International Geotechnical Symposium on Theory and Practice of Earth Reinforcement/Fukuoka Japan/5-7 October 1988
 ISBN 90 6191 8200

Design and construction of steel bars with anchor plates applying to strengthen the high embankment on soft ground

Masami Fukuoka
Science University of Tokyo, Tokyo, Japan

Masaaki Goto
Kiso-Jiban Consultants, Chiba, Japan

ABSTRACT: A high embankment for interchange was to be constructed on soft ground of 10m in thickness without causing harmful displacement at the ground surface near the toe of the embankment. A new method i.e. the steel bar reinforcing method, was proposed to reduce deformation at the ground surface near the toe of the embankment and to strengthen the embankment. The precision of predicting displacement was improved by conducting soil surveys, building test embankments and analyzing divergencies of predicted values from values observed in the test embankments. Displacement was thus successfully minimized. For predicting the displacement of the ground surface near the toe and the proportion of tensile forces in each layer of the steel bars, finite element method (FEM) was found to be the best method available.

1 INTRODUCTION

A high embankment on soft ground was contemplated at Hayashima Interchange, at the north end of the expressway connecting Kojima (Honshu) and Sakaide (Shikoku). Though the area at the toe of the embankment was mainly used for rice fields, significant ground surface displacement at the toe could not be tolerated. For strengthen the embankment, the steel bar reinforcing method, i.e. placing steel bars with anchor plates at the bottom of embankment, was adopted. This method was used on a small scale at Muchiki about 20 km south of Hayashima Interchange. The results of the test were used in the construction work at Hayashima.

2 OUTLINE OF MUCHIKI EMBANKMENT

Figure 1 shows cross section of the embankments at Muchiki. Sand piles were driven into the soft ground and steel bars with anchor plates were placed at the bottom of the embankment.

Ac: ALLUVIAL SILTY CLAY
Ams: ALLUVIAL SILTY SAND
Dc: DILUVIAL SILTY CLAY
Ds: DILUVIAL SAND
N: N-VALUE OF S.P.T.
SD: SAND DRAIN
H: POINT FOR MEASURING HORIZONTAL DISPLACEMENT

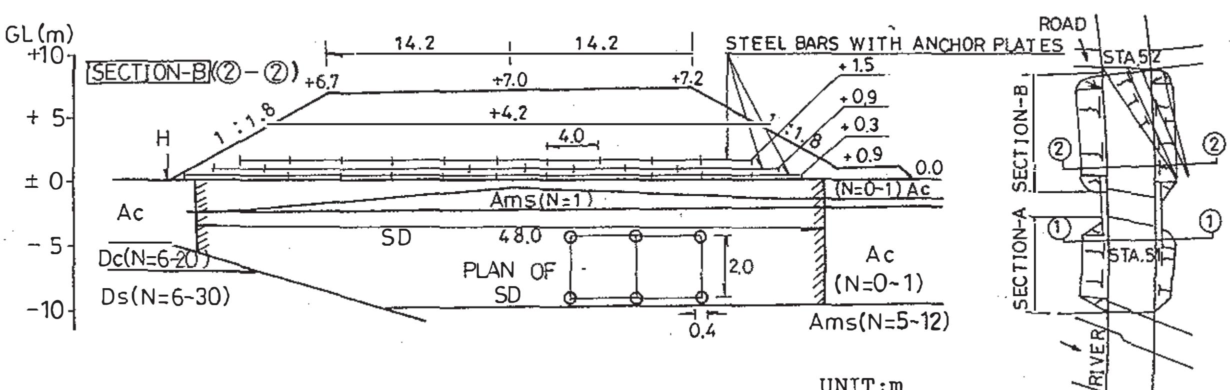


Fig. 1 Plan and cross sections of Muchiki embankment

Properties of the soils are described in Fig. 2. Triaxial compression tests and consolidation tests were also performed. The soil constants used for design were as follows.

