Chapter Sixty-eight

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Chapter Sixty-eight

EARTH RETAINING SYSTEMS

68-1.0 INTRODUCTION

The intent of this chapter is to inform designers, earth retaining system manufacturers, and earth retaining system suppliers of the procedures and responsibilities associated with the preparation of plans for earth retaining systems.

More detailed information and design methods are described in the FHWA publication *SA-96-038, Geotechnical Engineering Circular No. 2, Earth Retaining Systems*.

68-1.01 Consideration of an Earth Retaining System

An earth retaining system should be considered if any of the conditions exist as follows:

1. right of way is too limited for constructing side slopes and maximum use of it must be made;
2. there is a proximate live-load surcharge which must remain in place. Such surcharges may include buildings, highways, or railroads.
3. impact to any adjacent contextually sensitive areas must be lessened;
4. right-of-way costs are prohibitively high; or
5. overall project costs would be higher if an earth retaining system is not used.

The actual need for an earth retaining system should be determined during preliminary engineering with the actual type of wall (gravity, cantilever, mechanically stabilized earth, etc.) being determined during design. The determination of need for an earth retaining system should include an economic comparison between constructing the system, acquiring the required right of way, changing the roadway alignment, or otherwise avoiding system construction.

68-1.02 Relation of Earth Retaining System to Clear Zone or Obstruction-Free Zone

Making maximum use of limited right of way will often place systems within the clear zone or obstruction-free zone. Traffic barrier should be placed adjacent to earth retaining systems as shown on the INDOT *Standard Drawings*. If a system is placed outside the clear zone or obstruction-free zone, it should not require a traffic barrier treatment.

68-1.03 Relation of Earth Retaining System Backfill to Utility Lines or Drainage Structures

Ideally, utility lines or surface drainage structures should not be placed within system backfill. If these must be placed within the backfill, the designer should coordinate the system design with the Design Division's Utilities Unit or Hydraulics Unit, respectively. Drainage structures required to outlet under drains should be placed within the system.

68-1.04 Aesthetics Considerations

An earth retaining system or retaining wall is one of the key design elements. Along with the direct function of holding back earth, it provides opportunities for aesthetic enhancement of transportation systems. A retaining wall acts as a link between various highway structures and adjacent land forms. Where multiple walls exist along a corridor, repetition of a similar design will provide continuity throughout that corridor. Therefore, the designer should be aware of the total impact of retaining walls within the roadway corridor and determine how to treat them aesthetically so that they complement the surrounding environment. The designer should be conscious of the traveler's view of the wall as well as the view of those adjacent to the corridor.

The aesthetic elements surrounding any particular retaining wall are key to public acceptance of a wall project. Early in the wall design process, the designer should review any comments about the wall generated from public meetings during the preliminary design and environmental documentation process. Where possible, their comments should be considered in the design.

There is often uncertainty associated with aesthetics and there is no universally accepted theory. Simply defined, aesthetic qualities are the visual qualities that contribute to a perception of well-being and quality of life as defined by a cross-section of society.

Because of the uncertainties surrounding aesthetics, and the lack of any universally accepted theory of aesthetics, the following three-step process has been established to help the designer to fully consider the aesthetic elements of a retaining wall. These steps need to be integrated and considered together, not as discrete individual actions. Aesthetics consists of a careful blending and balancing of materials (wood, concrete, or steel), with design elements (line, form, color, and texture) and architectural elements (wall caps, parapets, fencing, etc.)

68-1.05 Aesthetics Guidelines

68-1.05(01) Step 1

Determine whether to involve a landscape architect. Consideration should be given to involving a landscape architect as follows:

1. The wall will exceed 3 m in height.

2. Extenuating circumstances are present, regardless of wall height. For example, in a rural area, the public may request special aesthetic treatments to enhance a scenic area. Other examples include, but are not limited to, historic areas, tourist areas, or other public requests.

A landscape architect can provide important information, guidance and early assistance with aesthetic considerations. Involving a landscape architect in the design process for a retaining wall will not only result in a more aesthetically pleasing design, but can also result in cost-savings. It is easier and more cost-effective to determine the real costs of a design rather than requiring expensive add-ons later.

68-1.05(02) Step 2

Determine if the wall will be placed in an urban or rural setting.

An urban setting would one generally dominated by structures with a variety of colors, textures and architectural styles. The surrounding landscape is often more orderly and manicured, and involves incorporated areas.

A rural setting is more natural, may include agricultural or forested areas, and generally involves unincorporated areas.

1. <u>Aesthetic Treatment of Wall in Urban Area</u>. In general, more attention should be given to that aesthetic treatment of a wall placed in an urban area. The high volume of users, as well as adjacent land owners, who view such a structure are increasingly demanding that it be given aesthetic treatment so as to reduce any negative visual impacts that may result. Figure 68-1A, General Aesthetic Guidelines for Retaining Wall in Urban Area, provides general guidance regarding various aspects of retaining wall design for an urban area.

2. <u>Aesthetic Treatment of Wall in Rural Area</u>. The amount of aesthetic treatment given to a wall which is placed in a rural setting is dependant upon further classification. A rural highway can be classified as either a commercial or scenic route.

   a. A commercial route generally carries high levels of commercial traffic, medium levels of commuter traffic and medium the low levels of tourist traffic. It may be either 2 lanes, or 4 lanes divided or undivided. It requires minimal aesthetic treatment.

   b. A scenic route carries high levels of tourist traffic, and medium to low levels of commercial and commuter traffic. It is highly scenic, and passes through, links or is adjacent to parks, tourist areas, recreational areas or historic areas. It may be either 2 lanes, or 4 lanes divided or undivided. High priority will be given to the recreational driving experience and aesthetic treatment.

Figure 68-1B, General Aesthetic Guidelines for Retaining Wall in Rural Area, gives general guidance regarding various aspects of retaining wall design for a rural area.

68-1.05(03) Step 3

Other factors to consider are as follows:

1. A wall should not dominate the area of effective vision of the driver.

2. The wall should be used to mount necessary light fixtures.

3. The walls should be extended to meet overpasses and bridge abutments.

4. The wall elevation should follow the natural grade of the land.

5. The ends of the wall should be tapered to meet adjacent slopes.

6. The wall should be aligned to follow adjacent landforms or as required by roadway alignment.

7. Where possible, wall alignment should be varied.

8. Backfill slopes should not exceed 2:1 for revegetation.

9. Drainage should be provided at the base of the wall.

10. Fixtures, wall finishes, patterns, line, form, color and texture should be coordinated for the transportation system within a given locality.

If a wall is to be viewed by the public or is located in a context-sensitive area, the designer should consider the use of an aesthetic treatment. Most aesthetic treatments will include the use of form liners for relief on the wall face and a coping element to finish the top of the wall. Commonly available form liners include ashlar stone, brick or block, fractured fins, fluted or vertical ribs, riverstone, textured treatment, and wood grains. The cost of using commonly available form liners adds about 10% to the cost of wall face area.

Other commercially available form liners or unique face treatments may also be considered. Color, although typically not a consideration, can be varied through the use of stains or coatings.

If an aesthetic treatment is to be included with a wall, the designer should note the type of treatment in the plans and provide any specific details necessary for construction in the special provisions.

68-2.0 PLAN PREPARATION PROCEDURES FOR EARTH RETAINING SYSTEMS

68-2.01 Wall Types

The earth retaining systems that have been approved by INDOT for inclusion into project plans are as follows:

68-2.01(01) Group 1 Systems

For each of these systems, the designer should develop a complete design and set of plan details.

1. Rigid Semi-Gravity Wall. This consists of a cast-in-place reinforced concrete cantilever wall on one row of piles.

2. Non-Gravity Cantilever Walls. These include sheet-pile walls and reinforced concrete retaining walls on one row of piles.

3. Anchored Walls. These include tied-back reinforced concrete retaining walls, and tied-back sheet piling.

4. Soil Nailed Walls.

68-2.01(02) Group 2 Systems

For these systems, the designer will make only a conceptual application. The designer will review a contractor-chosen proprietary design after the contract letting, through shop plans and computations.

1. Prefabricated Modular Gravity Walls. These include metal bin walls, concrete cellular walls, and concrete T-walls.

2. Mechanically Stabilized Earth Walls. These include wall systems with inextensible reinforcing.

3. Modular Block Walls. These include concrete block facings with or without reinforced backfill.

A modular block wall may be used only up to 2.5 m in height without soil reinforcement, or 4.5 m in height with soil reinforcement where the wall does not support a roadway or other structure. Wall height is defined as the distance from the top of the leveling pad to the top of the wall.

68-2.02 Applications

The cast-in-place reinforced concrete retaining wall will be considered the basic wall for all applications. All other wall types may be considered if they are more economical or provide unique solutions to site specific problems.

Group 1 systems are non-proprietary systems, and Group 2 systems are proprietary systems. Group 1 systems may be solely included in a project. Group 2 systems must have competitive alternatives to be included in a project.

Many earth retaining wall systems have proprietary features. Some companies provide services including design assistance, preparation of plans and specifications for the structure, supply of the manufactured wall components, and construction assistance.

68-2.03 Design Procedure

The designer should decide upon appropriate wall designs in accordance with this chapter, and provide the Design Division's project manager with documentation of these decisions prior to the geotechnical investigation. A copy of all correspondence and computations for each suggested retaining wall system should be included. A copy of these documents should be submitted with the structure size and type plans.

At the Field Check Plans submission, the designer will provide the Materials and Test Division's Geotechnical Section a set of plans showing top and bottom elevations, beginning and end stations, and stations of step locations in the bottom of the wall. The set of plans should include cross sections.

1. Responsibility. The designer will be responsible for the complete design and detailing where cast-in-place rigid, semi-gravity walls and non-gravity cantilever walls are specified.

   The designer will be responsible for the conceptual application, external stability and review of proprietary designs for all other types of earth retaining systems.

2. Methods. Earth retaining systems should be designed in accordance with the AASHTO *LRFD Bridge Design Specifications* and the INDOT *Standard Specifications*.

