A Geogrid-Reinforced Soil Wall for Landslide Correction on the Oregon Coast

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ABSTRACT

In June and July 1983, the Oregon State Highway Division constructed a geogrid-retained soil wall to stabilize a landslide on the Oregon coast. The project was an FHWA Experimental Features Project. The experimental aspects of the project were to assess construction problems of near-vertical walls with high-density polyethylene geogrids and to investigate the feasibility of establishing vegetation on the wall face to provide a natural appearance at an esthetically sensitive site. The experience gained in the design and construction of the geogrid wall is presented. Problems encountered during construction are discussed and recommendations are made for improved methods for future application. It is concluded that geogrid wall construction is practical. Geogrids are more labor intensive than conventional geotextiles, but their greater strength and ultraviolet light resistance are compensating advantages. Establishment of vegetation on the face of a geogrid wall is possible by placing sod strips between the backfill and the geogrid. A coarse backfill or a filter fabric should be used if sod is not placed against the face to limit the loss of fines.

The Oregon State Highway Division has utilized a high-density polyethylene grid-reinforced wall to stabilize a landslide on the Oregon coast. The geogrid used was Tensar SR-2. Geogrids have been used around the world and have the potential for many applications (1). They have not, however, been used previously for near-vertical retaining walls in the United States.

The slide correction was performed as an FHWA Experimental Features Project and was constructed during the summer of 1983. The objectives of the project were to assess the construction of geogrid walls and to investigate establishment of vegetation on the wall face. The purpose of this report is to present the experiences gained in the design and construction of the geogrid wall.

BACKGROUND

The Experimental Features Project is located just off the Oregon Coast Highway on Otter Rock Highway 182 in the vicinity of Devil's Punch Bowl State Park, approximately 15 miles north of Newport, Oregon. Figure 1 shows the general location of the project on the Oregon coast.

The experimental geogrid wall was a replacement for a 12-ft concrete rubble wall. The replacement was necessitated by a slide failure that occurred in December 1981. The slide dropped the pavement 4 ft on the easterly edge, severely cracked a concrete rubble wall, and forced the closure of the main entrance to the popular Devil's Punch Bowl State Park. Figure 2 shows the original concrete rubble wall and the extent of the slide failure.

Three alternatives were considered by the Oregon State Highway Division for stabilizing the slide. The first alternative was a tie-back soldier pile wall with precast concrete panels and a lightweight backfill. The second alternative was a nonwoven geotextile retaining wall with a gunite facing. The geogrid wall, the third alternative, was chosen over the other two alternatives for two reasons:

1. It had the lowest estimated cost and
2. The open face allowed establishment of vegetation, which provided a natural appearance compatible with the surroundings of the state park.

The geogrid wall had the lowest estimated cost because it did not require a facing for protection from ultraviolet (UV) light as did the conventional geotextile wall. In the planning stages of the project, preliminary designs for both the geogrid and a conventional geotextile were completed. For these preliminary estimates, the geotextile design required 36 layers of reinforcement using 11,500 yd^2 of fabric. Because of its greater strength, the geogrid wall only required 21 layers and 6,000 yd^2 of material. Although the geogrid wall did require handling less reinforcing material, the unit cost for placing the geogrid was estimated to be greater than that for the geotextile, for two main reasons:

1. The geogrid was supplied in rolls 3.3 ft wide, whereas the geotextile rolls were 16 ft wide; therefore, many more individual geogrid pieces must be handled; and
2. The geogrid required forming thicker layers, so more robust, complex forms were needed.

It was estimated that the backfill placement costs would be nearly the same for the two materials. The geotextile wall would have been less expensive because the material had a lower unit price, but because of its low UV resistance, it would have required an additional expense for a protective facing. Thus, the geogrid wall was selected because it did not require a facing to protect it from sunlight and it was possible to provide a more natural appearance that would not detract from the esthetics of the park.

SITE INVESTIGATION

Site investigation was carried out by the local Highway Division Soils and Geology Section during July 1982. Six boreholes were located within the slide area, and two steel inclinometer tubes were installed to establish the plane of failure and to monitor the groundwater levels. Monitoring of the site was carried out during the winter of 1982.

