Long-term Behavior of Geogrid Reinforced Soil Walls

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ABSTRACT: Two types of 5m high geogrid reinforced soil walls (gradient V:H=1:0.1) with two kinds of wall facing (wrapping type and L-shaped concrete block type) were trial constructed in 1990, and an 8m high vertical reinforced soil wall with concrete block wall facing was trial constructed in 1995. From the beginning of the construction stage, wall displacement or strain of the geogrid, earth pressure, settlement of the foundation ground, etc. were measured for a long period of time. In 2002, when the first walls were about 12 years old and the second wall was about 7 years old, parts of the reinforced soil walls were excavated and examined the state of the wall facing and geogrid combinations and the vegetation on the wall surface. The long term behavior of the geogrid reinforced soil walls was evaluated based on these measurements and observations.

1 INTRODUCTION

Two types of 5m high reinforced soil walls with two kinds of wall facing (wrapping type and L-shaped concrete block type) were trial constructed using geogrids in 1990, and an 8m high reinforced soil wall with concrete block wall facing was trial constructed in 1995 in the site of the Public Works Research Institute (PWRI). From the construction stage, the wall displacement or strain of the geogrid, the earth pressure at the bottom of the reinforced soil walls, etc. were measured for a long period, revealing that no substantial change of the environments around any of these soil walls has occurred till the present time.

The use of geogrids to build reinforced soil walls has been increasing year by year since the geogrid reinforced soil method was first introduced to Japan in 1983. Because the method was at the research and development stage when it was introduced, many of the reinforced soil walls were applied as temporary or trial construction, with the result that there were few full-scale reinforced soil wall built using geotextiles that have been measured continually for more than 10 years.

In addition to the long-term measurements of performances on these three kinds of reinforced soil walls were observed, several portions of the reinforced soil walls were excavated to confirm the state of the wall facing and geogrid combinations and the vegetation on the wall surfaces. This report describes the results of a study of the long-term behavior of geogrid reinforced soil walls based on these measurements and observations.

2 OUTLINE OF THE REINFORCED SOIL WALLS AND SURVEY

Three kinds of reinforced soil walls, Type 1 to Type 3 were constructed. Table 1 presents outlines of the reinforced soil walls that were constructed. Type 1 and Type 2 walls were constructed in 1990 (Onodera et al., 1992) and Type 3 wall was constructed in 1995 (Ochiai et al., 1996, Nakajima et al., 1996, Tsukada et al.,
Different kinds of geogrids were laid in each type as shown in Table 2. Outlines of each type of reinforced soil wall are described as follows.

Table 1. Outline of geogrid reinforced soil walls

<table>
<thead>
<tr>
<th>Type</th>
<th>Height (m)</th>
<th>Gradient (V:H)</th>
<th>Geogrid</th>
<th>Wall facing</th>
<th>Safety Factor for Circular Slip</th>
<th>Construction Process</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.0</td>
<td>1:0.1</td>
<td>N</td>
<td>L-shaped concrete block</td>
<td>Fc=1.375</td>
<td>Start: 6/Dec/89, Completion of banking: 25/Jan/90 (50days), Surcharge banking: 2/Jun/90 (178days)</td>
</tr>
<tr>
<td>2</td>
<td>5.0</td>
<td>1:0.1</td>
<td>T (SR2)</td>
<td>Sand bag (wrapping type)</td>
<td>Fc=1.242</td>
<td>Start: 6/Dec/89, Completion of banking: 25/Jan/90 (50days), Surcharge banking: 2/Jun/90 (178days)</td>
</tr>
<tr>
<td>3</td>
<td>8.0</td>
<td>1:0.0</td>
<td>T (SR55)</td>
<td>Concrete block</td>
<td>Fc=1.057</td>
<td>Start: 12/Mar/95, Completion of banking: 28/Apr/95 (48days), Surcharge banking: 9/May/95 (58days)</td>
</tr>
</tbody>
</table>

Table 2. Properties of geogrids

<table>
<thead>
<tr>
<th>Type of Geogrid</th>
<th>N</th>
<th>T (SR2)</th>
<th>T (SR55)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main Material (covering materials)</td>
<td>Glass fiber (vinyl ester)</td>
<td>HDPE</td>
<td>HDPE</td>
</tr>
<tr>
<td>Structure</td>
<td>Bonded</td>
<td>Extruded</td>
<td>Extruded</td>
</tr>
<tr>
<td>Pitch of Ribs (mm)</td>
<td>100 □ 30</td>
<td>166 □ 22.5</td>
<td>166 □ 22.5</td>
</tr>
<tr>
<td>Maximum Tensile Strength</td>
<td>100 kN/m</td>
<td>80 kN/m</td>
<td>50 kN/m</td>
</tr>
<tr>
<td>Design Tensile Strength</td>
<td>40 kN/m</td>
<td>32 kN/m</td>
<td>30 kN/m</td>
</tr>
</tbody>
</table>