3. Constraints and Conditions. The primary environmental condition affecting reinforcement type selection and potential performance of earth retaining structures with reinforced backfill is the aggressiveness of the backfill material that can cause deterioration to the reinforcement.

   The lower limit to height is usually dictated by economy. Where used with a traffic barrier, a low wall on a good foundation of less than 3 to 4 m is often uneconomical, as the cost of the overturning moment leg of the traffic barrier approaches one-third of the total cost of the MSE structure in place. For a cantilever retaining wall, the barrier is simply an extension of the stem with a smaller impact on overall cost.

   The total size of structure (area of facing elements) has little impact on economy compared with other retaining wall types. However the unit cost for an MSE wall of less than 300 m^2 is likely to have a 10 to 15 percent premium.

68-2.03(01) Wall Type Selection

Other considerations in determining the acceptability of a particular retaining system should include the following:

1. geotechnical constraints;
2. future uses of the site;
3. differential deflection or settlement of wall sections;
4. project specific special features;
5. long and short term wall stability;
6. comparable degree safety;
7. accessibility to construction site;
8. staged construction limitations;
9. right-of-way limits;
10. site imposed physical limitations;
11. seismic activity;
12. wall inundation;
13. aesthetics;
14. economics;
15. environment; and
16. construction time constraints.

The decision to select any retaining wall system should involve technical feasibility and economy compared with a cast-in-place retaining wall. With respect to economy, some of the factors to be considered are as follows:

1. earthwork situation (cut or fill);
2. wall area;
3. average wall height;
4. foundation conditions;
5. availability and cost of select backfill material;
6. availability and cost of required right-of-way;
7. complex horizontal and vertical alignment changes;
8. need for temporary excavation support systems;
9. maintenance of traffic during construction; and
10. aesthetics.

The various wall systems have different performance histories, and this sometimes creates difficulty in adequate technical evaluation. Some systems are more suitable for permanent walls, others are more suitable for low walls, and some are more applicable for rural areas while others are more suited for urban areas. The selection of the most appropriate system will thus depend on the specific project requirements. See Figure 68-2A, Classification of Earth Retaining

Systems; Figure 68-2B, Fill-Section Wall System Selection Chart; and Figure 68-2C, Cut-Section Wall System Selection Chart for system section guidelines.

The Materials and Tests Division's Geotechnical Section should be informed of potential systems to be included with a project, so that it can provide site specific recommendations.

68-2.03(02) Contract Document Requirements

1. Final Plans and Design Requirements. Plans for conventional cast-in-place reinforced concrete retaining walls and permanent sheet pile walls should be fully detailed to include, but not be limited to, plan view, elevation view, sections as required, reinforcement schedules, detail clarification, allowable bearing pressure and bill of materials.

   All other earth retaining systems details should contain the project-specific information as follows:

   a. Beginning and ending wall stations;

   b. Elevation of top of wall at beginning and end of wall at 15 m intervals, all profile break points, and roadways profile data at wall line;

   c. Original and proposed ground profiles in front of and behind the retaining wall;

   d. Cross sections at retaining wall locations showing limits of excavation and backfill;

   e. Horizontal wall alignment;

   f. Details of wall appurtenances such as traffic barriers, copings, and drainage outlets;

   g. The locations and figurations of signs and lighting including conduit locations;

   h. Right-of-way limits;

   i. Construction sequence requirements including traffic control, access, and stage construction sequences;

j. Elevation of highest permissible level for foundation construction. Location, depth and extent of all unsuitable material to be removed and replaced;

k. Quantities table showing estimated wall area and quantities of appurtenances and traffic barriers;

l. Elevations of bearing pads, location of bridge seats, skew angle, and all horizontal and vertical survey control data at abutments including clearances and details of abutments;

m. Extreme high water and normal water levels at stream locations;

n. Allowable soil bearing pressure for retaining wall with reinforced backfill;

o. Magnitude, location and direction of external loads due to bridges, overhead signs and lights, and traffic and slope surcharges.

p. Limits and requirements for drainage features beneath, behind, or through the earth retaining structure;

q. Special facing panel and module finishes or colors; and

r. Critical soil properties that do not meet the minimums set out in the INDOT *Standard Specifications*.

The plans should be sealed and signed by a professional engineer registered in the State of Indiana. Such engineer will be responsible for the complete design of conventional cast-in-place concrete retaining walls and permanent sheet pile walls and for the conceptual application and location of all other earth retaining systems.

The feasibility of using an MSE wall, reinforced soil slope, or any other type of earth retention system depends on the existing topography subsurface conditions and soil/rock properties. It is necessary to perform a comprehensive subsurface exploration program to evaluate site stability, settlement potential, need for drainage etc., before repairing a slope or designing a new concrete retaining wall, MSE wall system, or other type of earth retaining system.

The designer should calculate the maximum applied bearing pressure and compare it to the allowable soil bearing pressure recommended in the geotechnical report. If the recommended allowable soil bearing pressure is less than the maximum applied bearing pressure, the designer should contact the Materials and Tests Division’s geotechnical engineer for additional guidance.

The limits for establishing pay quantities for each wall system group should be as shown in Figure 68-2D.

2. Special Provisions. Special provisions should be provided for earth retaining systems not included in the INDOT *Standard Specifications* or the recurring special provisions.

68-3.0 CAST-IN-PLACE REINFORCED CONCRETE CANTILEVER WALL

68-3.01 Foundation Information

A cantilever wall consists of a base slab or footing from which a vertical wall or stem extends upward. Reinforcement is provided in both members to supply resistance to bending. A cantilever wall can be founded on spread footings or on piles. Pertinent soils information on loading conditions, foundation considerations, consolidation potential, and external stability is included in the geotechnical report.

Normal installation of structure backfill material behind a cantilever wall should be with 1:1 backfill slopes. If site restrictions do not allow for the use of 1:1 structure backfill slopes, the designer should submit a memorandum to the Materials and Tests Division's geotechnical engineer requesting soil properties at the site. The memorandum should be submitted at the preliminary field check stage if possible. It the designer is not using 1:1 backfill slopes, then more vigorous design methods should be used.

For a wall on a spread footing, the resultant of the soil pressure distribution should be located within the middle one-third of the footing.

68-3.02 Design Procedure

The wall should be designed in accordance with the AASHTO *LRFD Bridge Design Specifications*.

Unfactored dead loads and live loads are used to determine the Factor of Safety against sliding and rotation.

Concrete design is based on the *LRFD Bridge Design Specifications* using the AASHTO load factors of 1.30 for vertical earth pressure, 1.69 for lateral earth pressure, and 2.17 for lateral earth pressure from live load surcharge. Concrete strength should be 21 MPa for footings and 24 MPa for stems. Reinforcing steel yield strength shall be 420 MPa.

68-3.02(01) Overturning

The minimum overturning Factor of Safety required is 2.0. When calculating overturning, moments are taken about the toe of the spread footing and about the centerline of the front line of piles for the pile footing. The vertical effect of surcharge acting above the footing should not be included when considering overturning.

68-3.02(02) Sliding

The minimum sliding Factor of Safety required is 1.5 for a spread footing and 1.0 for a pile footing. The low safety factor for a pile footing is used because the lateral resistance of piles is based on lateral deflections which are less than what can be tolerated by a retaining wall. The designer should verify that the lateral forces are adequately resisted by the piling.

Factors resisting sliding for a spread footing on soil include the following.

1. Passive Earth Pressure, P_p, in Front of Shear Key. For values of soil properties needed to determine passive earth pressure, contact the Materials and Tests Division's geotechnical engineer.

2. Friction Coefficient Between Soil and Concrete, μ_2. For clay, this value should be taken as 0.30 to 0.35. For sand, this value should be taken as 0.35 to 0.45.

For non-cohesive soils, friction between soil and soil in front of shear key is used. Multiply vertical load times the friction factor. For cohesive soils, use the soil cohesion, *c*, times the area of the soil in front of the shear key. See Figure 68-3A, Factor of Safety Against Sliding for Spread Footing, for an example of a wall with a shear key.

For cohesive soil properties M_1, M_2, and soil cohesion and adhesion, contact the Materials and Tests Division's geotechnical engineer.

Factors resisting sliding for a pile footing include the following:

1. Horizontal component of battered piles. Maximum batter is 4V:1H.

2. Lateral resistance of battered or vertical piles in addition to horizontal component of battered piles. Recommended values of lateral resistance of piles may be obtained from the Materials and Tests Division's geotechnical engineer.

Soil friction under the footing should not be used, as consolidation of the soil may eliminate contact between the soil and footing.

For a spread footing on rock, the footing should be embedded into the rock a minimum of 150 mm.

68-3.02(03) Stem Design

The criteria to be used when designing the stem are as follows:

1. For stem height from 5 to 8 m inclusive, the back face is battered 12V:1H. The designer has the option to batter the rear faces depending on the site requirements.

2. The minimum stem thickness is 300 mm for a stem with a constant thickness. The minimum stem thickness at the top is 250 mm for a stem that is battered. Stem thickness at the bottom is based on load requirements and/or batter.

3. Stem height is determined by site conditions.

4. The stem should be located to produce the most economical footing.

5. Shear stress in the wall should be checked at the base of the stem.

6. No. 13 reinforcing bars spaced at 450 mm should be used in the front of the stem as longitudinal and vertical reinforcing for temperature reinforcement.

7. Moment should be determined at the base of the stem and where required for bar cutoffs.

8. Loads from railings or parapets on top of the wall need not be applied simultaneously with loads from earth pressure. These are dynamic loads which are resisted by the mass of the wall and passive earth pressure.

68-3.02(04) Footing Design

The criteria to be used when designing the footing are as follows:

1. Minimum footing thicknesses are 450 mm for a spread footing and 750 mm for a pile footing.

2. The bottom of the footing should be placed at a minimum of 0.9 m below the finished ground line. If the finished ground is on a grade, the bottom of the footing may be sloped to a maximum grade of 5 percent. If the grade exceeds 5 percent, the footing should be placed level and steps should be used.