The soil profile, defined by the exploration phase, consisted of a 12-ft layer of medium to stiff yellow-brown sand and a layer of soft gray silty

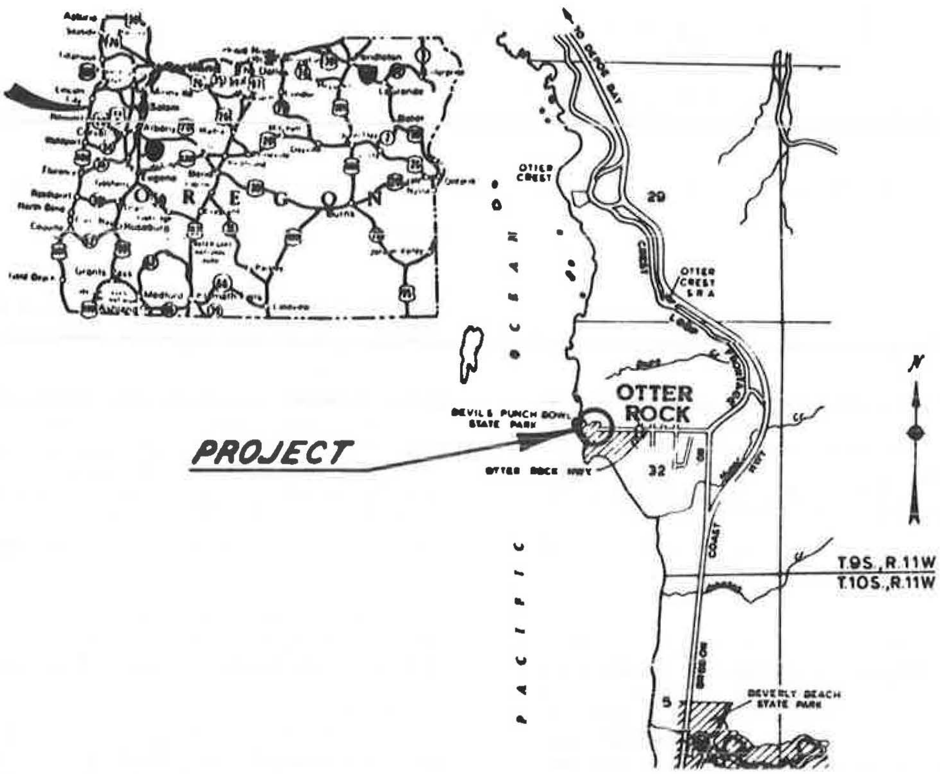

FIGURE 1 Geogrid wall site location.

FIGURE 2 Site before construction.

clay varying in thickness from 0 to 12 ft and underlain by gray shale. The failure plane defined by the inclinometer tubes was at the clay-shale interface. Figure 3 shows a typical cross section of the slide and the failure plane. The slide resulted from water that caused the fractured shale to deteriorate into a soft weak clay. Two faults in the slide area caused the hard gray shale to fracture, and excess water from the sand layer triggered the slide. Therefore, the main objectives of the slide correction were to control the water flowing in the sand layer and to prevent further deterioration of the shale.

SLIDE CORRECTION DESIGN

The general scheme of the slide correction was to excavate to the firm intact shale, build the layered geogrid wall, and provide perforated drain pipes below the sand layer to control the groundwater. Figure 4 shows a typical cross section of the geogrid wall.

The decision was made to build the geogrid wall on a 6 (vertical) to 1 (horizontal) slope to attain a neat face and provide an area for natural vegetation. The final section was dictated by the presence of an existing 24-in. storm sewer pipe, a public restroom facility, and the requirement of maintaining two 12-ft travel lanes and a 4-ft shoulder plus guardrail. The bottom of the excavation was to be made to Elevation 45 to intercept the firm shale below the failure surface. The geogrid wall was to be founded on a 1-ft layer of well-compacted gravel at an elevation of 46 ft.

The front view of the geogrid wall approximates a trapezoid the bottom of which is 70 ft long and is tapered on both sides to a top length of 170 ft. The wall at the top is stepped to fit the vertical curve of the roadway. The sag point elevation is 74.5 ft, which dictates the minimum height of the wall to be 29.5 ft. An elevation view of the wall and the controlling elevations are shown in Figure 5.

The design also called for common backfill to be placed over the lower face of the wall to reestablish the natural ground surface. Above the natural ground line sod was to be placed between the gravel backfill and the Tensar geogrid. Use of sod was believed to be the most economical way to establish vegetation on the face. To accommodate future growth, a dirty backfill (class-B backfill) was placed in the first 2 ft behind the sod, and a cleaner gravel (class-A backfill) was used as the remainder of the fill.

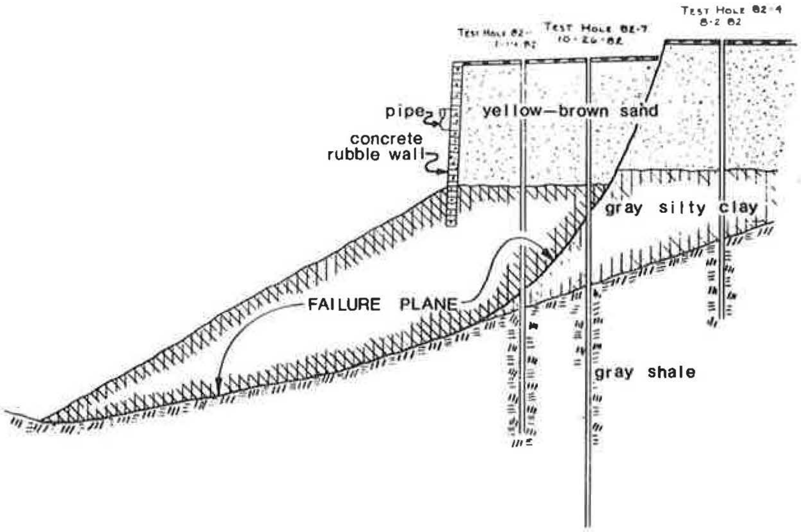

FIGURE 3 Geologic cross section of site.

