \text{ lbs} + 1.75 \times 833 \text{ lbs} \\ &= \mathbf{4455 \text{ lbs/ft}} \end{aligned}$$

Lateral Resisting Forces

$$\begin{aligned} R_r &= E V_r (W_f + W_1) \times \tan \phi \text{ of foundation} \\ &= 1.00(1200 + 9600) \times 0.577 \\ &= \mathbf{6232 \text{ lbs/ft}} \\ CDR_{sl} &= R F_{sl} (R_r / R_d) = 1.00(6232/4455) = \mathbf{1.40} > 1.0 \text{ OK} \end{aligned}$$

9) Sliding at Lowest Reinforcement Level

Lateral Driving Forces (*at depth of 9.33'*)

$$\begin{aligned} R_d &= E H_d \times P_{ah} + L L_d \times P_{qh} \\ &= 1.50 \times 1739 \text{ lbs} + 1.75 \times 777 \text{ lbs} \\ &= \mathbf{3968 \text{ lbs/ft}} \end{aligned}$$

Lateral Resisting Forces (*at depth of 9.33'*)

$$\begin{aligned} \tau_{unit} &= E V_r (1250 + N \tan (29)) \\ &= 1.00 (1250 \text{ plf} + (9.33' \times 1.00' \times 120 \text{ pcf}) \tan (29)) \\ &= \mathbf{1871 \text{ plf}} \\ \tau_{soil} &= E V_r (\gamma H (B - W_u)) \times \tan \phi \text{ (of reinforced material)} \times C_{ds} \\ &= 1.00 (120 \text{ pcf} \times 9.33' \times 8') \times 0.675 \times 0.90 \\ &= \mathbf{5441 \text{ lbs/ft}} \\ CDR_{sl} &= R F_{sl} (R_r / R_d) = 1.00 ((1871 + 5441)/3968) = \mathbf{1.84} > 1.0 \text{ OK} \end{aligned}$$



AASHTO LRFD METHODOLOGY LEVEL SURCHARGE - 250 PSF

10) Bearing Pressure *(Note: Live load is added for “e”)*

Eccentricity *(using the driving factors)*

Equation (3i)

$$\begin{aligned} e &= B/2 - (M_r - M_o) / R_v \\ M_r &= EV_d \times W_f (W_u/2) + EV_d \times W_1(W_u + L/2) + LL_d \times W_q(H/2) \\ &= 1.35 \times 1200 (1.00'/2) + 1.35 \times 9600(1.0' + 8.0'/2) + 1.75 \times 2000(10'/2) = 83110 \text{ ft-lbs} \\ R_v &= EV_d (W_f + W_1) + LL_d \times W_q \\ &= 1.35(1200 + 9600) + 1.75 \times 2000 = 18080 \text{ lbs/ft} \\ e &= 9.0'/2 - (83110-17279)/(18080) = 0.86' \\ &= \mathbf{0.86'} \end{aligned}$$

Applied Bearing Pressure

Equation (3j)

$$\begin{aligned} \sigma_v &= R_v / (L - 2e) \\ &= (18080) / (9.0' - 2 \times 0.86') \\ &= \mathbf{2484 \text{ lbs/sf}} \end{aligned}$$

Eccentricity *(no load factors applied)*

Equation (3i)

$$\begin{aligned} e &= B/2 - (M_r - M_o) / R_v \\ M_r &= W_f (W_u/2) + W_1(W_u + L/2) + W_q(H/2) \\ &= 1200 (1.00'/2) + 9600(1.0' + 8.0'/2) + 2000(10'/2) \\ &= 58600 \text{ ft-lbs} \\ M_o &= P_{ah} (H/3) + P_{qh} (H/2) \\ &= 1998 (10'/3) + 833 (10'/2) \\ &= 10818 \text{ ft-lbs} \\ R_v &= W_f + W_1 + W_q \\ &= 1200 + 9600 + 2000 = 12800 \text{ lbs/ft} \\ e &= 9.0'/2 - (58600-10818)/(12800) \\ &= \mathbf{0.77'} \end{aligned}$$

Applied Bearing Pressure

Equation (3j)

$$\begin{aligned} \sigma_v &= R_v / (L - 2e) \\ &= (12800) / (9.0' - 2 \times 0.77') \\ &= \mathbf{1716 \text{ lbs/sf}} \end{aligned}$$

Note:

The external analysis above is limited to simple overturning, sliding, applied bearing pressure and bearing capacity for the reinforced mass based on a level toe. No attempt has been made to evaluate the more complicated geotechnical concerns of settlement and global stability. Geotechnical site and soils evaluation is a site specific art and can not be programmed.