3. Maximum pile spacing in any row is 3 m.

4. Maximum pile batter is 4V:1H.

5. Piles should be embedded 300 mm into footing. Reinforcing steel should be placed on top of piles.

6. For a spread footing, reinforcing steel should be placed with 100 mm clear from the bottom of the footing. The edge clear distance should be 50 mm.

7. The footing moment should be determined at the face of the stem based on vertical loads and resultant soil pressure. No reinforcing steel is provided if the required area is less that 10 mm^2/m.

8. A design for heel moment without considering the upward soil or pile reaction is not required unless such a condition actually exists.

9. For the toe, shear should be determined at a distance from the face of the stem equal to the effective *d* distance of the footing. For the heel, shear should be determined at face of stem.

68-3.02(05) Shear Key Design

The criteria to be used when designing the shear key are as follows:

1. The key should be placed in line with the stem except under severe loading conditions.

2. The key width should ordinarily be 300 mm. The minimum key depth is 300 mm.

3. The key in should be placed in unformed excavation against undisturbed material.

4. The key should be analyzed for the forces shown in Figure 68-3B, Factor of Safety Against Sliding for Spread Footing -- Example.

5. The shape of a shear key in rock is determined by the site conditions.

68-3.02(07) Design Procedure Steps

1. Determine the tentative size of the wall.

2. Determine the magnitude of all forces acting on the wall.

3. Determine the stability of the wall against sliding and overturning.

4. Determine maximum foundation pressure.

5. Reproportion the wall if necessary and begin at Step 2 again.

6. Design the reinforcing steel for stem, toe, and heel.

7. Reproportion the wall if necessary and begin at Step 2 again.

68-3.02(08) Miscellaneous Design Information

If a wall is adjacent to a traveled roadway or sidewalk, pipe drains should be placed in back of the wall instead of weep holes. A 150 mm pipe underdrain should be used, with the flow line at the bottom of a 600 mm by 600 mm square course of fine aggregate. This system should be discharged into a storm system sewer or ditch. For rehabilitation of an existing retaining wall, plan details should be developed to replace inadequate pipe underdrain systems. A minimum slope of 0.5% should be used for pipe underdrains.

Construction joints in the footing should be offset a minimum of 300 mm from the wall joints. Reinforcing steel should be placed through the footing joints. Expansion joints should be placed in the stem at 30 m maximum spacing.

The lower limit of granular backfill is to the bottom of the footing.

68-4.0 REATAINING WALLS WITH GROUND REINFORCING

68-4.01 Mechanically Stabilized Earth (MSE) Wall

68-4.01(01) Applications

MSE walls are cost-effective alternatives for many applications where reinforced concrete or gravity type walls have traditionally been used to retain soil. These include bridge abutments and wingwalls as well as areas where the right-of-way is restricted, such that an embankment of cut-backslope with stable side slopes cannot be constructed. They are particularly suited to economical construction in steep-sided terrain, in ground subject to slope instability, or in areas where foundation soils are poor. MSE walls are not suitable for some applications as listed in the AASHTO *LRFD Bridge Design Specifications*.

Some additional successful uses of MSE walls include the following:

1. temporary structures which have been especially cost-effective for temporary detours necessary for highway reconstruction projects; and

2. phased construction.

The relatively small quantities of manufactured materials required, rapid construction, and competition among the developers of different proprietary systems has resulted in a cost reduction relative to traditional types of retaining walls. An MSE wall is likely to be more economical than another wall system for a wall higher than about 3 m or where special foundations would be required for a conventional wall.

One of the greatest advantages of an MSE wall is its flexibility and capability to absorb deformations due to poor subsoil conditions in the foundations. Also, based on observations in seismically active zones, this type of structure has demonstrated a higher resistance to seismic loading than has a cast-in-place concrete structure.

Precast concrete facing elements can be made with various shapes and textures, with little extra cost, for aesthetic considerations. Masonry units, timber, and gabions also can be used with advantage to blend into the environment.

68-4.01(02) Advantages and Disadvantages

1. Advantages.

   a. Uses simple and rapid construction procedures and does not require large construction equipment;

   b. does not require experienced craftsmen with special skills for construction;

   c. Requires less site preparation than another alternative;

   d. needs less space in front of the structure for construction operations;

   e. reduces right-of-way acquisition;

   f. does not need rigid, unyielding foundation support because an MSE structure is tolerant to deformations;

   g. is cost-effective; and

   h. is technically feasible to a height in excess of 25 m.

2. Disadvantages.

   a. Requires a relatively large space behind the wall or outward face to obtain enough wall width for internal and external stability;

   b. requires select granular fill. At a site where there is a lack of granular soils, the cost of importing suitable fill material may render the system uneconomical;

   c. suitable design criteria are required to address corrosion of steel reinforcing elements and deterioration of certain types of exposed facing elements such as geosynthetics by ultraviolet rays and potential degradation of polymer reinforcement in the ground; and

   d. the design often requires a shared design responsibility between material suppliers and owners and greater input from geotechnical specialists in a domain often dominated by structural engineers.

68-4.01(03) Relative Costs

Site-specific costs of an MSE wall are a function of many factors, including cut-fill requirements, wall size and type, in-situ soil type, available backfill materials, facing finish, or temporary or permanent application. It has been found that an MSE wall with a precast concrete facing is usually less expensive than a reinforced concrete retaining wall for a height of greater than about 3 m and average foundation conditions. A modular block wall is competitive with a concrete retaining wall at a height of less than 4.5 m.

68-4.01(04) Description of MSE Wall Systems

1. Systems Differentiation. Since the expiration of the fundamental process and concrete facing panel patents obtained by the first proprietary manufacturing company for MSE wall systems and structures, the engineering community has adopted the generic term mechanically stabilized earth to describe this type of retaining wall construction.

   A system for an MSE wall structure is defined as a complete supplied package that includes design, specifications and all prefabricated materials of construction necessary for the complete construction of a soil-reinforced structure. Technical assistance during the planning and construction phase is also included.

2. Ground Reinforcement. An MSE wall system can be described by the reinforcement geometry, stress transfer mechanism, reinforcement material, and the type of facing and connections.

   a. Reinforcement Geometry. The types of reinforcement geometry that can be considered are as follows:

      (1) Linear Unidirectional. Strips, including smooth or ribbed steel strips.

      (2) Composite Unidirectional. Grids or bar mats characterized by grid spacing greater than 150 mm.

      (3) Planar Bidirectional. Continuous sheets of welded wire mesh and woven wire mesh. The mesh is characterized by element spacing of less than 150 mm.

   b. Reinforcement Material. Reinforcement material consists of metallic reinforcements, typically of mild steel. The steel is usually galvanized.

c. Reinforcement Extensibility. The two classes of extensibility are as follows:

(1) Inextensible. The deformation of the reinforcement at failure is much less than the deformability of the soil. INDOT permits only inextensible reinforcement in an MSE wall system.

(2) Extensible. The deformation of the reinforcement at failure is comparable to or even greater than the deformability of the soil.

3. Facing Systems. The types of facing elements used in the different MSE systems control their aesthetics because they are the only visible parts of the completed structure. A wide range of finishes and colors can be provided in the facing. In addition, the facing provides protection against backfill sloughing and erosion and provides in certain cases drainage paths. The type of facing influences settlement tolerances. Major facing types are as follows:

a. Segmental Precast Concrete Panels. The precast panels have a minimum thickness of 140 mm and are of a cruciform, square, rectangular, diamond, or hexagonal geometry. Temperature and tensile reinforcement are required but will vary with the size of the panel. Vertically adjacent units are usually connected with shear pins.

Concrete copings should be placed on the tops of the top panels where the wall will be visible to traffic or pedestrians. Copings should be reinforced and may be either precast or cast-in-place. Copings should not be detailed on the plans. The minimum section is shown in the INDOT *Standard Drawings*.

b. Welded Wire Grids. Wire grid can be bent up at the front of the wall to form the wall face. This type of facing is used mainly for temporary structures.

c. Gabions. Gabions, or rock-filled wire baskets, can be used as facing with reinforcing elements consisting of welded wire mesh, welded bar-mats, geogrids, geotextiles, or the double-twisted woven mesh placed between or connected to the gabion baskets.

d. Post-Construction Facing. For a wrapped faced wall, the facing, whether geotextile, geogrid, or wire mesh, can be attached after construction of the wall by shotcreting, or placing cast-in-place concrete or other materials. This approach adds cost but is advantageous where significant settlement is anticipated.

Facings using welded wire or gabions have the disadvantages of uneven surface, exposed backfill materials, more tendency for erosion of the retained soil, possible shorter life from corrosion of the wires, and more susceptibility to vandalism. These disadvantages can, of course, be countered by providing shotcrete or by hanging facing panels on the exposed face and compensating for possible corrosion. The greatest advantages of such facings are low cost, ease of installation, design flexibility, good drainage (depending on the type of backfill) that provides increased stability, and possible treatment of the face for vegetative and other architectural effects. The facing can easily be adapted and well-blended with natural country environment. These facings, as well as geosynthetic wrapped facings, are especially advantageous for construction of temporary or other structures with a short-term design life.

4. Reinforced Backfill Materials. An MSE wall requires high quality backfill for durability, good drainage, constructability, and good soil reinforcement interaction which can be obtained from structure backfill.

   An MSE wall used as a bridge abutment should be backfilled as shown in Figure 68-4M, Fill Material Placement at MSE Wall Bridge Abutment.

5. Miscellaneous Construction Materials. A wall using precast concrete panels requires bearing pads in the horizontal joints that provide some compressibility and movement between panels and precludes concrete to concrete contact. These materials should be in concordance with the INDOT *Standard Specifications*.

   All joints are covered with a polypropylene geotextile strip to prevent the migration of fines from the backfill.

68-4.01(05) Establishment of Project Criteria

The designer should consider each topic area presented in this section at the preliminary design stage and determine appropriate elements and performance criteria. The process consists of the successive steps as follows:

1. Alternates to an MSE Wall. Cantilever, gravity, semigravity or counterforted concrete wall, or reinforced soil slopes are the usual alternatives to an MSE wall and abutments.