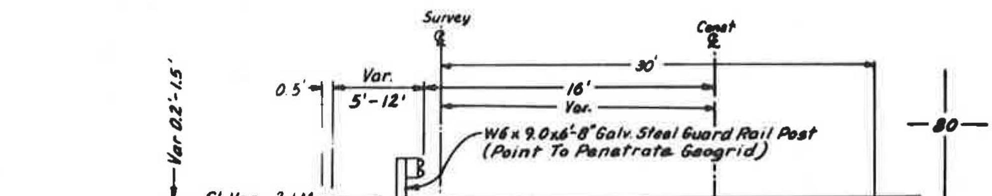

FIGURE 4 Cross section of geogrid wall.

GEOGRID WALL DESIGN

The geogrid polymer is a high-density polyethylene stabilized with carbon black to provide UV light resistance. The grid material is illustrated in Figure 6. The grids are supplied in rolls 3.3 ft wide and 98 ft long. Tensar SR-2 has a strength of 5.413 kips/ft in the principal direction and a weight of 27.61 oz/yd^2. Strain at failure is 12 percent and strain at 40 percent of maximum strength is 3 percent. In comparison, a conventional nonwoven geotextile, Trevira 1127, has a strength of 1.1 kips/ft and a weight of 6.5 oz/yd^2.

The backfill material used for the geogrid wall was a graded crushed basalt with 2-in. maximum size; the A-zone material had a maximum of 10 percent fines, and the B-zone had a maximum of 20 percent fines to accommodate the growth of the sod. Specifications required at least 95 percent of standard optimum dry unit weight (AASHTO T99). The bulk density

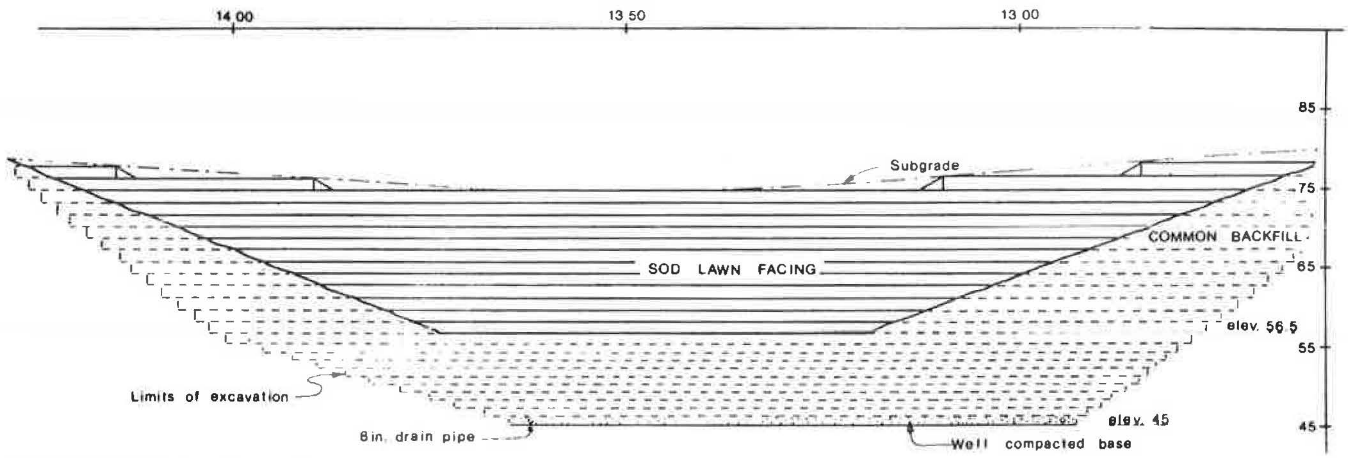

FIGURE 5 Wall elevation section.