2.1 Type 1 and Type 2 reinforced soil walls

Type 1 and Type 2 reinforced soil walls were 5m high with slope gradients of 1:0.1. The wall facing used to build Type 1 was piling 50cm high L-shaped concrete blocks. Type 2 was constructed with wall facing by wrapping the soil bags using a geogrid. The L-shaped concrete blocks and the geogrids were connected by attaching steel bars linked to the geogrids to hooks installed in the concrete blocks. Figure 1 shows the configurations of the Type 1 and Type 2 reinforced soil walls. The banking material was sandy soil, and because it contained fine-grain as reinforced soil as banking material of reinforced soil wall, drainage sand layer were taken by placing horizontally and vertically behind the wall facing as shown in Figure 1. The banking material was compacted to be 85% or more of maximum dry density.

The arrangement of geogrids was designed in order to satisfy the design safety factor (Fs=1.2) through stability calculations for circular slip in order to ensure the safety of a truck driving test above the reinforced wall after completion of the banking work.

(a) Cross-section
Four months after completion of wall construction, an overburden embankment equivalent to the surcharge traffic load considered for the design (10kN/m²) was constructed on top of each soil wall and monitored for a long period.

2.2. Type 3 reinforced soil wall

The type 3 wall is a vertical 8m high reinforced soil wall with wall facing of piled 0.5m high concrete blocks. The concrete blocks are hollow blocks that are 0.5m high, 1.0m wide and 0.35m thick and are filled with crushed stone. They are connected to the geogrid by attaching steel pipes linked to the geogrids to hooks installed in the concrete blocks. Figure 2 shows the configuration of the Type 3 reinforced soil wall.

The banking material is the same sandy soil used for Type 1 and Type 2. A drainage layer was placed horizontally inside the embankment, and a vertical drainage layer of crushed stone was placed behind the wall facing. The banking material was compacted at least 90% of maximum dry density.

The geogrid arrangement was designed so that it would be extremely close in order to verify the effectiveness of concrete block wall facing, and so that when stability calculations were performed according to the PWRl design manual (1992), however, the design safety factor against circular slip was set $F_s \geq 1.0$ as critical condition. Immediately after completion of the banking work, an overburden embankment equivalent to the surcharge traffic load considered for the design (10kN/m²) was constructed on top of the soil wall.

Figure 2. Cross-section and front view in 2002 of Type 3 wall constructed in 1995
2.3 Outline of the survey

Measurements were conducted for a long period of time from the beginning of the construction stage, continuing after its completion. As shown in Figure 3, the measurement items for each reinforced soil wall were, in the case of Type 1 and Type 2 wall, the vertical and horizontal displacement of the wall, the strain of the geogrid, and the vertical earth pressure at the bottom of the reinforced soil wall, while in the case of type 3, they were the horizontal displacement of the wall, the settlement of the foundation ground of the reinforced soil wall, the interior and top of the embankment, displacement of the ground in front of the wall, strain of the geogrid, the horizontal earth pressure acting behind the wall facing, and the vertical earth pressure at the bottom of the reinforced soil wall.

In 2002, twelve years (4,570 days) after construction of the Type 1 and Type 2 and 7 years (2,768 days) after construction of the Type 3, several partitions of reinforced soil walls were excavated and examined the state of construction of the Type 3, several partions of reinforced soil walls were excavated and examined the state of wall facing and geogrid combinations and vegetation on the wall surface, and tested the properties of the banking material.

Records of rainfall and earthquakes after construction were also studied. The results of rainfall monitoring at a nearby monitoring station showed that the maximum daily rainfall was 192mm/day (Sep. 22, 1996) and the maximum hourly rainfall was 50mm/hour (Sep. 8 1991, 11:00 a.m. to noon). Results of nearby seismic observations showed that the maximum acceleration was 139gal (June 14, 2002, horizontal direction).