   In a cut situation, an insitu wall such as a tieback anchored wall, soil nailed wall, or nongravity cantilevered wall is often more economical. Where limited right-of-way is available, a combination of a temporary insitu wall at the back end of the reinforcement and a permanent MSE wall is often competitive.

2. Facing Considerations. The development of project-specific aesthetic criteria is principally focused in the type, size, and texture of the facing, which is the only visible feature of any MSE structure.

For a permanent application, consideration should be given to an MSE wall with precast concrete panels. It is constructed with a vertical face. The precast concrete panels can be manufactured with a variety of surface textures, colors, and geometrics.

At a more remote location, a gabion, timber faced, or vegetated MSE wall may be considered.

For a temporary wall, significant economy can be achieved with wire facings, geosynthetic wrapped facings, or wood board facing. It may be made permanent by applying shotcrete or cast-in-place concrete in a post-construction application, provided that the wall design meets the criteria for a permanent wall.

3. Performance Criteria. Performance criteria for an MSE structure with respect to design requirements are governed by design practice or codes such as contained in the AASHTO *Standard Specifications for Highway Bridges*, and the INDOT *Standard Specifications*. Performance criteria also include loads, design heights, embedment, settlement tolerances, foundation capacity, effect on adjoining structures, etc.

Recommend minimum factors of safety with respect to failure modes are as follows:

a. External Stability.

Sliding	F.S. ≥ 1.5 (MSEW); 1.3 (RSS)
Eccentricity *e*, at Base	< L/6 in soil L/4 in rock
Bearing Capacity	F.S. ≥ 2.5
Deep Seated Stability	F.S. ≥ 1.3
Seismic Stability	F.S. ≥ 75% of static F.S. (All failure modes)

b. Internal Stability. Pullout Resistance F.S. ≥ 1.5.

(1) Design Limits and Wall Height. The length and height required to meet project geometric requirements must be established to determine the type of structure and external loading configurations.

(2) Length of Reinforcement. The minimum reinforcement length is 0.7H for an MSE wall. Longer lengths may be required for a structure subject to surcharge loads.

(3) External Loads. The external loads may be surcharges required by the geometry, adjoining footing loads, line loads as from traffic, traffic impact loads, or sound barrier loads. Traffic line loads and impact loads are applicable where the traffic lane is located horizontally from the face of the wall within a distance less than one half the wall height.

(4) River Banks and Floodplain Areas. The base of the wall must be above the ordinary high water elevation. No. 8 stone should be placed behind the wall instead of structure backfill up to the Q_{100} high water elevation.

(5) Wall Embedment. The minimum embedment depth to the top of the leveling pad shall be 0.9 m, except for a structure founded on the rock at the surface, where no embedment is required.

For a wall constructed along a river or stream where the depth of scour has been reliably determined, a minimum embedment of 0.6 m below the Q_{500} scour depth is recommended.

(6) Seismic Activity. Due to their flexibility, MSE walls are quite resistant to dynamic forces developed during a seismic event. See the AASHTO *LRFD Bridge Design Specifications* for seismic design considerations.

4. <u>Consideration of effects of site on corrosion/degradation of reinforcements</u>.

5. <u>Consideration of effects of site with regard to river banks and floodplain areas</u>.

68-4.02 Modular Block Facing Units with Reinforced Backfill

A concrete modular block retaining wall is a Group 2 system. The maximum height should be 4.5 m, measured from the top of the leveling pad to the top of the wall.

A concrete modular block retaining wall is constructed from blocks which are typically available in a large variety of facial textures and colors, providing a variety of aesthetic appearances. They range in facial area from 0.05 to 0.1 m^2. An integral feature of the facing is a front batter ranging from nominal to 15 deg. The shape of the blocks usually permits the wall to be built along a curve, either concave or convex. The blocks are dry-stacked, therefore mortar or grout is not used to bond the units together, except for the top two layers.

68-4.02(01) Design Procedure

Design procedures are described for two cases as follows:

1. Case A: Horizontal backslope ($\beta = 0°$), sloping backfill ($\beta > 0°$)
2. Case B: Broken-back backfill ($\beta > 0°$)

The angle β is shown in Figure 68-4 I, External Stability Calculations, Sloping or Horizontal Backfill, ($\beta \geq 0°$)

For a wall with a setback of 0, the active earth pressure coefficient for external stability, K_a, may be determined from Equation 68-4.1 (Rankine's formula).

$$K_a = (\cos B)\frac{\cos(\beta - X)}{\cos(\beta + X)} \qquad \text{(Equation 68-4.1)}$$

where $X = \sqrt{\cos^2\beta - \cos^2\phi_r}$

For a wall with a setback of 1, K_a may be determined from Equation 68-4.2 (Coulomb's formula), with $\delta = 0$.

$$K_a = \frac{\cos^2(\phi_r + \alpha)}{\cos^2\alpha[\cos(\alpha - \delta)]\left(1 + \sqrt{Z/Y}\right)^2} \qquad \text{(Equation 68-4.2)}$$

where φ_r = angle of internal friction of the retained soil (from geotechnical report)
α = wall setback angle from vertical
δ = interface friction angle between reinforced soil zone and retained soil (use 0°)
β = backslope angle (see Figure 68-4 I)
$Z = \sin(\phi_r + \delta)\sin(\varphi_r - \beta)$
$Y = \cos(\alpha - \delta)\cos(\alpha + \beta)$

1. Analysis of Overturning. The factor of safety against overturning, FS_{OT}, should be checked as follows:

$$FS_{OT} = \frac{\Sigma\ \mathrm{Re}\mathit{sisting\ Moments}}{\Sigma\ \mathit{Overturning\ Moments}}$$

and must be ≥ 2.0. $FS_{OT} =$

$$\frac{0.5V_3(L_2 + H\tan\alpha) + V_2(H\tan\alpha + W_W + 0.67L) + H_1\sin C(L_2 + 0.5h\tan\alpha) + H_2\sin C(L_2 + 0.33h\tan\alpha)}{0.5h(H_1\cos C) + 0.33h(H_2\cos C)} \geq 2.0$$

(Equation 68-4.3)

where:

$V_3 = 0.5LW_iH$
$V_2 = 0.5LW_i(h - H)$
W_i = Unit weight of reinforced infill soil
$H_1 = hSurK_aW_r$
$C = \delta - \alpha$ [C cannot exceed β.]
$H_2 = 0.5hK_aW_r$
δ = external friction angle, the lesser of φ_i or φ_r
φ_i = internal friction angle of reinforced infill soil
φ_r = internal friction angle of retained soil
$H = H + \tan\beta[L + (L\tan\beta)(\tan\alpha)]$
$V_1 = SurW_i[L + (h - H)]\tan\alpha$
$R = V_1 + V_2 + V_3 + (H_1 + H_2)\sin C$

Note: For overturning and sliding analysis, *Sur* is assumed to act outside the reinforced soil zone, therefore V_1 is not used. V_1 is used to compute maximum bearing pressure.

2. Analysis of Sliding. The factor of safety against sliding friction, FS_{SF}, should be checked as follows:

$$FS_{SF} = \frac{\Sigma \text{ Resisting Forces}}{\Sigma \text{ Driving Forces}}$$

and must be ≥ 1.5.

$$FS_{SF} = \frac{(V_2 + V_3)\tan S}{(H_1 + H_2)\cos C} \geq 1.5$$

(Equation 68-4.4)

Where tan S is the coefficient of sliding friction from the geotechnical report.

Also, sliding should be checked at the level of the first geogrid from the bottom using the geogrid coefficient of direct sliding, but including the shear strength between modular block units. If the geogrid coefficient of direct sliding is unknown, use 0.65 tan S.

3. Analysis of Soil Bearing Pressure. The bearing pressure at the bottom of the reinforced soil mass and blocks, *BP*, is determined by using the Meyerhof stress distribution.

$$BP = \frac{R}{L_2 - 2e} \quad \text{(Equation 68-4.5)}$$

where e is determined by taking moments about the center of the base length L_2.

$$e = 0.5hH_1 \cos C + 0.33hH_2 \cos C - 0.5H_1 \sin C\left(L_2 + h\tan\alpha\right) - H_2 \sin C -$$
$$0.5V_1\left(h + H\right)\tan A - 0.5V_1W_W - V_2\left(H\tan\alpha + 0.67L + W_W - 0.5L_2\right) - \frac{2R}{V_3 H \tan\alpha}$$

$BP \leq$ Allowable bearing capacity

The allowable bearing capacity is provided by the Materials and Tests Division's Geotechnical Section.

Equations 68-4.3 and 68-4.4 for Case B are correct where the breakpoint of the slope is $\geq$ L from the back face of the wall as shown in Figure 68-4J. If the breakpoint is less than L away, these equations must be modified.

If a break in the slope behind the wall is located horizontally within a distance of $2H$, Case B may be used. If the break is located at $2H$ or greater from the wall, Case A should be used.

The only difference between Case A and Case B is the magnitude for forces H_1 and H_2. The magnitude of these forces is a function of K_a, which is shown at the beginning of the design procedure. Both cases use this formula for K_a. However, for Case B, angle I should be substituted for angle β. Where the break in the slope behind the wall is located $0.5H$ from the back face of the reinforced soil mass, live load surcharge, *Sur*, should be considered in the design. If the break is located at $0.5H$ or greater from the back face of the reinforced soil mass, *Sur* should not be considered in the design.

The failure plane for a modular block MSE wall with geogrid, or extensible, reinforcement is defined by a straight line passing through the heel on the retained-earth side of the lowermost bock at an angle ά from the horizontal. The angle ά is calculated from Equation 68-4.6.

$$\tan\left(\alpha - \phi\right) = \frac{\left\{x\left(x + y\right)\left[1 + y\tan\left(\delta - \alpha\right)\right]\right\}}{\left(x + y\right)\left[1 + \tan\left(\delta - \alpha\right)\right]} \quad \text{(Equation 68-4.6)}$$

where

$X = \tan\left(\phi_i - \beta\right)$

$Y = \cot\left(\phi_i + \alpha\right)$

φ_i = angle of internal friction of reinforced infill soil

δ = angle of friction at back of wall (assume 2/3 φ_i)
See Figures 68-4K and 68-4L for definitions of α and β.