AASHTO LRFD METHODOLOGY LEVEL SURCHARGE - 250 PSF

11) Bearing Capacity

Equation (3k)

$$Q_{ult} = cN_c + \gamma DN_q + 0.5\gamma(B-2)N_\gamma$$

where:

$$N_c = 30.14, N_q = 18.4, N_\gamma = 22.40$$

$$B = (B-2e) = (9.0' - 2 \times 0.86) = 7.28'$$

$$D = 1.0' \text{ level embedment}$$

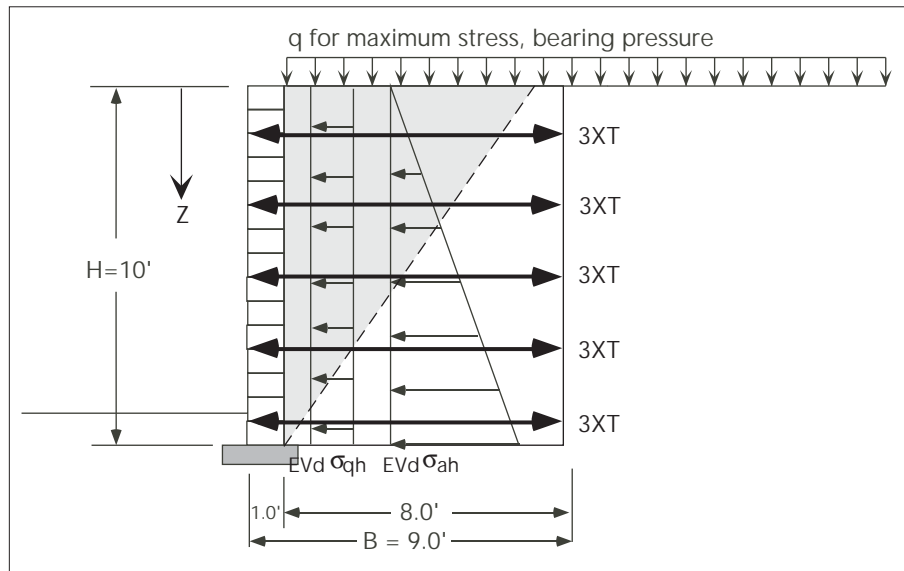
$$c = 0$$

$$Q_{ult} = 0 + (120)(1)(18.4) + (0.5)(120)(7.28)(22.40) \\ = 11992 \text{ psf}$$

$$CDR_{br} = RF_{br} (Q_{ult}/\sigma_v)$$

$$CDR_{dr} = 0.65 (11992/2484) = 3.14 > 1.0 \text{ OK}$$

Internal Stability - The internal analysis must look at the maximum loads at each grid level, connection strength, pullout resistance, and local stability concerns:



The internal earth pressure at any level is calculated as follows:

$$\begin{aligned} \sigma_{ah} &= EV_d \gamma Z k_a \cos(\beta) \\ &= (1.35)(120 \text{pcf})(Z)(0.283) \cos(0) \\ &= 45.5 (Z) \text{ plf} \\ \sigma_{qh} &= EV_d q k_a \cos(\beta) \\ &= (1.35)(250 \text{psf})(0.283) \cos(0) \\ &= 95.5 \text{ plf} \end{aligned}$$

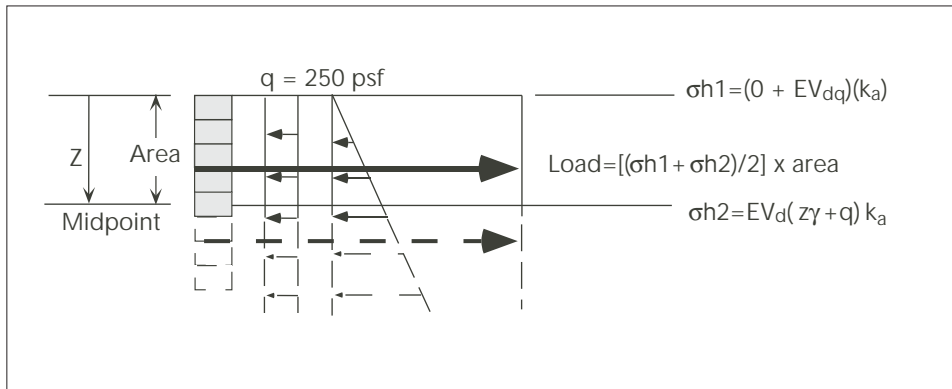
The calculated pressure is applied to the tributary area of each reinforcement level which determines the tensile load in the geogrid reinforcement.