Figure 3. Instrumentation layout

3 MEASUREMENT RESULTS

3.1 Deformation of the wall

Figure 4 shows the distribution of the horizontal displacement of the wall after the completion of each type wall. To exclude the effect of displacement of the foundation ground, the reference point was set at the bottom of the wall facing to represent the displacement as the relative displacement. If relatively flexible wall facing material is used as in the Type 2 case, the displacement of top layer of wall tends to lean forward. If, however, relatively stiff wall facing material as in the Type 1 and Type 3 is used, an arc-shaped distribution with the maximum point near the middle of the wall height is formed and the displacement tends to advance throughout the entire wall height. Although these displacement distributions seem to vary slightly according to the gradient of the wall, it is assumed that, in the steep reinforced soil with gradient from V:H=1:0.0 to 1:0.6, these results are the typical displacement characteristics corresponding to the structures of the wall facing.

Figure 5 shows change over time of the horizontal displacement of the wall from the beginning of banking work. The longitudinal axis represents the horizontal displacement value which is the value obtained by divid-
ing the average horizontal displacement of the wall by the wall height. The straight line \( \Rightarrow \) to \( \Rightarrow \) in the figure was obtained by the logarithmic approximation of the horizontal displacement rate after the surcharge banking. For all types of walls, most of the increase of horizontal displacement occurred during banking, and later the displacement gradually increased approximate to the straight line in the logarithmic coordinates. Therefore, the speed of displacement tended to decrease, as the deformation of the body of the reinforced soil wall settled.

Differences based on the stiffness of the wall facing appeared conspicuously in displacement during banking work, and it is inferred that in the case of Type 2 with relatively flexible wall facing, the displacement increased about twice as much as in the relatively stiff wall facing case, and that the stiffness of the wall facing is extremely effective in restraining the deformation.

The maximum horizontal displacement of the wall in 2002 was 2.3% of the wall height in Type 2 with relatively flexible wall facing and 1.2% in Type 3 with relatively stiff wall facing. These are values that fall below the control reference value (allowable maximum value of 3.0% (0.03H)) of wall horizontal displacement for the vertical reinforced wall with concrete panel wall facing.

Figure 4. Lateral displacement of each wall

Figure 5. Horizontal displacement rate of each wall

### 3.2 Strain of the geogrids

Figure 6 shows the typical distributions of strain of the geogrids classified according to differences of their wall facing. If relatively flexible wall facing as in the Type 2 is used, the strain of the geogrid at each layer has a distribution shape resembling a parabola with its peak near the active failure line. If stiff wall surface material like that in the Type 1 and Type 3 case is used, the strain of each geogrid at each level has a distribution shape that is uniform or triangular with its peak near the wall facing, revealing the effects of differences in the form of the wall facing. In this way, the strain distributions in Figure 6 can be assumed as characteristic distribution shapes according on differences in the form of wall facing. The quantity of strain of geogrid in the reinforced soil with stiff wall facing is overall smaller than that with relatively soft wall facing, and it is assumed that the confining effects for the embankment due to the longitudinal stiffness of the wall facing is high.

In Figure 7, the rate of increase of the strain \( \frac{de}{d\log t}, t: \) days) at each point of each geogrid after the surcharge banking in the Type 3 wall is plotted. It shows that the rate of increase of the strain is high near the wall surface in all geogrids, and that localized increase in tension occurs through the entire height of the soil wall. In particular, the largest strain increase rate was found in geogrids installed near the middle of the wall height, 5.65 m high (8th layer) and 3.65 m high (7th layer) from the bottom.

In Figure 8, the settlement by settlement plates (No. 1 to No. 5) installed inside and at the top of the Type 3 wall are substituted for the relative settlement to the settlement of the bottom of the wall facing to plot change over time from the beginning of the banking work. Hereupon, the settlement of the bottom of the wall facing is assumed as the settlement at each point of the wall facing considering the stiffness of the wall facing material. Therefore, a characteristic of the settlement of the reinforced soil body is that at the foundation ground level (No. 1), the settlement of the wall facing is dominant to the embankment side, but inside the embankment (No. 2 to No. 5), the embankment side tends to settle down larger than the wall facing, and in particular, the largest relative settlement occurs at the middle of the embankment height (settlement plate No. 2). This
conforms with the location of the geogrid where the rate of increase of the strain near the wall facing shown in Figure 7 is highest. And the relative settlement of the embankment fluctuates until about 400 days and it is linked to the tendency of the increase of the strain of the geogrid. Consequently, differences in the settlement of the banking material and of the wall facing can be inferred as the principal cause of the localized rise of strain of the geogrid near the wall facing.