The failure plane for Cases A and B with extensible reinforcement is based on angle α as shown above.

The horizontal stress, σ_h, at each reinforcement level for extensible reinforcement may be computed by multiplying the vertical stress, σ_v, at that level, by the active earth pressure coefficient K_a.

$$K_a = \frac{\cos^2(\phi_i + \alpha)}{\cos^2 \alpha \cos(\alpha - \delta)\left(1 + \sqrt{Z/Y}\right)^2} \qquad \text{(Equation 68-4.6)}$$

where
$Z = \sin(\phi_i + \delta)\sin(\phi_i - \beta)$
$Y = \cos(\alpha - \delta)\cos(\alpha + \beta)$
φ_i = peak angle of internal friction of the reinforced soil zone, or 34 deg
δ = interface friction angle which is assumed to be two-thirds of the angle of internal friction of the reinforced infill soil, or 22.7 deg

The vertical stress, σ_v, is based on the vertical loads being distributed over a length determined by the Meyerhoff formula. The same procedure should be applied to calculate the maximum bearing pressure at the bottom of the reinforced soil mass shown in the external stability equations. The same equations can be used, except *h* and *H* must be decreased by the distance from the top of the leveling pad to the level of the extensible reinforcement where vertical earth pressures are being calculated. If this procedure results in *R* appearing to the right of center of L_2 (see Figure 68-4 I), then calculate σ_v based on the height of overburden plus surcharge at the center of the contributing area, L_a, for the geosynthetic reinforcement being considered. The values of *d* and L_a are shown in Figure 68-4L, Case B: Horizontal Backslope.

4. Soil-Reinforcement Forces. For both extensible and inextensible reinforcements, the surcharge should be included for stress calculations. The force in the soil reinforcement is determined at the location of the failure plane as follows:

$$\begin{aligned} Re &= 0.5ZH_1\cos C + 0.33ZH_2\cos C - H_1\sin C[0.5L_2 + (H - 0.5Z)]\tan\alpha - \\ &\quad H_2\sin C[0.5L_2 + (H - 0.67Z)]\tan\alpha - V_1(0.5L_2 + H\tan\alpha - 0.5L) - V_2(H - 0.5Z)\tan\alpha \end{aligned} \qquad \text{(Equation 68-4.7)}$$

where:

$V_3 = 0.5LW_iH$
$H_1 = ZSurK_aW_r$
δ = external friction angle, the lesser of φ_i or φ_r

φ_i = internal friction angle of reinforced infill soil
φ_r = internal friction angle of retained soil
$C = \delta - \alpha$ [C cannot exceed β.]
$H_2 = 0.5ZK_aW_r$
K_a (see Equation 68-4.6)
W_r = unit weight of retained soil
L = length of soil reinforcement
$V_1 = LSurW_i$
$V_2 = ZLW_i$
W_i = Unit weight of reinforced infill soil

$$\sigma_v = \frac{R}{L-2e} \qquad \text{(Equation 68-4.8)}$$

If an alternate method is required to calculate φ_v,

$$\sigma_v = W_i(d + Sur) \qquad \text{(Equation 68-4.9)}$$

where d and its location are shown in Figure 68-4L.

For extensible reinforcement, $\sigma_h = \sigma_v K_a$, where K_a is based on Equation 68-4.6.

For extensible reinforcement, $\sigma_h = \sigma_v K_a$.

Multiplying σ_h times its contributing area will provide the force in the soil reinforcement, F_g. This is the force in the reinforcement at the failure plane. Because geogrid reinforcement is continuous, the contributing area is the vertical spacing, and the resulting force is on a per-meter basis.

The vertical forces V_1, V_2, and V_3, and horizontal forces H_1 and H_2 should be determined using calculations accompanying the stability check. However, V_3, H_1, and H_2 are based on the soil plane above the reinforcement. The procedure outlined above should be followed to find the force in the soil reinforcement.

The forces in the geogrid at the back face of the blocks and at the failure plane are assumed to be equal at the bottom of the wall. They vary linearly to a point at one-half the wall height where the force is equal to 85% of the force at the failure plane. For the upper half of the wall, the force at the back face of the blocks is assumed to equal 85% of the force at the failure plane.

The force F in the geogrid is equal to σ_h times the contributing area. Since geogrid reinforcement is continuous, the contributing height is normally used.

The connection strength between a geogrid and the blocks should be determined by National Concrete Masonry Association Test Method SRWU-1. The service state condition strength should be based on a deformation of the geogrid relative to the block, measured at the face of the blocks, or 13 mm. The connection strength used for design should be the lesser of the peak connection strength or the service state connection strength.

$$\frac{Connection\,Strength}{F} \geq 1.5$$

The allowable force, F, in the geogrid reinforcement should be in accordance with the AASHTO *Standard Specifications for Highway Bridges*. The values of limit state tensile load, T_l, and serviceability state tensile load, T_w, as described in the AASHTO *Standard Specifications for Highway Bridges*, should be determined.

A factor of safety or reduction factor for extensible reinforcement with respect to environmental and aging losses, *FD*, and a factor of safety or reduction factor for extensible reinforcement with respect to construction damage, *FC*, are required. If project-specific test results are available, *FD* should be taken as 2.0. If tests are not available, *FD* should be taken as 1.1 minimum. If project-specific test results are available, *FC* should be taken as 3.0. If tests are not available, *FC* should be taken as 1.3 minimum. In addition, an overall factor of safety, *FS*, should be taken as 1.78.

5. Pullout Capacity of Extensible Reinforcement. The pullout capacity is developed by extending the geogrid beyond the failure plane for a sufficient distance to develop a force F_U, equal to $1.5F$. The minimum length of soil reinforcement is $0.7H$, 1.8 m, or the distance to the failure plane plus 0.9 m, whichever is greater. F_U should be calculated as follows:

$$F_U = 2\sigma_v L_A f_d \tan\phi_i \qquad \text{(Equation 68-4.10)}$$

where:
σ_v = vertical stress, or $120d$ as shown in Figure 68-4 I
L_A = length of reinforcement beyond the failure plane
f_d = equivalent coefficient of direct sliding derived from pullout tests
φ_i = angle of internal friction of the reinforced-soil zone, or 34 deg

Geogrid pullout may occur as a result of a combination of soil shearing on plane surfaces parallel to the direction of geogrid movement and soil bearing on transverse geogrid surfaces perpendicular to the direction of geogrid movement. Ultimate pullout capacity should be based on a maximum elongation of the embedded geogrid of 13 mm, measured at the leading edge of the compressive zone within the soil mass.

68-4.02(02) Summary of Design Safety Factors and Requirements

1. Safety Factors.

   a. Overturning, ≥ 2.0
   b. Sliding, ≥ 1.5
   c. Pullout, ≥ 1.5
   d. Connection Strength, ≥ 1.5 at 13 mm deformation
   e. Overall, Limited State Deformation, ≥ 1.78
   f. Global, ≥ 1.3

2. Blocks Data.

   a. A block should consist of one piece.
   b. Minimum thickness of front face = 100 mm
   c. Minimum thickness of internal cavity walls other than front face = 50 mm
   d. f_c' = 35 MPa

3. Traffic Surcharge. Live load surcharge = 37.8 kN/m^2

4. Retained Soil.

   a. Unit weight = 18.9 kN/m^3
   b. Angle of internal friction, φ_i, should be determined from test information shown in the geotechnical report.

5. Design Life. Design life should be 75 years minimum.

6. Soil-Pressure Theory. Either Coulomb's or Rankine's theory should be used at the designer's discretion.

7. Soil Reinforcement.

   a. Should be either inextensible or extensible.
   b. Minimum length should be 70% of the wall height, and not less than 1.8 m.
   c. Length should be equal throughout the wall height.
   d. Maximum vertical spacing between layers = 600 mm.
   e. Should extend a minimum of 900 mm beyond the failure plane.

68-5.0 PREFABRICATED MODULAR GRAVITY WALLS

68-5.01 Modular Block Gravity Wall without Ground Reinforcing

The proprietary modular blocks used in combination with ground reinforcing (See Section 68-4.0) can also be used as a pure gravity wall. The height to which it can be constructed is a function of the width of the blocks, the setback of the blocks, the backslope angle, and the angles of internal friction of the retained soil behind the wall. The base of the block wall shall be placed at least 0.9 m below the finished grade elevation. A wall of this type is limited to heights of 1.5 m or less, and is limited to a maximum differential settlement of 1/200. However a wall of this type may not be used in a critical situation, e.g., where supporting a highway or other structure.

Dry-cast modular block wall units are relatively small, squat concrete units that have been specially designed and manufactured for retaining wall applications. The mass of these units commonly ranges from 15 to 50 kg, with units of 30 to 50 kg routinely used for highway work. Unit heights typically range from 100 to 200 mm for the various manufacturers. Exposed face length usually varies from 200 to 450 mm. Nominal width (dimension perpendicular to the wall face) of units typically ranges between 230 and 600 mm. Units may be manufactured solid or with cores. Full height cores are filled with aggregate during erection. Units are normally dry-stacked (i.e. without mortar) and in a running bond configuration. Vertical adjacent units are interconnected to prevent sliding.

The material specifications for the blocks used for a gravity wall are identical to those for the blocks used for a modular block wall with ground reinforcing.

The design of a modular block gravity wall should be in accordance with the special provisions for the project and the policy and procedures as stated herein.

The modular-block manufacturer should check the wall for overturning and internal stability and make certain that the bearing capacity requirements are satisfied. The Materials and Tests Division's geotechnical engineer will check the wall for sliding, global stability, and settlement, and provide the allowable bearing pressure and the equivalent fluid pressure acting on the back of the wall.