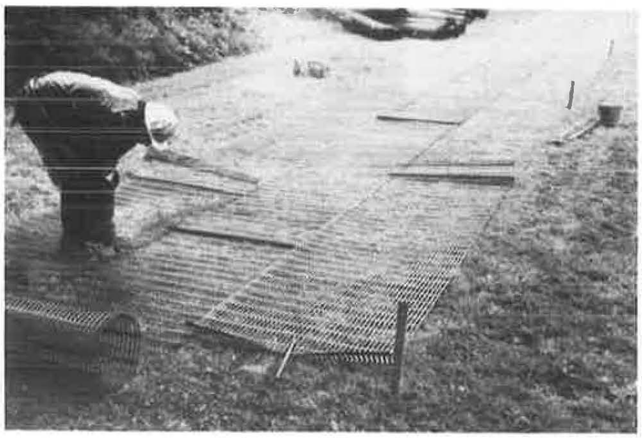

FIGURE 6 Fastening the geogrid strips together.

and angle of internal friction for the backfill were assumed to be 140.0 lb/ft^3 and 40 degrees, respectively.

To limit possible creep of the reinforcement, the working stress for the geogrids was taken as 40 percent of the ultimate strength. The open structure of the grids allowed for the interlocking of the backfill material across the grid; therefore, the full soil friction was assumed to be developed at the soil-geogrid interface.

The wall was designed with the assumption that the grids had to resist the active Rankine lateral earth pressures by the portion of the reinforcement extending beyond the theoretical Rankine failure surface. The method of analysis was described by Lee et al. (2) and Hausmann (3) for Reinforced Earth walls and was modified for geotextile walls by Bell and his co-workers at Oregon State University (4,5). This method has been used by the Forest Service (6,7), New York Department of Transportation (8), Colorado Department of Highways (9), and others to construct successful geotextile walls in the United States.

Geogrid lengths and vertical spacings were calculated to provide minimum safety factors of 2.0 for dead load only and 1.15 for dead load plus live load, whichever was more restrictive. The reduced factor with live loads was allowed because

1. After construction, truck traffic would be limited to recreational vehicles and an occasional service vehicle, and

2. The allowable working load included a safety factor of 2.5 against a short-term failure.

The vertical spacing calculated for the geogrid wall was 1 ft at the bottom of the wall and approximately 4.6 ft at the top. For appearance and construction considerations, the wall was detailed with 5-ft steps. Each step was set back 6 in. from the one below to give the wall an average batter of 1:6 (see Figure 4). The lower three layers were given reinforcement spacings of 1 ft, the midheight layers spacings of 1.5 ft, and the top two layers reinforcement spacings of 3 ft. To give a uniform appearance the geogrids were folded back into the backfill at midlayer height for the top two layers. This fold was only anchored a distance of 5 ft into the backfill because the embedment was required to stabilize the face and was not required for overall stability. The anchored distance at the top was the same as the 5-ft overlap embedment used for each layer.

The geogrid reinforcement lengths were 16 ft. This length was required at the top for resistance to failure by pullout of the reinforcement and at the bottom to provide resistance to horizontal sliding of the total reinforced block.

To keep the costs of the geogrid wall competitive, it was necessary to select a simple effective method of supporting the face during construction. According to John Tempelman of Netlon Limited, Blackburn, England, scaffolding from the ground level in front of the wall has been used successfully in England and elsewhere. The steep site, wall geometry, and the need to operate equipment in front of the wall made scaffolds impractical for this wall. As has been done on geotextile walls (6,8,9) the state suggested the use of movable self-supporting forms.

Because reinforcement spacing was 3 ft at the top, a 3-ft forming system was required. The decision was made to use the same system throughout and construct the wall with 3-ft steps. Experience on a wall in Glenwood Canyon, Colorado, indicated that the simple movable forms previously used were not suitable for layers greater than about 15 in. Therefore, a forming system was suggested by the state in the contract documents that incorporated the same concepts of the previously used geotextile forms but had special features to allow for thicker layers.

The suggested forming system is shown in Figures 7 and 8. The contract documents indicated that the contractor could use another system or modify the suggested method. The state had hoped that the contractor would add ideas and modify the system during