AASHTO LRFD METHODOLOGY LEVEL SURCHARGE - 250 PSF

12) Maximum Grid Tension

The calculated grid tensions (plf) are tabulated below:



| | GRID | DEPTH | z | σ_{ah} | σ_{qh} | σ_{tot} | Ave | Area | Load |
|----|------|--------|-------|---------------|---------------|----------------|-----|------|------|
| | | TOP | 0.0 | 0.0 | 95.5 | 95.5 | | | |
| 5) | 3XT | 1.33' | | | | | 149 | 2.33 | 348 |
| | | MID | 2.33' | 106 | 95.5 | 201.5 | | | |
| 4) | 3XT | 3.33' | | | | | 248 | 2.00 | 496 |
| | | MID | 4.33' | 199 | 95.5 | 294.5 | | | |
| 3) | 3XT | 5.33' | | | | | 341 | 2.00 | 681 |
| | | MID | 6.33' | 291 | 95.5 | 386.5 | | | |
| 2) | 3XT | 7.33' | | | | | 433 | 2.00 | 866 |
| | | MID | 8.33' | 383 | 95.5 | 478.5 | | | |
| 1) | 3XT | 9.33' | | | | | 517 | 1.67 | 863 |
| | | BOTTOM | 10.00 | 460 | 95.5 | 555.5 | | | |

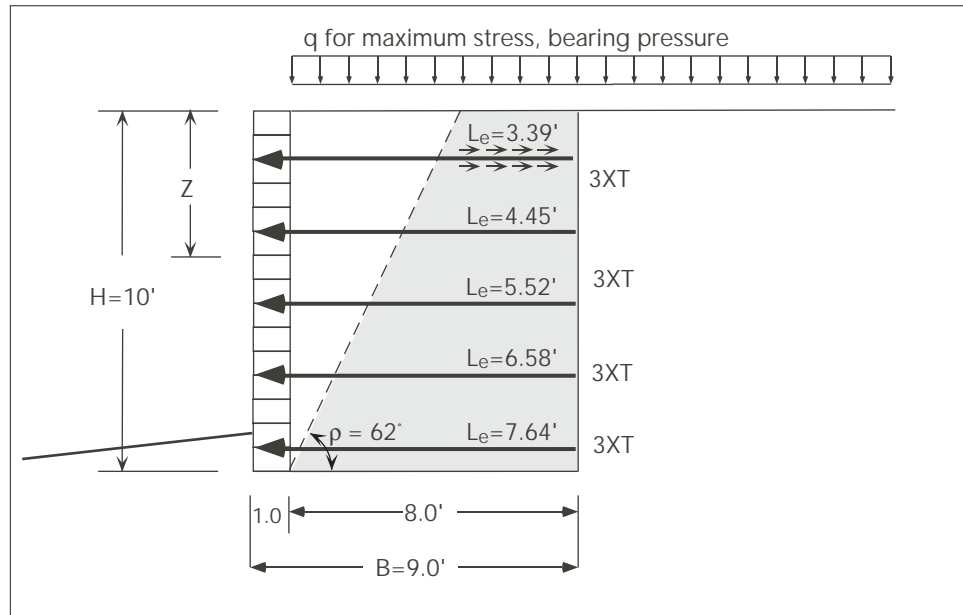
Mirafi 3XT has an allowable design capacity of 1648 plf from the first page which is greater than the calculated value at each level. Therefore, Mirafi 3XT is OK for all five levels in tension.



AASHTO LRFD METHODOLOGY LEVEL SURCHARGE - 250 PSF

13) Pullout Resistance

Pullout safety factors are determined on a level by level basis. The effective lengths and calculated pullout is determined at each level and compared to a CDR > 1.0.



Check each grid level for available pullout resistance against previously calculated tensile loads, a live load surcharge is not considered as a resisting force:

Pullout Resistance = $(\alpha)(E_v r)(\gamma H_{ov})(2L_e)(\tan(\phi)C_i)$ with H_{ov} = average height of overburden and default scale effect correction factor $\alpha = 0.80$