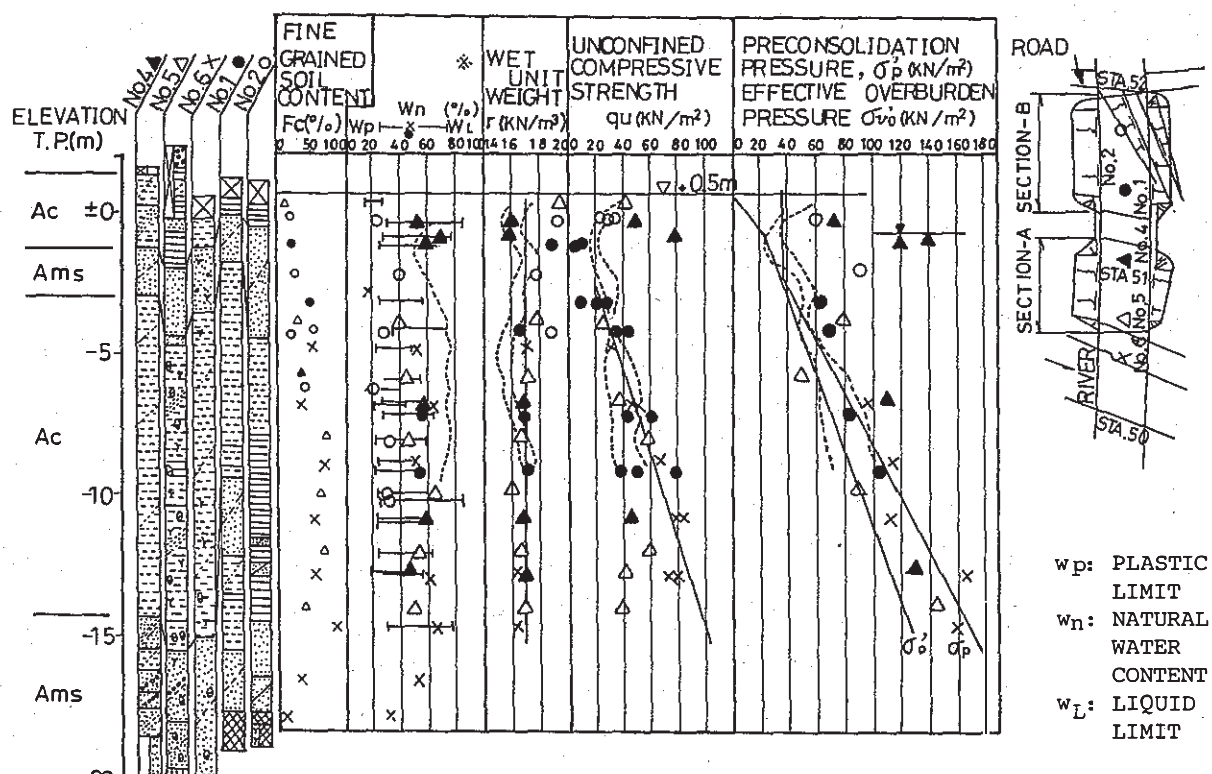
For soft clays: unit weight γ is 17 kN/m^3, unconfined compressive strength qu is 20 kN/m^2 from the ground surface to the depth of 3m, and $\{20+6.6(z-3)\}$ kN/m^2 below the depth of 3m. Preconsolidation pressure σ_p' and effective overburden pressure σ_{v0}' increase with depth, as shown in Fig. 2. Coefficient of consolidation c_v is 3.3 x 10^{-7} m^2/s. For an embankment of weathered granite: unit weight γ is 20 kN/m^3, cohesion c is 10 kN/m^2, angle of internal friction ϕ' is 30°.