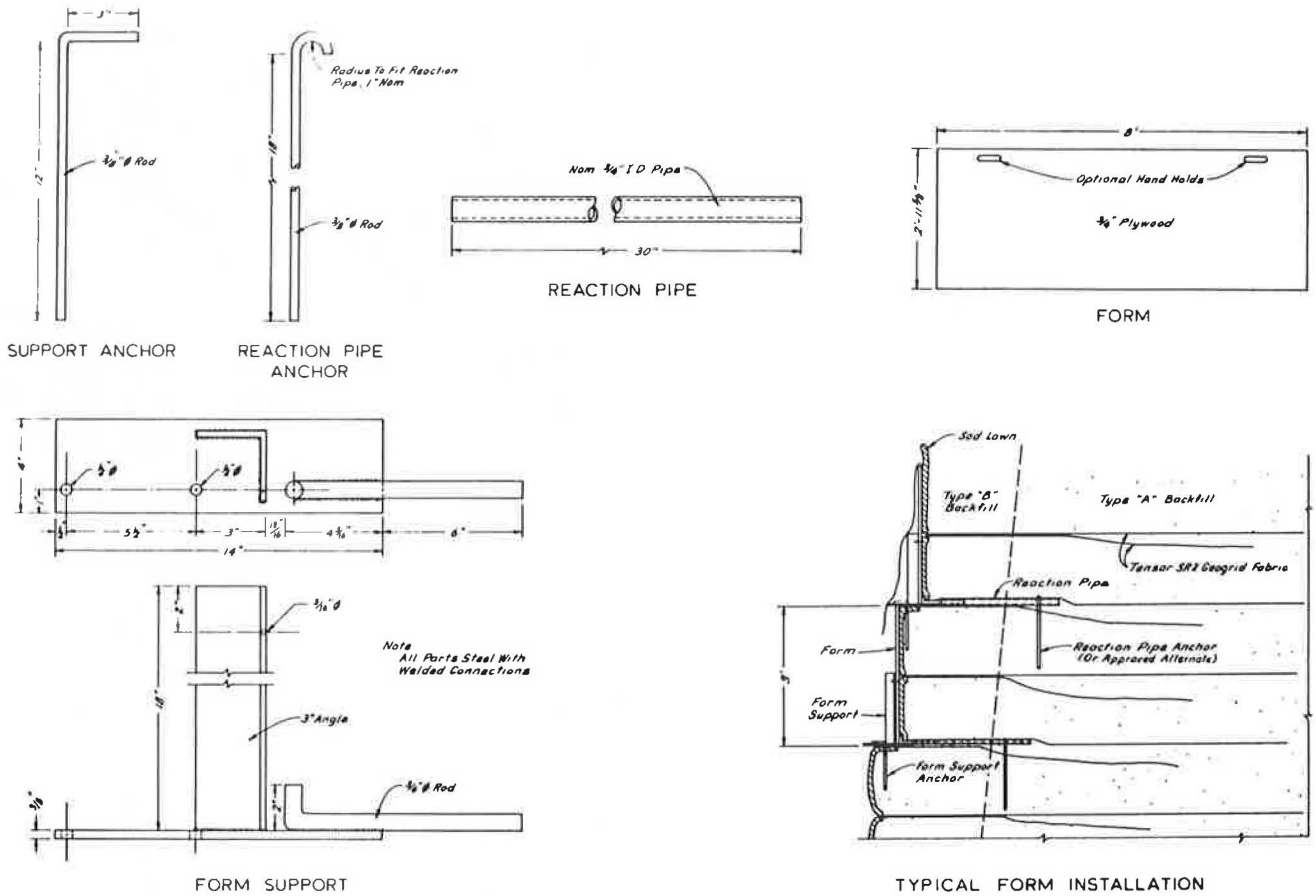

FIGURE 7 Suggested construction form details.

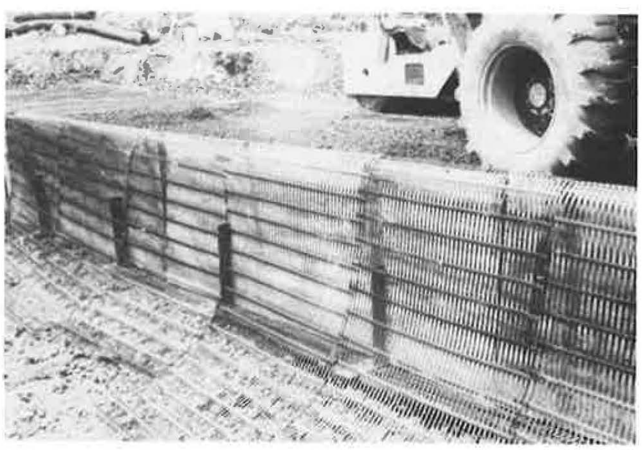

FIGURE 8 Suggested forming system with geogrid in place.

construction, which would lead to the development of a more efficient forming system that could be used on future projects.

The suggested form consisted of a 3 x 8-ft sheet of 3/4-in. plywood held in place by the upright on the form support. To resist overturning, the form support was anchored in the backfill. There was concern that if the form support base extended into the backfill far enough to provide stability, friction would make it difficult to pull the base out at the completion of the layer. Therefore, a sacrificial reaction pipe was anchored in the backfill, and the rod on the form support was inserted into the pipe. The rod on the form support was bent upward to prevent kickout of the bottom of the plywood form. Because there was little friction on the form support base, an anchor rod was used to provide lateral resistance.

As shown by the typical installation in Figure 7, it was anticipated that the forms for a completed layer would be left in place while the next layer was constructed. The lower form would add stability to the upper form and help maintain vertical and horizontal alignments. The form supports would be leveled and shimmed as required, depending on the placement of the lower layer. When the upper layer was completed, the lower forms would be removed and moved up to form the next layer and so forth. It was believed that this system and procedure would be simple, expedient, and stable for the 3-ft layers.