68-5.01(01) Design Procedure

When designing a modular block gravity wall without setback, the active earth pressure coefficient K_a may be determined from the Rankine formula.

When designing a modular block gravity wall with setback, the active earth pressure coefficient K_a may be determined from the Coulomb formula. The interface friction angle between the blocks and soil behind the blocks may be assumed to be zero.

68-5.01(02) Design Considerations

The forces acting on a modular block gravity wall are shown in Figure 68-5A, Modular Block Gravity Wall Analysis. The unit weight of the block is assumed to be 22.0 kN/m^3. The unit weight of the drainage aggregate inside or between the blocks is assumed to be 19.0 kN/m^3. No passive soil pressure is allowed to resist sliding. Sheer between the blocks must be resisted by friction, keys or pins.

1. Overturning. For overturning, moments are taken about the outside corner of the block. The vertical components of the soil pressure forces may be conservatively ignored.

   Factor of Safety for Overturning = Resisting Moment / Overturning Moment.
   Factor of Safety for Overturning must be equal to or greater than 2.0.

2. Sliding. The Factor of Safety for Sliding = Resisting Forces / Driving Forces.
   Factor of Safety for Sliding must be equal to or greater than 1.5.

3. Bearing Pressure. The maximum bearing pressure at the bottom of the lower block must be less than or equal to the allowable bearing capacity which is provided in the geotechnical report.

68-5.02 Metal Binwall

A metal modular binwall system functions as a gravity wall utilizing its own weight and the weight of the soil inside the modules to resist any overturning and sliding. It is a proprietary wall system whose design is provided by the wall supplier. A steel modular wall system should not be used where the groundwater or surface runoff is contaminated with acid or where deicing spray is anticipated. Bins usually consist of adjoining closed face cells filled with structure backfill to form a gravity type wall. The base width of a binwall usually ranges from approximately 40 to 60% of the wall's height, depending on surcharges, backslopes, batter, etc. The base of the binwall should be placed at least 0.9 m below the finished grade elevation.

68-5.02(01) Design Procedure

The AASHTO *LRFD Bridge Design Specifications* should be used when appropriate for design of a metal binwall. The *Specifications* permit 80% of the weight of the soil to be effective in resisting overturning moments. The basis of this practice is empirical and recognizes the fact that some of the soil in the modules is in direct contact with the foundation soil. A value greater than 80% is permitted if the actual value can be verified by full scale field tests or if the bins are constructed with floors.

Longitudinal differential settlements along the face of the wall should result in a slope less than 1/200. Some precast concrete modular systems are relatively rigid and are subject to structural damage due to differential settlements, especially in the longitudinal direction. Therefore, the ultimate bearing capacity for footing design may be comparable to that for a cast-in-place wall because both are relatively sensitive to differential settlements.

The *Specifications* provide an equation for determining that factored pressure inside the bin module, in addition to other design provisions. A value of 1925 kg/m^3 may be used for the soil density.

Sliding and overturning stability computations for a metal wall system should be made by assuming that the system acts as a rigid body. The lateral earth pressure force per unit width behind a prefabricated modular wall should be taken as specified by the AASHTO *LRFD Bridge Design Specifications* Equation 3.11.5.7-1, and applied at a height and in a direction as specified therein. Where the rear of the modules forms an irregular surface (stepped surface), pressures should be computed on an average plane surface drawn from the lower back heel of the lowest module as shown in the *Specifications*. The angle of friction (δ) between the back of the modules and backfill is stated in the *Specifications* for three possible cases. This angle effects the magnitude and direction of the resulting lateral earth pressure force. For overturning, moments should be taken about the toe of the lower unit. When performing an external analysis of the system, only the forces acting on or inside the pressure plane may be utilized.

The concrete for a precast concrete modular bin wall should have a minimum 28 day compressive strength of 28 MPa. Reinforcement should be Grade 420 uncoated bars or welded wire fabric. Infill soil should be structure backfill. Drainage details for the wall system shall be shown on the plans.

68-5.02(02) Summary of Design Safety Factors and Requirements

1. Safety Factors.

   a. Overturning ≥ 2.0
   b. Sliding ≥ 1.5
   c. Global ≥ 1.3

2. Foundation Design Parameters. Use values provided by the Materials and Tests Division's Geotechnical Section.

3. Concrete Design Data.

   a. f_c' = 28 MPa, or as required by design
   b. f_y = 410 MPa
   c. Use uncoated reinforcing bars or welded wire fabric

4. Traffic Live Load Surcharge. Use 0.6 m or 37.8 kN/m^2

5. Retained Soil.

   a. Unit weight = 5.75 kN/m^3
   b. Use the angle of internal friction value provided by the Materials and Tests Division's Geotechnical Section.

6. Soil Pressure Theory. Rankine's Theory or Coulomb's Theory should be used at the designer's discretion.

68-5.03 Gabion Wall

68-5.03(01) Background

A gravity retaining wall may be constructed from rock-filled wire baskets commonly called gabions or gabion baskets. The gabions are manufactured from a heavy wire mesh formed into rectangular baskets. Common basket sizes include a standard depth of 0.9 m; heights of 300, 450, or 900 mm; and lengths of 2, 3, or 4 m. Individual baskets are placed on the prepared earth surface, reinforced with internal tie wires, and filled with riprap stone. After the baskets are filled, the lids are closed and wired shut to form a relatively rigid block. Succeeding rows of gabions are laced to the filled underlying gabions and are filled in the same manner until the wall reaches the design height. Proprietary gabion basket manufacturers will supply details for the wires, lacing, and lid closure. However, the manufacturers do not provide internal or external wall design. External stability considerations are usually determined by the Materials and Tests Division's Geotechnical Section.

A gabion wall can be used for a variety of applications. A wall on a grade may be accommodated by either putting steps in the wall or by slopping the base of the wall. A gabion wall on a grade of 5% or more has a more pleasing appearance if steps are utilized. A gabion wall may be constructed adjacent to streams or lakes so that at least a portion of the wall may be below water line. For this application, it is normally necessary to dewater the wall site during construction. For a water installation, the wall should be protected against erosion or scour by the use of riprap or other suitable protection. A gabion wall may also be constructed along a curved alignment. However, a sharp curve with a radius of less than 7.5 m may be difficult to construct and should be avoided. A layer of geotextile fabric should be placed on the back side of the wall prior to backfilling to prevent soil migration and loss. The minimum embedment for a gabion wall is 0.5 m.

The durability of a gabion wall is dependant upon maintaining the integrity of the gabion baskets. Galvanized steel wire is required for all gabion installations. In areas of high corrosion potential due to soil, water, salt spray, or abrasion conditions, a polyvinyl chloride coating should be required in addition to galvanizing. Conditions at individual sites should be assessed to determine corrosion potential. Although gabions are manufactured from a heavy gage wire, there is a potential for damage due to vandalism. The potential for such vandalism should be considered at each specific site.

In gabion wall design, the mass of a wall will increase disproportionately with increases in height. In other words, doubling the height of a wall will more than double the mass of the wall. The ratio of the base width to height will vary, but in no circumstances should this value fall below 0.5. In practice, this value will normally range from 0.5 to 0.75 depending on backslope, surcharge and angle of internal friction of retained soil. A gabion wall has shown good economy for a low to moderate height but loses this economy as height increases. A height of about 5 m should be considered as a practical limit for a gabion wall.

A gabion wall is normally tilted back into the slope for design stability. Typically, a declination of 6 deg is used, but other angles are acceptable. A geotechnical investigation and analysis is conducted by the Materials and Tests Division's Geotechnical Section to determine soil design parameters for retained and foundation soils. Consolidation potential due to wall loads is considered when determining foundation design parameters.

The rough texture of the gabion baskets provides an attractive surface for climbing vines and plants. Plantings of this type at the base of the wall may provide a more natural appearance within a few seasons.

68-5.03(02) Design Procedure

The design of a gabion wall is not specifically covered by the AASHTO *LRFD Bridge Design Specifications*.

Design of a gabion wall must consider loads placed on it by the retained soil and any surcharges. Resistance to these loads is developed by proportioning the cross sectional area of the wall to achieve a sufficient mass to ensure stability. The analysis proceeds by computing resisting loads due to mass and dividing these by corresponding driving loads due to soil and surcharge pressures. Sliding and rotation should be considered for the full height wall and at each gabion layer in the wall. The minimum acceptable safety factors are 1.5 for sliding and 2.0 for rotation about the toe.

The base pressure of the wall cannot exceed the allowable bearing capacity of the foundation soil. Wall base pressure may be determined by using the Meyerhoff method in which vertical loads are distributed over a base area reduced for eccentricity. This method is shown in Figure 68-5D, Broken-Back Slope – Simplified Example, and Figure 68-5E, Sloping Backfill – Simplified Example. More precise base pressures may be determined by a static analysis of all forces acting about location of the resultant. Global stability may be determined by conventional soil mechanic methods or programs. A safety factor of at least 1.3 should be attained for this condition.

Lateral earth pressures are determined by multiplying vertical loads by the coefficient of active earth pressure K_a. This value may be determined by either the Rankine method or the Coulomb method at the discretion of the designer.

In addition to the actual weight of the gabions, any earth backfill bearing directly on the gabions should be included as part of the wall system. Lateral earth pressure should be assumed to act on a vertical plane rising from the back of the wall base. These conditions are illustrated in Figures 68-5D and 68-5E.

Gabion wall analysis is often simplified by separating the wall into individual sections based on gabion placement. Surcharge loads should be added when determining driving loads but should not be included when computing resisting values.

68-5.03(03) Summary of Design Safety Factors and Requirements

1. <u>Safety Factors</u>.

a. Overturning ≥ 2.0
b. Sliding ≥ 1.5
c. Global ≥ 1.3

2. Foundation Design Parameters. Use values provided by the Materials and Tests Division's Geotechnical Section.

3. Traffic Surcharge. Traffic live load surcharge = 600 mm = 37.8 kN/m^2

4. Retained Soil.

   a. Unit weight = 5.75 kN/ m^3
   b. Angle of internal friction as determined from tests from the Materials and Tests Division's Geotechnical Section.