For FEM analysis, the following soil constants were used in addition to the above soil constants. Unit weight of submerged soil for As layer γ' is 10 kN/m^3. Poisson's ratio ν and coefficient of earth pressure K for As layer and embankment are 0.3 and 0.43, respectively. Poisson's ratio ν and coefficient of earth pressure K for Ac layer are 0.33 and 0.50, respectively. Coefficient of deformation E for clay is derived from the coefficient of volume compressibility m_v and the Poisson's ratio ν. Coefficient of deformation E for sand is obtained by the results of pressuremeter test, and the formula E = 7N (where, N is the N-value of the SPT) was used for reference. Coefficient of deformation E for embankment is 10 MN/m^2.

3 DESIGN OF STEEL REINFORCING BARS WITH ANCHOR PLATES

In 1963 Fukuoka used steel reinforcing bars with anchor plates to strengthen an embankment of 40m in height on soft ground stabilized with sand compaction piles. Sand piles were used at Muchiki instead of sand compaction piles to avoid noise pollution during construction. Sand piles of 40cm in diameter were driven at intervals of 2m and to depths of 8 - 13m in pattern to square.

Anchor bars were inserted only at the bottom of Muchiki embankment as it was lower than the 1963 embankment. Deformed steel bars of 22mm in diameter having steel bearing plates 250 x 300 x 9mm in dimension were placed at intervals of 50cm in horizontal direction and 60cm in vertical direction. As it was very hard to predict tensile forces acting on the



DESIGNING VALUE
HAYASHIMA'S VALUE

FINE-GRAIN means soil grain having grain size less than 7.5 x 10^{-5}m in diameter

Fig. 2 Soil map of sections A and B at Muchiki bars, three layers of steel bars were provided. Therefore, the number of bars per unit length of embankment was 6.

Pullout tests of bars were performed at the construction site. Pullout strength and pullout force at 1cm displacement were 120 - 140 and 70 kN/m, respectively.

The following methods were used for predicting maximum tensile forces on the bars.

(1) Circular arc analysis (Fig. 3(a)).
Tensile force on the bars is obtained by the following formula.

$$T \cdot h + R\,\Sigma\,(Su \cdot \Delta\ell) \geqq F \cdot W \cdot d \qquad (1)$$

$$T \geqq \frac{1}{h}\,\{F \cdot W \cdot d - R\,\Sigma\,(Su \cdot \Delta\ell)\} \qquad (1)'$$

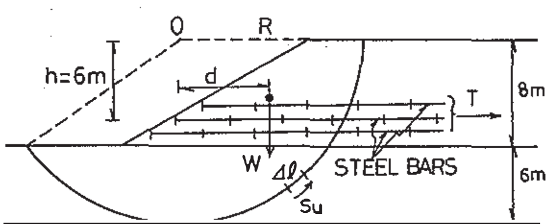
where, T : total tensile force, h : vertical length between center of circular arc and the point where total tensile force act, R : radius of circular arc, Su : shear strength, $\Delta\ell$: length of element acting shear force, F : factor of safety, W : total weight of soil inside of circular arc, d : horizontal length between center of circular arc and the point where total weight of soil inside of circular arc act.

365 kN/m of tensile force was obtained.

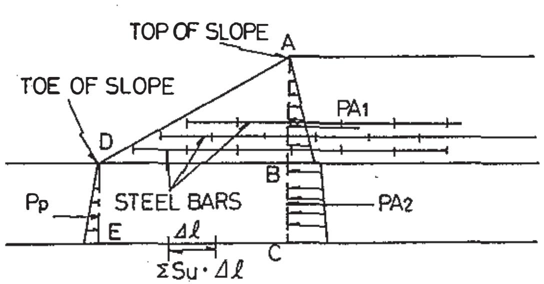
(2) Method of using earth pressures on assumed vertical walls (Fig.3(b)).
Tensile force on the bars T is calculated by the following formula.

$$S + T \geqq (P_{A1} + P_{A2} - Pp) \cdot F \qquad (2)$$

$$T \geqq (P_{A1} + P_{A2} - Pp)\,F - S \qquad (2)'$$



(a) Circular arc analysis



(b) Earth pressure method

Fig. 3 Method of estimating tensile force of reinforcing steel bars

Where, F is factor of safety, S is total shear stress along the bottom of the soft clay layer which is obtained by the following formula.