CONSTRUCTION

Final design of the geogrid wall was completed in February 1983, and the contract was awarded in April. The Highway Division estimated the project cost to be $165,802, and the low bid received was $166,328. A total of five contractors bid on the project, and the highest bid for the work was $269,000.

A summary of the salient features and a cost comparison with other walls appear in Table 1.

Excavation of the site began June 6. The month of June was quite wet and portions of the excavation slopes failed. Actual wall construction did not begin until the middle of the month and not before

TABLE 1 Data Sheet

Item	Description
Fabric	Tensar SR-2
Weight	27.6 oz/yd^2
Strength	451 lb/in.; 5,413 lb/ft
Chemical makeup	High-density polypropylene
How made	Drawn with heat
Roll dimensions	
Width	3.3 ft
Length	98 ft
Weight	62 lb
Height	29 ft
Face wall	3,900 ft^2; 2,500 ft^2 of sod
Amount of fabric	6,000 yd^2
Design angle of internal friction (ϕ)	40 degrees
Time	
Construction	21 days (32 days including excavation)
To reset forms	3 hr
Cost	
Fabric	\$4.50/yd^2
Fabric (cost to state)	\$6.75/yd^2
Labor	\$13,500/3,900 ft^2 = \$3.46/ft^2
Per square foot of wall	
Without backfill	\$11.44/ft^2
With backfill	\$24/ft^2 (A and B backfill)
30-ft concrete cantilever[a]	\$50/ft^2
30-ft Reinforced Earth[a]	\$18/ft^2
30-ft VSL Corporation retained earth[a]	\$20/ft^2
30-ft Doublewal[a]	\$20/ft^2
30-ft metal bin[a]	\$38/ft^2
Permanent tie back[a]	\$50+/ft^2

[a]1981 FHWA data that are still relevant (does not include backfill).

problems had been encountered with the excavation, the groundwater, and surface runoff.

Uncovered in the excavation were plugged horizontal drain pipes that had been installed in 1975. Once broken by the backhoe, they immediately began to allow the flow of water into the excavation. The added water resulted in further deterioration of the shale layer, and so the unplugged drain pipes were thought to have contributed to the slides at the excavation site.

The general procedure followed by the contractor in the early stages of the wall construction was as follows:

1. Set the proposed forms at gradeline;
2. Lay out prefabricated sections, made up of two to three sheets of geogrid;
3. Drape the fabric over the forms, allowing for required embedment lengths, and secure the fabric with No. 3 rebar anchor pins;
4. Place hog rings to secure the panels to one another at the face;
5. Place class-A backfill in 6-in. lifts to desired layer thickness;
6. Level and compact;
7. Place sod in position beyond the geogrid;
8. Place class-B backfill and compact;
9. Fold overlap and pin fabric to completed backfill; and
10. Continue lifts until the top of the 3-ft form is reached, then remove forms and move them up for the next 3-ft layer.

Figures 6 and 10 show aspects of the wall construction procedure. Figure 6 shows a worker securing the sheets of geogrid into a section and splicing the ends of the geogrid with No. 3 rebar. A masonry circular saw was used to cut the geogrid. Figure 8 shows the initial forming system and the draping of the grid over the form. Figure 9 is an overview of the initial wall construction that shows the restricted space and the placement of the class-A backfill. Figure 10 shows a worker hanging the sod strips on the form and shows the space left for the dirty class-B backfill. The light compaction equipment used near the face of the wall to compact the class-B backfill is shown in Figure 11, and Figure 12 shows the pinning of the overlap and deflections experienced with the initial forming system. In Figure 13 the equipment used by the contractor and part of the drainage network installed to intercept the groundwater can be seen. Figure 14 shows the completed geogrid wall.

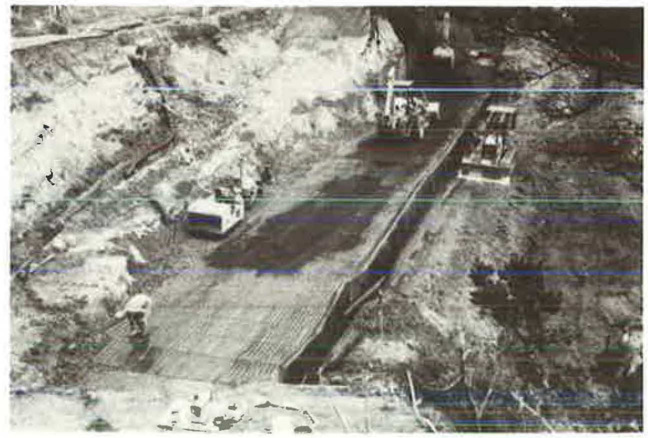

FIGURE 9 General view of construction site and placement of class-A backfill.