5. Soil Pressure Theory. Rankine's Theory or Coulomb's Theory should be used at the discretion of the designer.

68-6.0 SPECIAL EARTH RETAINING SYSTEMS

68-6.01 Steel Sheet Piling Nongravity Cantilever Wall

A steel sheet piling wall is normally used as a temporary wall, but it can also be used in a permanent location. See Figure 68-2C, Cut Wall System Selection Chart, for some characteristics, including advantages and disadvantages, of sheet piling walls.

68-6.01(01) Design Procedure

A description of the design of a sheet-piling wall along with some simplified earth pressure distributions is given in the AASHTO *LRFD Bridge Design Specifications*. This type of wall is also referred to as a flexible cantilevered wall. A steel sheet pile wall can be designed as a cantilevered wall up to approximately 4.6 m in height. A steel sheet pile wall above this height may require tiebacks with either prestressed soil anchors or deadman type anchors. Anchored wall design and details are discussed in Section 68-6.02. The preferred method of designing cantilever sheet piling is by the Conventional Method as described in the United States Steel *Sheet Piling Design Manual*. The Materials and Tests Division's geotechnical engineer will provide the soil design parameters including cohesion values, angles of internal friction, angles of wall friction, soil densities, and water table elevations.

Areas of permanent steel sheet piling above the ground line should either be coated or painted prior to driving, or made from weathering steel. Corrosion potential should be considered in steel-sheet-piling design.

The appearance of a permanent steel sheet piling wall may be enhanced by applying either precast concrete panels or cast-in-place concrete surfacing. Welded stud-shear connectors can be used to attach cast-in-place concrete to a sheet of piling. See Figure 68-6A, Sheet Piling Wall Concrete Facing Detail. Special surface finishes obtained by using form liners or other means, and concrete stain or a combination of stain and paint are recommended for the concrete facing.

For information on steel sheet piling required for railroad protection, see Section 17-4.04.

68-6.01(02) Summary of Safety Factors and Requirements

1. Safety Factors. The global safety factor should be greater than or equal to 1.3.

2. Foundation Design Parameters. Use values provided by Materials and Tests Division's Geotechnical Section.

3. Traffic Surcharge. Traffic live load surcharge = 0.60 m = 37.8 kN/m^2

4. Retained Soil.

   Unit weight = 18.9 kN/m^3
   Angle of international friction as determined from tests from Materials and Tests Division's Geotechnical Section

5. Steel Design Properties. Minimum yield strength = 270 MPa.

6. Design Life for Permanent Wall. Should be 75 years. Sacrificial thickness should be taken as 50 μm/yr.

68-6.02 Anchored Wall

An anchored wall uses vertical members as main load carrying members, such as soldier piles (i.e., rolled steel section), cylinder piles, sheet piles, or slurry walls to resist forces. The main members are connected to high strength steel bars or strand anchors, which are fixed into soil or rock with high strength grout and stressed to counteract the horizontal earth pressure loads. Figure 68-6B presents a typical section of an anchored wall. The feasibility of using an anchored wall should be verified with a geotechnical engineer. This type of wall is most practical in a cut section and is best suited for a situation where excavation for a retaining wall with a footing is impractical because of traffic, utilities, existing structures, or right-of-way restrictions. The greatest advantage in using an anchored wall is that it causes minimal disturbance to the soil behind the wall and any structures resting on this soil. Non-stressed anchors, called deadman anchors, rely on passive pressure of the soil in front of the deadman panel to resist horizontal forces. An anchored wall should be designed by an anchored wall specialty contractor subject to Department approval.

The designer should prepare a special provision including all necessary design and construction requirements for providing the anchored wall system shown on the plans.

68-6.02(01) Principles of Anchor Design

Anchor design includes the following:

1. evaluation of the feasibility of anchors;
2. selection of an anchor system;
3. estimation of anchor capacity;
4. determination of unbonded length, bonded length; and
5. selection of corrosion protection.

The designer should determine whether anchors can be economically used at a particular site based on the ability to install the anchors and to develop anchor capacity. The presence of utilities or other underground facilities may govern whether anchors can be installed.

Anchors may consist of bars, wires, or strands. The choice of appropriate type is usually left to the contractor but may be specified by the designer if special site conditions exist which precludes the use of certain anchor types. Strands and wires have advantages with respect to tensile strength, limited work areas, ease of transportation, and storage. Bars are more easily protected against corrosion, and are easier to stress and transfer load.

A reliable estimate of the safe anchor capacity should be provided by a geotechnical engineer to determine the feasibility of anchoring. The capacity of each anchor should be verified by testing. Testing should be part of anchor installation and should therefore be included in the special provisions. Requirements for methods and frequency of testing of anchors are provided in the AASHTO *LRFD Bridge Construction Specifications*. A range of typical system design values is listed as follows:

1. Design loads between 270 and 1070 kN.

2. The anchor wall system should be analyzed to ensure long-term stability. The minimum unbonded length should be specified in the special provisions, and is usually 4.6 m for soil and rock anchors. Longer free lengths may be required in plastic soils. In this situation, the designer should contact the Materials and Tests Division's geotechnical engineer.

3. The angle of inclination should be between 10 deg and 45 deg. A 15-deg angle is preferred to simplify grouting and minimize vertical forces imposed on the wall by the anchors. Steeper angles, up to 45 deg, are only recommended to reach deep bearing strata or avoid existing substructures.

The ultimate anchor transfer loads per unit length may be preliminarily estimated using the guide lines presented in the AASHTO *LRFD Bridge Design Specifications* for soil and rock. Final determination of the anchored wall design should be the responsibility of the anchored wall specialty contractor selected for wall construction.

The maximum allowable anchor design load in soil may be determined by multiplying the bonded length by the ultimate transfer load and dividing by a Factor of Safety of 2.5.

The maximum allowable anchor design load in rock may be determined by multiplying the bonded length by the ultimate transfer load and by dividing by a Factor of Safety of 3.0.

68-6.02(02) Earth Pressure Distribution

For an anchored wall with two or more anchors constructed from the top down, the earth pressure force resultant per unit width of wall (N/mm) may be determined from the AASHTO *LRFD Bridge Design Specifications*.

The design should first be considered using the active earth pressure coefficient, K_a, unless structures exist within a lateral distance equal to twice the wall height. For this case, an average earth pressure coefficient, K, should be computed as follows:

$$K = K_0 - \left(\frac{x}{2H}\right)(K_0 - K_a)$$

Where: x = Distance from structure to wall
H = Height of wall
K_0 = Coefficient of at-rest earth pressure

Notes: K_a permits lower wall design pressure if small wall displacements can be tolerated, i.e., ground subsidence occurs.
K_0 increases wall design pressure but limits wall displacement, i.e., ground subsidence is limited.

68-6.02(03) Corrosion Protection

See the AASHTO *LRFD Bridge Design Specifications* for corrosion protection guidelines. Corrosion protection requirements for the anchor head, the unbonded length, and the anchor length should be included in the special provisions for the anchored wall.

68-6.02(04) Determination of Anchor Spacing

Suggested temporary test loads are between 75 and 80 percent of Guaranteed Ultimate Tensile Strength (GUTS). Suggested limits for design loads are between 0.5 and 0.6 of GUTS (typically 0.53 percent).

Typical horizontal spacing of piles of 1.8 to 3.0 m and vertical spacing of anchors of 2.4 to 3.6 m are commonly used. The minimum spacing of 1.2 m in both directions is not recommended for considering the effectiveness and disturbance of anchors due to installation.

68-6.02(05) Design of Soldier Pile Anchored Wall

1. Design of Soldier Piles. Vertical wall elements, such as soldier piles, should be designed to resist all horizontal and vertical loads in accordance with the AASHTO *LRFD Bridge Design Specifications*. If anchor inclination is required, the structural analysis of the soldier piles should consider the interaction effects of combined axial load and flexure in accordance with the *Specifications*.

2. Depth of Embedment. For cantilever piles without anchors, the embedment should be determined to satisfy horizontal force equilibrium and moment equilibrium about the bottoms of the piles.

   For piles with anchors, the depth of the embedment is determined by moment equilibrium of lateral forces about lowest anchor level.

   The moment resistance of the soldier pile member should be neglected at the level of the lowest anchor.

   Depth of embedment, *D*, should also be sufficient to provide necessary vertical capacity or adequate kick-out resistance through development of passive pressure.

3. Design of Timber Lagging. The lagging thickness is determined from past construction experience as related to depth of excavation, soil condition, and soldier pile spacing. In other cases, soil pressure distribution recommended by the Materials and Tests Division's geotechnical engineer is used to determine the thickness of lagging.

4. Design of Fascia Wall. The fascia wall should be reinforced concrete and should be designed in accordance with the AASHTO *LRFD Bridge Design Specifications*. The minimum structural thickness of fascia wall should be 230 mm. Architectural treatment of facing should be addressed in the special provisions.

   Concrete strength should not be less than 24 MPa at 28 days. The wall should extend a minimum of 600 mm below the ground line adjacent to the wall.

   Permanent drainage systems should be provided to prevent hydrostatic pressures from developing behind the wall. A cut which slopes toward the proposed wall will invariably encounter natural subsurface drainage.

   Vertical chimney drains, prefabricated drains, or porous engineering fabrics can be used for normal situations to collect and transport drainage due to a weep hole or pipe located at the base of the wall. Concentrated areas of subsurface drainage may be controlled by installing horizontal drains to intercept the flow at a distance well behind the wall. See Section 68-6.06(06) for weep hole installation details.

5. Stage Construction Check. The earth pressure distribution for an anchored wall changes during wall installation. The procedure for checking the stability of the wall system for temporary construction loadings should be the responsibility of the anchored wall specialty contractor subject to Department approval.

6. Design of Bond Length. The bond length should not be shown on the plans.

For design purposes, the required bond length should be approximated with sufficient accuracy to permit cost estimates and right-of-way acquisitions to be made confidently.

The bond transfer values for soil grout length (or bond length) should be verified by testing to determine the required bond length.