![Figure 6. Strain of geogrid in Type 2 and Type 3 walls](image)

**3.3 Vertical earth pressure at the bottom of reinforced soil wall**

Figure 9 shows the distribution of the vertical earth pressure at the bottom of the Type 2 and Type 3 wall from the beginning of banking work. In the case of Type 2 with relatively flexible wall facing, the banking process caused vertical earth pressure equivalent to the overburden pressure to act on the entire bottom of the reinforced soil area. In Type 3 with stiff wall facing, high earth pressure that exceeds the overburden pressure is produced at the bottom of the wall facing from the initial stage of the banking, and behind the wall facing, a unique distribution that is far lower than the overburden pressure is applied in a range that extends almost to the active failure zone. These distributions are characteristics dependent on the structure of the wall facing, and little change can be seen, even approximately one year after the surcharge banking. The design values of vertical earth pressure do not represent the actual distribution, and the present design model that presumes

![Figure 9. Vertical earth pressure at the bottom of reinforced soil wall](image)
the reinforced soil area to be a gravity retaining wall can not be used to appropriately evaluate the behavior of a reinforced soil wall. In the case of Type 3 that is a vertical wall with relatively high stiffness, the concentration of stress at the bottom of the wall is a problem, but as in the Type 2 case, the vertical earth pressure of a reinforced soil wall could be considered to be equivalent to the overburden pressure. Hence, it will be important to study a design model that can be used to appropriately express the earth pressure at the bottom of reinforced soil wall.

4 DEFORMATION BEHAVIOR OF THE TYPE 3 REINFORCED SOIL WALL

A reinforced soil wall with a vertical gradient such as Type 3 is a structure that is prone to conspicuously show the effects of deformation. In the foundation ground where Type 3 wall was constructed, the bearing capacity obtained by a plate bearing test exceeds the required design value of vertical earth pressure, while near a depth of 10m from the ground surface, there is a soft layer with N value of 10 and less. Because this means that in Type 3, the impact of the deformation of the foundation ground appears, the overall behavior of the reinforced soil wall including the deformation of the foundation ground was also considered as follows.

4.1 Ground deformation under wall facing

Figure 10 shows the degree of increase of settlement of the ground surface in front of the reinforced soil wall, the bottom of the wall facing, and the foundation ground after the execution of the surcharge banking. Under the effects of the concentrated earth pressure at the bottom of the wall facing, the settlement has a more protruding distribution than at other locations. And there is a tendency for settlement to occur in the surrounding ground, pulling in the wall facing. Figure 11 shows change over time of the settlement at each of these points after the execution of surcharge banking. It is inferred that the settlement tends to change to secondary consolidation behavior that can be approximate to the straight line in the logarithmic coordinates from about 300 days after the surcharge banking, and the speed of settlement tends to decrease. The settlement at the bottom of the wall facing after the surcharge banking was 52mm, a value that is about 0.7% of the height of the reinforced soil wall. This settlement is smaller than the residual settlement of an embankment on ordinary soft ground, pulling in the wall facing. Figure 11 shows change over time of the settlement at each of these points after the execution of surcharge banking. It is inferred that the settlement tends to change to secondary consolidation behavior that can be approximate to the straight line in the logarithmic coordinates from about 300 days after the surcharge banking, and the speed of settlement tends to decrease. The settlement at the bottom of the wall facing after the surcharge banking was 52mm, a value that is about 0.7% of the height of the reinforced soil wall. This settlement is smaller than the residual settlement of an embankment on ordinary soft ground, pulling in the wall facing.

4.2 Deformation of the reinforced soil body

Figure 12 shows the change of the measured vertical earth pressure at the bottom surface of the reinforced soil wall from the beginning of banking work. As above stated, it has been shown that the banking process caused localized high increase in the earth pressure at the bottom of the wall facing. But from about 30 days after completion of the surcharge banking, the vertical earth pressure at all measuring points including the bot-
tom of the wall facing changed to almost steady. Therefore, even as ground settlement continued, the functions of the reinforced soil wall were maintained.

Figure 13 shows change over time of the inclination of the foundation ground and of the wall facing under the effects of settlement after the completion of the surcharge banking. In the same way as the change of the settlement near the wall facing, the inclination of the foundation ground and of the wall facing both progressed until 300 days after completion of the surcharge banking, but later, regardless of the continued gradual increase of the inclination of the foundation ground that accompanies the progress of the settlement of the wall facing, the inclination of the wall facing tended to settle and the deformation behavior to differ. In the Type 3 wall, this causes a unique distribution with high concentrated vertical earth pressure at the bottom of the wall facing, and this in turn, results in a tendency for the inclination to continue till the present as localized unequal settlement occurs in the foundation ground that consists of a weak ground. But the body of the reinforced soil wall follows this deformation as it maintains its functions as a sound soil structure.