$L_e = (L - \text{Height}) / \tan \rho$

| | Height | Grid | α | $E_v r$ | H_{ov} | γ | L_e | $\tan 34$ | C_i | RF_{po} | Pullout/Load | | CDR_{po} |
|----|--------|------|----------|---------|----------|----------|-------|-----------|-------|-----------|--------------|-----|------------|
| 5) | 8.67' | 3XT | 0.8 | 1.00 | 1.33 | 120 | 3.39 | .674 | 0.90 | 0.90 | 525 | 348 | 1.36 |
| 4) | 6.67' | 3XT | 0.8 | 1.00 | 3.33 | 120 | 4.45 | .674 | 0.90 | 0.90 | 1726 | 496 | 3.13 |
| 3) | 4.67' | 3XT | 0.8 | 1.00 | 5.33 | 120 | 5.52 | .674 | 0.90 | 0.90 | 3407 | 681 | 4.50 |
| 2) | 2.67' | 3XT | 0.8 | 1.00 | 7.33 | 120 | 6.58 | .674 | 0.90 | 0.90 | 5617 | 866 | 5.84 |
| 1) | 0.67' | 3XT | 0.8 | 1.00 | 9.33 | 120 | 7.64 | .674 | 0.90 | 0.90 | 8302 | 863 | 8.66 |

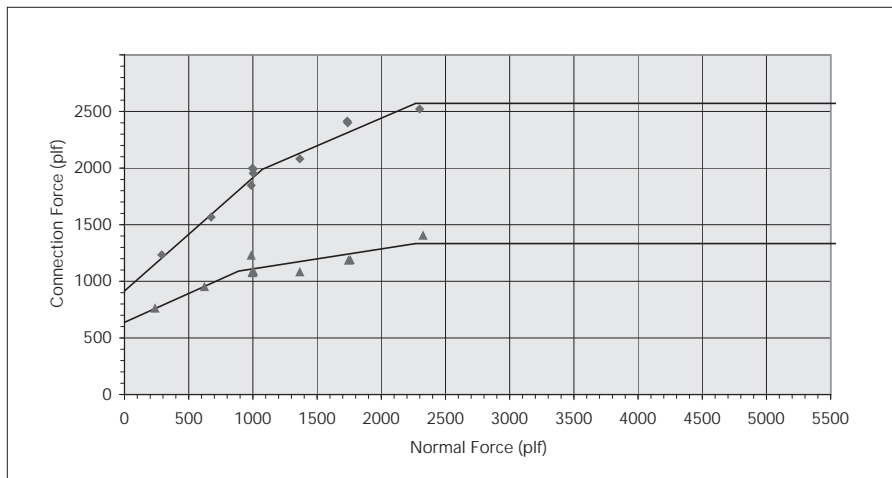
OK - All capacity demands ratios are greater than 1.0



AASHTO LRFD METHODOLOGY LEVEL SURCHARGE - 250 PSF

14) Connection Strength

The last major item to check is the geogrid connection strength. KeyWall incorporates the laboratory connection test data for all Keystone unit types connected to different geogrid types. The following chart is applicable for Compac II units and Mirafi 3XT geogrid in this example:



The equations for these connection curves are:

$$\text{Peak Connection} = 915 \text{ plf} + N \tan 45^\circ \text{ up to } N = 1074 \text{ plf}$$

$$N > 1074 = 1465 + N \tan 26^\circ < 2571 \text{ plf max}$$

| | Height | Grid | Depth | N | $[\phi_{\text{GEO}} / (RF_{\text{cn-d}} \times RF_{\text{cn-cr}})] \times T_{\text{peak}} = T_{\text{cl}}$ | Load |
|----|--------|------|-------|------|--|------|
| 5) | 8.67' | 3XT | 1.33' | 160 | 0.9 | 348 |
| 4) | 6.67' | 3XT | 3.33' | 400 | 0.9 | 496 |
| 3) | 4.67' | 3XT | 5.33' | 640 | 0.9 | 681 |
| 2) | 2.67' | 3XT | 7.33' | 880 | 0.9 | 866 |
| 1) | 0.67' | 3XT | 9.33' | 1120 | 0.9 | 863 |

OK - Calculated loads are less than the maximum allowable for Peak connection criteria.

(AASHTO LRFD method does not check serviceability as the default setting)



AASHTO LRFD METHODOLOGY LEVEL SURCHARGE - 250 PSF

15) Other Design Checks

The KeyWall program also checks the spacing between geogrid levels and the cantilever at the top of wall against the stability of the facing units. Keystone Compac II units are typically spaced no greater than 3 blocks between geogrid levels to remain stable during construction and eliminate concerns over local stability. The cantilever at the top of wall is also checked against the final loading condition as a small gravity wall. By inspection, the two unit vertical cantilever is ok with the 250 psf surcharge and the three block maximum spacing between geogrids will be stable during construction and in the final design condition.

Summary

The hand calculations verify the attached computer output. The data and methods conform to the AASHTO LRFD design method as outlined in the Keystone Design Manual.