As the geogrid wall gained in height, several problems began to occur. The first was that the contractor was not achieving 95 percent of the standard maximum dry density. The frequent rain showers and the backfill gradation did not allow the material to drain, so the in-place moisture content was several points above optimum. The decision was then made to lower the density requirement to 90 percent and place a rock blanket of material 1.5 to 2.5 in. thick against the excavation backslope to intercept groundwater and improve the drainage.

The second problem was the sagging and bulging of the wall face. This problem was caused by excessive flexibility in the proposed forms and the loss of class-B backfill through the grid where sod had not been placed between the geogrid and the backfill.

FIGURE 10 Hanging sod on back of forms.

FIGURE 11 Compacting class-B backfill near the forms.

FIGURE 12 Pinning geogrid overlap in place.

FIGURE 13 General view of construction site during backfill compaction.

The time between when the forms were removed and when the face was covered by common backfill was long enough for significant amounts of the fine class-B backfill to be lost from behind the grid. Where sod had been placed against the geogrid reinforcement, the fines were inhibited from movement and the wall face was nearly vertical. The problem

FIGURE 14 Completed geogrid wall.

of bulging was not deemed important in the lower layers, because they would be covered. However, the sagging of the wall resulted in modification by the contractor of the method of forming the face of the wall.

As stated previously, the suggested forming system was too flexible. The combination of the 3/4-in. plywood forms and the 18-in. form supports on 4-ft centers resulted in the deflection of the forms. A more serious problem, which led the contractor to modify the forming method, resulted from the loss of support from under the forms.

As discussed in the preceding section, it was expected that the forms for a completed 3-ft step would be left in place until the forms above had been set and at least the first lift of that step was in place. The contractor elected not to follow the double-form system and moved the forms as each 3-ft step was completed. Also, the contractor used plastic rather than steel reaction pipes. The result of both decisions was that the stability of the forms was totally dependent on the support of the backfill directly under the metal plate of the previous 3-ft layer (see Figure 7). Without the lower form in place, the slight inevitable bulging of the face resulted in tipping of the form support. Loss of the finer backfill compounded the problem of the form support, and with the form support stiffened only by the plastic reaction pipe, the form tipped even further. Also, because of the loss of backfill material, the effectiveness of the form support anchor was reduced, which caused the form system to become unstable.

The contractor's solution to the forming problem is shown in Figures 15 and 16. Figures 15 and 16 show the modified forming system and the new forms. The forms employed by the contractor were stiffened with 2 x 4-in. lumber and braced against a 2 x 4-in. support extending 4 ft into the backfill to provide an anchor. The protruding end of the horizontal anchor was supported by a vertical member and an 8-in. spike was driven at the end of the support into the lower layer. The bottom of the 3/4-in. plywood form was held in place by 2 x 4-in. lumber nailed to the anchor support. At least three braces were used on each 8-ft forming unit. The new forming system required considerably more time to construct but did provide a stable face against which to build.

The geogrid wall was completed July 27. The construction time was considerably longer than the estimated 10 working days. This resulted from adverse weather conditions, difficulties in scheduling the work because of the confined space, and the labor-

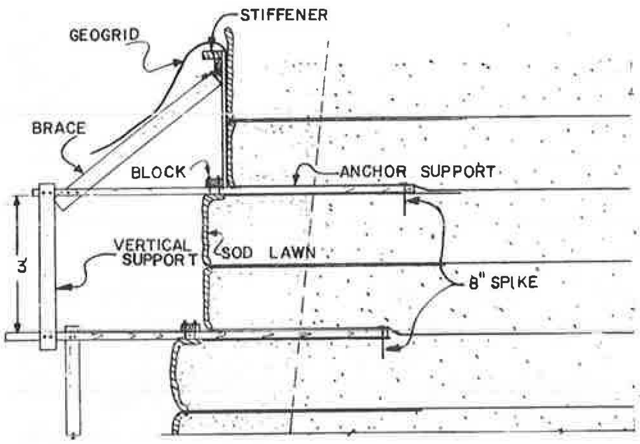

FIGURE 15 Modified forming system.

FIGURE 16 Modified forming system and heavy vibrating roller used for backfill compaction.

intensive nature of the construction. The completed wall is shown in Figure 14.

EVALUATION AND RECOMMENDATIONS

The geogrid wall has only been in service a short time, but it appears to have stabilized the site. The sod facing grew and the appearance was satisfactory, but lack of irrigation killed most of the sod by mid-September 1983. The geogrids have potential and are competitive in cost with the conventional geotextile walls where a natural appearance is desirable. Improvements in construction techniques are necessary to fully utilize the potential of geogrid materials.