$$S = \Sigma\,(Su \cdot \Delta\ell) \qquad (3)$$

where, Su : unconfined compressive stress divided by 2, $\Delta\ell$: length of element between top and toe of slope. P_{A1} is horizontal earth pressure on the assumed vertical wall at the top of slope of the embankment which is obtained by the following formula.

$$P_{A1} = 1/2 \cdot K \cdot \gamma \cdot H^2 \qquad (4)$$

where, k : coefficient of earth pressure, γ : wet unit weight, H : embankments height. P_{A2} is the active earth pressure on the vertical wall at the same position in the soft foundation. Pp is the passive pressure on the vertical wall in the soft foundation at the toe of slope.

358 kN/m of tensile force was obtained.

(3) Estimation by observations at Ebetsu testing embankment.
The Japan Road Public Corporation tested an embankment of 8m in height over soft ground of 10m in thickness. Two layers of steel strips of 2.3mm in thickness and 140mm in width, were laid at the bottom of the embankment. Horizontal and vertical spacings were 0.5 and 1.0m, respectively. Maximum tensile stress on the strip of the lower layer at the center of the embankment was greater than the yielding stress of 3 MN/m^2. Analyzing this case, 386 kN/m of tensile force was obtained for the case of Muchiki.

(4) Model test.
A model test of 1/10 scale was performed. The tensile force on the steel bars was calculated as 292 kN/m for the case of Muchiki.

(5) FEM analysis (the first stage).
The soil constants used were given above. The steel bars were assumed to be bar elements resisting the deformation of the embankment by axial tensile force. We assumed that E_sA value of the bars was 25 MN/m (real E_sA value was 160 MN/m), in order that the tensile stress was nearly equal to the allowable stress. Where, E_s is Young's modulus and A is the cross sectional area of a steel bar. The tensile forces per unit length of the embankment were calculated as 112.1 kN/m at the lowest layer, 50.8 kN/m at the middle layer and -10.6 kN/m at the highest layer. Thus, the total tensile force was 163 kN/m. We knew later that this E_sA value was too small.

4 PREDICTED AND OBSERVED VALUES

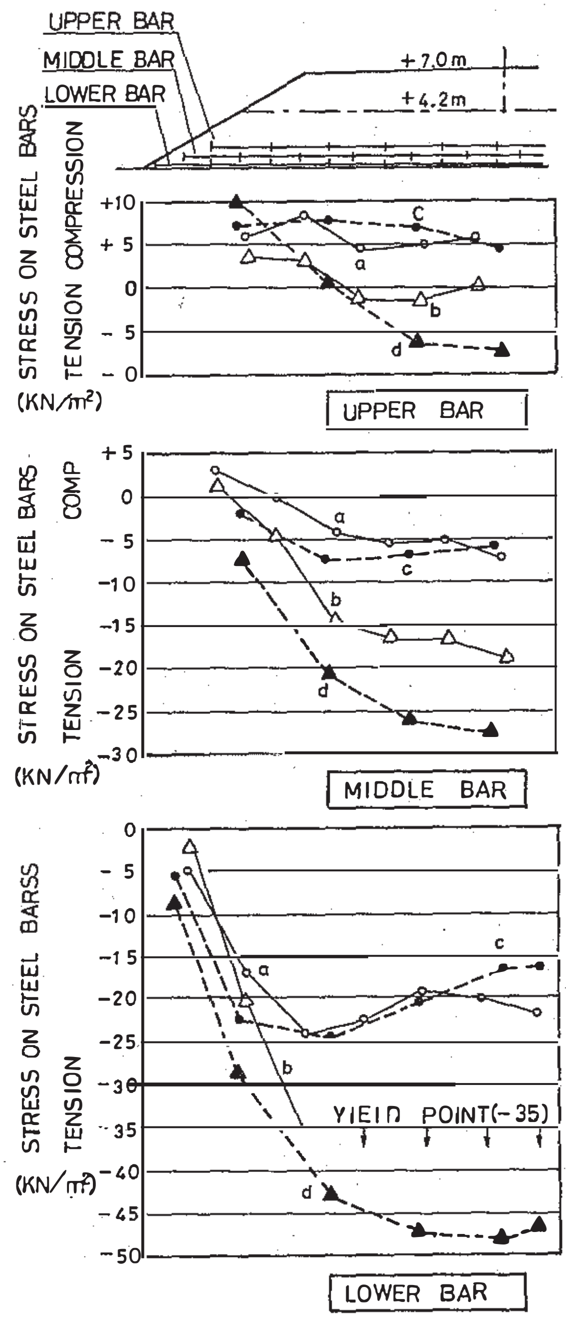
The results of calculation using different methods were given in the previous paragraph for design. Based on engineering judgement, three layers of deformed steel bars of 22mm in diameter were provided with horizontal spacing of 50cm. Allowable tensile strength per bar was 69.7 kN, and for 6 bars, 418 kN/m. Values predicted by FEM and by Ebetsu test embankment were 163 kN/m and 386 kN/m, respectively.