AASHTO LRFD METHODOLOGY LEVEL SURCHARGE - 250 PSF



RETAINING WALL DESIGN

KeyWall_2010 Version 3.7.1 Build 1

Project: Part 6; Design Examples

Project No: NA

Case: Appendix E

Design Method: AASHTO-LRFD (vertical soil interface) Strength I

Date: 8/31/2010

Designer: JLG

Design Parameters

| Soil Parameters: | ϕ | c psf | γ pcf |
|------------------------------|-----------------------------|---------|--------------|
| Reinforced Fill | 34 | 0 | 120 |
| Retained Zone | 30 | 0 | 120 |
| Foundation Soil | 30 | 0 | 120 |
| Reinforced Fill Type: | Sand, Silt or Clay | | |
| Unit Fill: | Crushed Stone, 1 inch minus | | |



Load Factors for Strength I are:

(the larger values are driving (EVd), the lesser values are resisting (EVr))

| | | | |
|---|--------------|------------|------|
| EH | 1.50 to 0.90 | RF_bearing | 0.65 |
| EV | 1.35 to 1.00 | RF_pullout | 0.90 |
| ES | 1.50 to 0.75 | RF_sliding | 1.00 |
| LL | 1.75 to 0.00 | RF_tension | 0.90 |
| RFcn-d = 1.10 Applied to Peak Connection | | | |
| RFcn-cr = 1.10 Applied to Peak Connection | | | |

Reinforcing Parameters: Mirafi XT Geogrids

| | T_{ult} | RF_{cr} | RF_d | RF_{id} | $LTDS$ | T_{al} | C_i | C_{ds} | α |
|-----|-----------|-----------|--------|-----------|--------|----------|-------|----------|----------|
| 3XT | 3500 | 1.58 | 1.10 | 1.10 | 1831 | 1648 | 0.90 | 0.90 | 0.80 |

Analysis: Case: Appendix E

AASHTO LRFD

| | | | |
|----------------|-------------------------------|--------------|-------------------------|
| Unit Type: | CompacII / 120.00 pcf | Wall Batter: | 0.00 deg. |
| Leveling Pad: | Crushed Stone | embedment: | 1.00 ft |
| Wall Ht: | 10.00 ft | | |
| Level Backfill | Offset: 0.00 ft | | |
| Surcharge: | LL: 250 psf uniform surcharge | DL: | 0 psf uniform surcharge |
| | Load Width: 100.00 ft | Load Width: | 100.00 ft |

| Results: | <u>Sliding</u> | <u>Overturning</u> | <u>Bearing</u> | <u>Shear</u> | <u>Bending</u> |
|----------|----------------|--------------------|----------------|--------------|----------------|
| CDR | 1.40 | 2.81 | 3.14 | 3.74 | 0.55 < < |

Calculated Bearing Pressure: 2483 / Unfactored bearing = 1715 psf

Eccentricity at base: 0.86 ft

Reinforcing: (ft & lbs/ft)

| Layer | Height | Length | Calc. Tension | Reinf. Type | Allow Ten | Pk Conn | Serv Conn | Pullout CDR |
|-------|--------|--------|---------------|-------------|-----------|---------|-----------|-------------|
| 5 | 8.67 | 9.0 | 347 | 3XT | 1648 ok | 800 ok | N/A | 1.37 ok |
| 4 | 6.67 | 9.0 | 496 | 3XT | 1648 ok | 978 ok | N/A | 3.14 ok |
| 3 | 4.67 | 9.0 | 679 | 3XT | 1648 ok | 1157 ok | N/A | 4.54 ok |
| 2 | 2.67 | 9.0 | 863 | 3XT | 1648 ok | 1335 ok | N/A | 5.87 ok |
| 1 | 0.67 | 9.0 | 859 | 3XT | 1648 ok | 1496 ok | N/A | 8.72 ok |

Reinforcing Quantities (no waste included):

3XT 5.00 sy/ft

NOTE: THESE CALCULATIONS ARE FOR PRELIMINARY DESIGN ONLY AND SHOULD NOT BE USED FOR CONSTRUCTION WITHOUT REVIEW BY A QUALIFIED ENGINEER



Distributed By:

We reserve the right to improve our products and make changes in the specifications and design without notice. The information contained herein has been compiled by KEYSTONE and to the best of our knowledge, accurately represents the KEYSTONE product use in the applications which are illustrated. Final determination of the suitability for the use contemplated and its manner of use are the sole responsibility of the user.



Keystone Retaining Wall Systems, Inc. • A CONTECH Company
4444 West 78th Street • Minneapolis, MN 55435
(952) 897-1040 • www.keystonewalls.com

©2011 Keystone Retaining Wall Systems, Inc. Patents Pending. • SO-KWM01