Reinforced soil walls with concrete blocks as their wall facing are used often in Europe and U.S. to take advantage of their ability to improve the scenery. But, as seen in the results of the measurement, it is necessary to estimate an appropriate design value by preparing a model that can quantitatively evaluate various phenomena such as the localized increase of tensile strain of the geotextile near the wall facing or a discontinuous vertical earth pressure distribution at the bottom.

5 RESULTS OF OBSERVATIONS OF THE REINFORCED SOIL WALLS

About 12 years after construction of the Type 1 and 2 walls and about 7 years after construction of the Type 3 wall, the external appearance of each wall was observed and part of each reinforced soil wall was excavated to observe its interior in order to examine the state of vegetation on their wall surfaces, the soundness of structure of the wall facing, etc as shown figure 14.

The wall facing of the Type 2 consists of seeded soil bags wrapped by geogrid and this wall has a gradient of V:H=1:0.1, but immediately after completion of the embankment, growth of vegetation from the seeded soil bags used as wall facing material was confirmed. Twelve years after construction, the vegetation growing from the seeded soil bags was still sparse, but at level differences on the wall facing, shrubs and other vegetation were observed. According to the examination of the covering material used as soil bags, it was found that exposed bag material had been degraded by ultraviolet radiation, but the bag material inside the embankment had not deteriorated. Meanwhile, the deterioration of the exposed geogrid could not be confirmed visually.

The visual observations of the laid geogrids as reinforcement at the excavated points found no evidence of any defects such as tears or other damage. Within the observation of the excavation portions, no defects such as breakage or other damage could be seen on the connections between the geogrid and the concrete blocks used as wall facing on the Type 1 and Type 3 walls. And the roots of shrubs growing naturally on the top and other parts of the reinforced soil walls were still growing after passing through the geogrids.

The above observations have confirmed that the three reinforced soil walls are in sound condition.
6 CONCLUSION

Various measurements of three types of reinforced soil walls such as displacement of wall, strain of geogrid, vertical earth pressure at the bottom of reinforced soil wall, settlement of embankment and foundation ground, etc. were conducted for a long period of time. And part of each reinforced soil wall was excavated and observed after twelve and seven years from their completion. This has obtained the following knowledge concerning the behavior of geogrid reinforced soil walls over a long period of time and actual condition of wall facing and geogrid lain inside the embankment.

[1] Concerning the deformation of the wall after construction, in the case of relatively flexible wall facing material, displacement advances in the top layer, but in the case of relatively stiff wall facing material, arc-shaped deformation with its peak close to the middle of the wall height appears.
[2] Concerning the increase of horizontal displacement of the wall, for all three types of reinforced wall, most of the increase occurs during banking, and the increase of displacement after the completion is gentle gradual increase proportional to log t, and tends to settle.

[3] Strains of the geogrid are, in the case of flexible wall facing, distributed in the shape of a parabola with its peak close to the active failure line, and if relatively stiff wall facing is done, its distribution is shaped like a triangle with its peak close to the wall facing.

[4] In the case of relatively stiff wall facing, in all the laid geogrids, a localized high strain increase rate is observed near the wall facing, with the highest rate of strain increase confirmed in geogrid laid particularly close to the middle of the wall height. The principal cause of this phenomenon is assumed to differential settlement near the boundary between wall facing and embankment caused by differences in the characteristics of the settlement of the embankment material and that of the wall facing.

[5] Concerning the vertical earth pressure at the bottom of reinforced soil walls, in the case of flexible wall facing, earth pressure equivalent to the overburden pressure is produced across the entire bottom of the reinforced soil area, while in the case of stiff wall facing work, the earth pressure has a unique distribution with high localized pressure at the bottom of the wall facing.

[6] The study has also confirmed that the geogrid and wall facing effectively provides a restraining effect against deformation of the foundation ground that continues after construction so that it follows this ground deformation maintaining its functions as a sound soil structure.

[7] The partial excavation and survey of the reinforced soil walls found no evidence of any damage to the geogrids or the wall facings that were used, confirming that they were in sound condition.

Continued measurements of the three types of reinforced soil wall will be carried out to study their further long-term behavior.

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