(1) Observed tensile forces on steel bars.
Figure 4 show the measured tensile stresses on steel bars. The total tensile force of steel bars at the center of the embankment was 426 kN/m, which is much larger than the predicted value by FEM(the first stage). Tensile forces of the bars in the lower layer were the largest among the tensile forces of bars in the three layers. The ratio of the tensile force on the lower layer to that on the middle layer was about 2. This ratio is similar to the result of FEM analysis. The tensile forces acting on the bars at the upper layer were negative when the height of the embankment was low, and positive when the height became high. Total tensile force can be obtained by conventional circular arc analysis, but this analysis cannot provide the proportion of tensile forces in each layer of the bars. It is necessary to use FEM for obtaining the tensile forces on bars in each layer.

(2) FEM analysis (the second stage).
The E_sA-value of a steel bar was assumed to be 25 MN/m at the first stage of prediction ((5) FEM analysis), but this assumption was not very correct. Therefore, considering relative displacement between an anchor plate and surrounding soil, the E_sA-value was assumed to be 130 MN/m (real E_sA-value of steel bars was 160 MN/m).

The observed values and predicted values (the second stage by FEM) were given in Fig. 5 at the embankment heights of 4.2m and 7.0m. Calculated tensile force agreed with the measured value at the bank height 4.2m, but there was little difference between them at an embankment height of 7.0m. Observed tensile stress on lower bar was higher than the yield value (35 kN/m^2). The steel bars were elongated at the yield stress without breaking, and they were serving as strengthening bars for the soil. It was felt that this condition should be improved in the future.

(3) Pressure on each plate.
It was very difficult to predict bearing forces acting on the anchor plates at the



a: BANK HEIGHT 4.2m 122 DAYS AFTER
b: BANK HEIGHT 7.0m 193 DAYS AFTER
c: PREDICTED BY FEM (BANK HEIGHT 4.2m)
d: PREDICTED BY FEM (BANK HEIGHT 7.0m)

Fig. 4 Stresses on steel bars

beginning. The bearing forces acting on the plates were calculated from the tensile forces on the tensile bars, and they are given in Fig. 5. The ratio of the pressure between middle plate and lower plate was 50%. As the result of pullout test, maximum bearing capacity of