Other items to be considered are as follows:

a. A minimum bond length should be specified in the special provisions. The recommended values are 4.5 m in soil and 3.0 m in rock.

b. Bond lengths exceeding 12 m in soil or 7.6 m in rock do not efficiently increase the anchor capacity.

c. At a site with restricted right-of-way, the maximum bond length is the distance from the end of unbonded length to within 0.6 m of the right-of-way line.

d. To permit high pressure grouting without damage to existing facilities and to ensure adequate overburden pressure to mobilize the full friction between soil and grout, a 4.5 m minimum overburden cover over the bond zone is recommended for anchors of average capacity (i.e., 670 kN or less).

e. Anchors founded in mixed ground condition should be designed assuming the entire embedment is the weakest deposit.

f. The minimum unbonded length is 4.5 m.

68-6.02(06) Drainage

An anchored wall should have 100-mm diameter weepholes located a minimum of 300 mm above the final ground line and spaced about 3 m apart.

Drainage panels should be installed at each weephole and should extend from the base of the wall to a level 300 mm below the top of the wall as described in the AASHTO *LRFD Bridge Design Specifications*. A drainage panel may consist of a 600-mm wide strip of prefabricated geocomposite drain material. Drainage features should be shown on the plans.

68-6.03 Soil Nailed Wall

A soil nailed wall is an earth retaining system consisting of reinforced in-situ material which may be either original ground or an existing embankment. This is a specialty wall, and its feasibility should be verified with a geotechnical engineer. Construction is accomplished by excavating from the top of wall elevation down in stages that are typically 1.2 to 1.8 m in height. After each stage of excavation, soil reinforcing elements, or soil nails, generally consisting of reinforcing bars, are placed and grouted into drilled holes which have been drilled at a slight downward inclination from lever into the in-situ material. The face of each stage of excavation is protected by a layer of reinforced shotcrete. After the full height of wall has been excavated and reinforced, a finish layer of concrete facing is placed for the full height of the wall.

Soil nailing is most applicable for retaining excavations and for increasing the stability of slopes.

The designer is responsible for ascertaining feasibility of use of this wall type. A specialty contractor will be responsible for the structural design and preparation of the contract documents.

This type of wall should be considered experimental where the conditions exist as follows:

1. the wall height is greater than 9 m;
2. the wall is to be built in clay or soils with sufficient clay content such that the soil mass will behave as a clay (based on engineering considerations); and
3. the wall has an unusual surcharge load.

A permanent facing system is required. The permanent face of the wall should be vertical, although the shotcrete facing of the soil nailed wall may be battered.

Soil nailing has unique technical and economic advantages over an MSE retaining wall in the aspects as follows:

1. A soil nailed wall is constructed incrementally from the top down, which will eliminate the cost of temporary sheeting or shoring systems required for MSE wall excavation.

2. The volume of excavation is significantly reduced as compared to that for an MSE wall.

3. B borrow is not required for a soil nailed wall.

4. Soil nailed wall construction and excavation should proceed significantly faster than MSE wall construction due to less excavation volume and elimination of shoring.

5. Only light construction equipment and simple grouting equipment are required to install nails. Grouting of the boreholes is generally accomplished by gravity. This feature may be of particular importance for the project site in a traffic congested area.

However, the specific design details of the nail length and location must be developed by the contractor and submitted for review and approval by the Department. The soil parameters for soil nailed wall design are listed in Figure 68-6C.

A soil nailed wall in clay soils typically requires nail lengths between 0.7 and 1.0 times the height of the wall, with 0.85H being a typical ratio. Permanent wall easements may be necessary to accommodate the soil nails.

Ultimate pullout resistance (friction limit) of each nail is a function of the size and shape of the drill hole, strength characteristics and density of the soil in which it is placed, drilling method, length tested, method to clean the drill hole, and grouting method or pressure used, if any.

The construction of a soil nailed mass results in a composite coherent mass similar to that of an MSE wall. The locus of maximum tensile forces separates the nailed soil mass into the zones as follows:

1. An active zone, or potential sliding soil wedge, where lateral shear stresses are mobilized and result in an increase of the tension force in the nail; and

2. A resistance, or stable, zone where the generated nail forces are transferred into the ground.

The design of a soil nailed retaining structure is based on evaluation of the following:

1. Global stability of the structure and the surrounding ground with respect to a rotational or translational failure along potential sliding surfaces; and

2. Local stability at each level of nails.

Typically, a computer program is used for soil nailed wall analysis. Global stability analyses for an earth retaining system consist of evaluating a global safety factor of the soil nailed retaining structure and the surrounding ground with respect to a rotational or translational failure along potential sliding surfaces. It requires determination of the critical sliding surface which may be dictated by the satisfaction of the subsurface soil and intensity of surcharge loads, as well as the specific design of the reinforcing elements' spacing, length, and location. Because global stability is a function of the nail length and spacing, it is evaluated as part of the design of the wall, and cannot be evaluated independently of reinforcement spacing, as is typical for an MSE wall.

The special provisions should include requirements for the installation of a prefabricated vertical wall drain.

68-7.0 REINFORCED SOIL SLOPES

Reinforced Soil Slopes (RSS), are a cost-effective alternative for new construction where right-of-way or other considerations may make a steeper slope desirable. As shown in Figure 68-7A, Slope Reinforcement Using Geosynthetics to Provide Slope Stability, multiple layers of reinforcement are placed in the slope during construction or reconstruction to reinforce the soil and provide increased slope stability. Reinforced soil slopes are a form of mechanically stabilized earth that incorporates planar reinforcing elements in constructed earth sloped structures with face inclinations of usually 45 deg or less. Typically, geosynthetics are used for reinforcement.

68-7.01 Purpose of Reinforcement

The principal purpose for using reinforcement is to construct an RSS embankment at an angle steeper than could otherwise be safely constructed with the same soil as shown in Figure 68-7A. The stability allows for construction of steepened slopes on a firm foundation for a new highway and as a replacement for a flatter unreinforced slope or a firm foundation for a retaining wall. A roadway can also be widened over existing flatter slopes without encroaching on existing right-of-way. In the case of repairing a slope failure, the new slope will be safer, and reusing the slide debris rather than importing higher quality backfill may result in substantial cost savings. The minimum Factor of Safety for internal stability is 1.3.

The second purpose for using reinforcement is at the edges of a compacted fill slope to provide lateral resistance during compaction as shown in Figure 68-7B, Slope Reinforcement Providing Lateral Resistance During Compaction. The increased lateral resistance allows for an increase in compacted soil density over that normally achieved and provides increased lateral confinement for the soil at the face. Even modest amounts of reinforcement in compacted slopes have been found to prevent sloughing and reduce slope erosion. Edge reinforcement also allows compaction equipment to more safely operate near the edge of the slope.

Right-of-way savings can be a substantial benefit, especially for a road widening project in an urban area where acquiring new right-of-way is always expensive and, in some cases, impossible. RSS also provide an economical alternative to a retaining wall. In some cases reinforced slopes can be constructed at about one-half the cost of an MSE wall structure.

Further compaction improvements have been found in cohesive soils through the use of geosynthetics with in-plane drainage capabilities (e.g., nonwoven geotextiles) that allow for rapid pore pressure dissipation in the compacted soil.

Compaction aids placed as intermediate layers between reinforcement in steepened slopes may also be used to provide improved face stability and to reduce layers of more expensive, primary reinforcement as shown in Figure 68-7A.

The use of vegetated-faced reinforced soil slopes that can be landscaped to blend with a natural environment may also provide an aesthetic advantage over a retaining wall type structure. However, there are some maintenance issues that must be addressed such as mowing grass-faced, steep slopes.

For an RSS structure, the choice of slope facing may be controlled by climatic and regional factors. For a structure of less than 10 m height with slopes of 1:1 of flatter, a vegetative “green slope” can be usually constructed using an erosion control mat or mesh and local grasses. Where vegetation cannot be successfully established or significant runoff may occur, armored slopes using natural or manufactured materials may be the only choice to reduce future maintenance.

In terms of performance, due to inherent conservation in the design, RSS are actually safer than flatter slopes designed at the same Factor of Safety. As a result, there is a lower risk of long-term stability problems developing in the slopes. Such problems often occur in compacted fill slopes that have been constructed to low Factors of Safety or with marginal materials (e.g. deleterious soils such as shale, fine grained low cohesive silts, plastic soils, etc.). The reinforcement may also facilitate strength gains in the soil over time from soil aging through improved drainage, further improving long-term performance.

68-7.02 Economics

Reinforced soil slopes are normally not constructed with rigid facing elements. Slopes constructed with a flexible face can thus readily tolerate minor distortions that can result from settlement, freezing and thawing, or wet-drying of the backfill. As a result, any soil meeting the requirements for embankment construction may be used in a reinforced soil slope system. However, a higher quality material reduces concerns for the durability of the reinforcement, and is easier to handle, place, and compact, which speeds up construction.

The performance of reinforced soil slopes is generally not affected by differential longitudinal settlements.

RSS construction with an organic vegetative cover should be carefully chosen to be consistent with native perennial cover that would establish itself quickly and would thrive with available site rainfall, and is maintenance free.

RSS maybe cost effective in a rural environment, where right-of-way restrictions exist or on a widening project where long sliver fills are necessary. In an urban environment, they should be considered where existing right-of-way is sufficient for construction, as they are always more economical than a vertically-faced MSE wall structure.

68-7.03 References

Reference publications regarding RSS include the following:

1. National Research Council, Transportation Research Board, *National Cooperative Highway Program Report 290, Reinforcement of Earth Slopes and Embankments*.

2. U.S. Department of Transportation, Federal Highway Administration, Publication *FHWA-RD-89-043, Reinforced Soil Structures, Volume I: Design and Construction Guidelines*.

3. U.S. Department of Transportation, Federal Highway Administration, Publication *FHWA-SA-93-025, Guidelines for Design, Specification, and Contracting of Geosynthetic Mechanically Stabilized Earth Slopes and Embankments*.