At suitable sites, scaffolding may be the solution to the forming problems. In other situations modifications of the movable forms originally suggested for this project are recommended. Several modifications to the forming system are proposed:

1. Stiffen the plywood form with 2 x 4-in. lumber along the top,
2. Secure adjacent forms to each other with battens,
3. Lengthen the upright on the form supports to be 6 in. shorter than the form,
4. Eliminate the reaction pipe and all anchor pins and extend the base plate of the form support 3 ft into the backfill,
5. Weld rings on the short end of the form support base plate so mechanical aides can be used if necessary to pull the plate free after the layer has been completed,
6. Use at least three form supports on each 8-ft form,
7. Use backfill coarser than the grid openings or use a layer of a geotextile behind the face of the wall to prevent loss of backfill through the grid, and
8. Be careful to compact near the forms and tightly secure the geogrid overlaps on the tops of the layers.

With these considerations, the forms should perform satisfactorily and may be removed and moved up with each layer. These changes will expedite construction and make the geogrid walls even more practical.

ACKNOWLEDGMENT

The project was constructed during the summer of 1983 as an FHWA Experimental Features Project. The wall was built by Dan D. Allsup, Contractor, Eugene, Oregon. The wall was designed in cooperation with the Regional Soils and Geology Section, the Headquarter's Bridge Foundation Unit, the Geotechnical Unit, and the Road Design Section of the Oregon Department of Transportation's Highway Division. The authors wish to specially thank Chuck Elroy, the project manager; Claudius Groves, the construction inspector; Tensar Incorporated, Ontario, Canada, for providing a field engineer during the early stages of the wall construction; and Judy Banegas, management assistant of the Oregon Department of Transportation, for typing the manuscript.

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Performance of an Earthwork Reinforcement System Constructed with Low-Quality Backfill

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ABSTRACT

A preliminary evaluation is presented of a retaining-wall system constructed on Interstate 80 at Baxter, California, using low-quality backfill materials. Four mechanically stabilized embankments were constructed. Two of these walls were instrumented to monitor performance over a 3-year period. Dummy bar-mats of different configurations were installed during construction and were subject to field pullout testing. These field pullout results are compared with laboratory tests conducted with the same backfill material at representative overburden and field moisture and density conditions. Field pullout test results are also compared with laboratory tests conducted before the project design. The results of these tests suggest that the laboratory pullout test values provide a conservative design. The transverse bars governed the pullout capacity of the bar-mat and the overburden stress did not significantly affect the pullout capacity in cohesive backfill. The contractor's method of construction significantly influenced the overall response of the soil-reinforced wall system. The satisfactory performance of this wall system after one severe winter season suggests that mat- (or mesh-) type soil reinforcement systems can be constructed successfully by using low-quality on-site materials as backfill.

The evolution of the mechanically stabilized embankment (MSE) has been described in some detail in the literature (1,2). Results of large-scale laboratory pullout tests by the California Department of Transportation (Caltrans) and others (3) have convincingly demonstrated the greatly increased pullout resistance of mesh-type reinforcement in that less steel is exposed to soil as compared with the case of flat reinforcing strips. Recognition of this may have been a factor in the introduction of ribbed reinforcing strips by the Reinforced Earth Company in 1978 (4). These early tests revealed not only a different failure mechanism as compared with the flat reinforcing strips but also extremely high pullout resistance in poor-quality backfill. Pullout resistance in a silty clay (Table 1) was found to compare favorably with that obtained in gravelly sand when strain criteria were used. These data confirmed the original premise that the use of mesh reinforcement could result in significant savings when quality backfill was not readily available.

TABLE 1 Comparison of Pull Resistance for Bar Mesh Embedded in Gravelly Sand and Silty Clay Soils *(1)*

Soil Type	Mesh Opening[a] (in.)	Confining Pressure (psi)	Yielding Load (kips)	Peak Load (kips)	Residual Load (kips)
Gravelly sand	4 x 8	10	17.3	37.5	25.7
Gravelly sand	4 x 8	20	20	44	35.8
Gravelly sand	4 x 8	25	20	60	43
Silty clay	4 x 8	10	19	39.5	33.5
Silty clay	4 x 8	20	21	55.5	37.5
Silty clay	4 x 8	40	24	66	59
Silty clay	5 x 14	10	10	29	27
Silty clay	5 x 14	20	11	30	30
Silty clay	5 x 14	40	12	41	41

[a]No. 3 reinforcing bars.

For the first MSEs, constructed on I-5 near Dunsmuir, California, in 1974, high-quality backfill was used so that a direct comparison in performance could be made with a Reinforced Earth (RE) wall of approximately equal height on the same project. The results, based on extensive instrumentation of both systems, were presented in some detail in 1982 (5). Although there was a great deal of interest in constructing a prototype MSE with marginal-quality backfill, either subsequent MSE projects were in areas where high-quality backfill was readily available or the nature of the installation was unsuitable for a long-term evaluation.

At the request of Caltrans District 3, a feasibility study for the construction of four MSEs near Baxter, California, was initiated in April 1979.