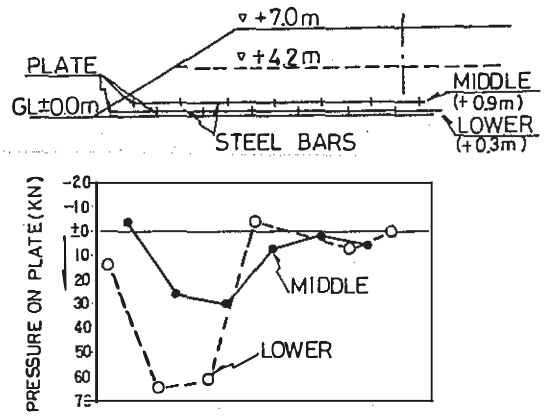


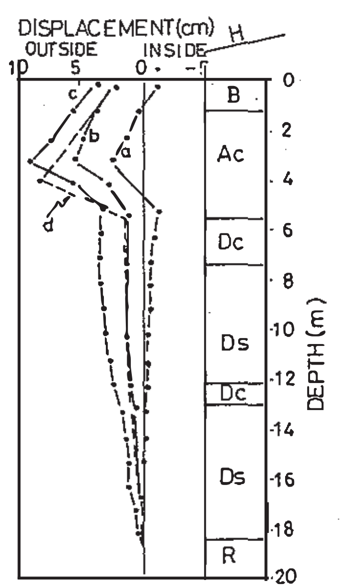
Fig. 5 Pressure on each plate

the anchor plate was 120 - 140 kN. Therefore, bearing forces acting on the plates were under allowable force.

(4) Settlement and displacement.

Observed settlement at the middle of the embankment with sand drains and steel bars reached 68cm at the end of construction and 86cm 406 days after completion. The observed value was 91% of the predicted value of 95cm.

Observed horizontal displacement (3cm) at the ground surface at the toe of the embankment and the predicted value (2cm) was nearly equal (Fig. 6).



Toe of slope of the embankment

a: BANK HEIGHT 4.2m 122 DAYS AFTER
b: BANK HEIGHT 7.0m
c: BANK HEIGHT 7.0m 193 DAYS AFTER
d: PREDICTED BY FEM (BANK HEIGHT 7.0m)

Fig. 6 Horizontal displacement

5 EMBANKMENT AT HAYASHIMA INTERCHANGE

The difference between the observed and predicted values of the Muchiki test embankment was investigated fully before constructing the embankment at the Hayashima Interchange. Points of improvement for the methods of prediction and design are stated as follows.

(1) Design.

The number of layers for the reinforcing bars, three for the Muchiki embankment was reduced to 2. Three layers were used at Muchiki because the prediction was thought to be not very accurate. Therefore, it was thought that breakage of the lower layer bars was quite possible. In that case, the upper layer bars could work instead of the lower layer bars. As a result of the embankment test, it was found that the lowest layer of bars yielded, but did not break. It was thought that the upper layer of bars could have been omitted by improving the methods of design. The ratio of the pressure between upper plate and lower plate was assumed to be 50%.

The dimension of the anchor plates was 225 x 225 x 9mm, which was smaller than Muchiki's plates. The dimension was decided based on an pullout test at the real construction site.

The diameter of steel bars was 29mm, which is larger than Muchiki's steel bars of 22mm in diameter. The diameter was decided based on allowable stress and facility of construction work.

The length of steel bars was decided based on circular arc analysis.

(2) Method of calculation by FEM.

Calculation by FEM was improved based on the tests on the Muchiki embankment. Another improvement was made at Hayashima. Considering the relative displacement between the anchor plate and the surrounding soil, a special joint element was adopted for the reinforcing bars. The constants for the joint element were provided by an pullout test at the site.

(3) Method of construction.

Total area, total volume of embankment, and maximum height of the Hayashima Interchange were 200,000 m^2, 1,200,000 m^3 and 11m, respectively. Depth of soft clay layer was about 10m. Vertical drains were installed under the middle of the embankment. Steel reinforcing bars with gravel compaction piles were placed under the slope of the embankment to strengthen the embankment and to prevent displacement of the ground surface near the toe of the embankment. Figure 7 shows the arrangement of reinforcing bars at Hayashima Interchange.

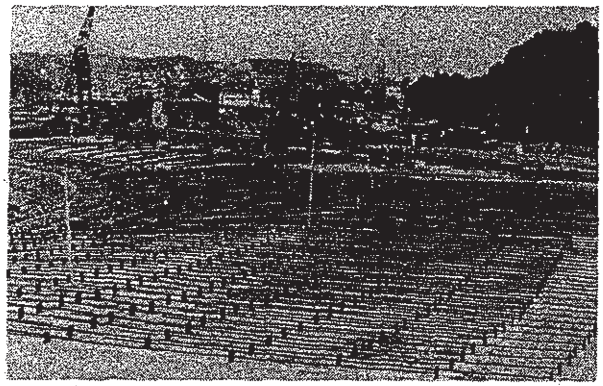


Fig. 7 Arrangement of reinforcing bars

(4) Prediction and performance.
Figure 8 represent the observed values and the predicted values by FEM.

Observed settlement and recorded stress on the upper bar were nearly equal to the predicted value. But recorded stress on the lower bar under the slope did not agree with the predicted one. Observed horizontal displacement at the toe of embankment was about 25% of predicted value. Therefore, steel reinforcing bars with gravel compaction piles were more effective than predicted.

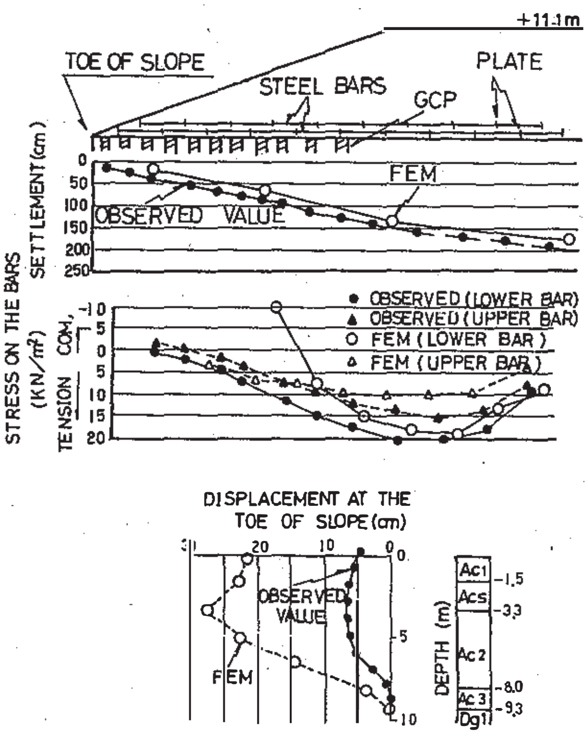


Fig. 8 Observed and predicted values by FEM at Hayashima Interchange

6 CONCLUSIONS

To strengthen the high embankment on soft ground of 10m in thickness, reinforcing steel bars with gravel compaction piles were used to prevent deformation at the toe of the embankment. This treatment was adopted on a small scale at Muchiki, and the predicted results were compared with the observed.

(1) FEM was used to analyze tensile forces acting on the reinforcing steel bars at the bottom of the embankment. Bar elements for the reinforcing bars were used.

(2) The EsA-value, which is the product of Young's modulus Es and cross sectional area A of the steel bars, was assumed to be 20 MN/m. When construction was half complete, this value was found to be too small, so it was raised to 130 MN/m. And calculated tensile force was nearly equal to the measured value.

(3) Number of layers, dimensions of anchor plates, and diameter of steel bars were changed based on the experiment at Muchiki.

(4) Joint elements were used in addition to bar elements for FEM analysis.

(5) The construction work proceeded safety without causing serious deformation at the ground surface near the toe of the embankment.

7 ACKNOWLEDGEMENT

This paper was prepared based on a report on the construction work by the Hayashima Interchange of the Honshu-Shikoku Bridge Authority. The authors express their sincere appreciation to the engineers of the Kurashiki Construction Office of the Authority and the committee members of Technology Center for National Land Development related to the construction work.

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