Geotechnical Aspects of Underground Construction in Soft Ground, Kusakabe, Fujita & Miyazaki (eds)
© 2000 Balkema, Rotterdam, ISBN 90 5809 1 066

Movement of stabilized coal-ash soil wall during excavation

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ABSTRACT: The amount of coal ash produced in Japan is expected reach to 7 million tons by the year 2000. It will be difficult to find the disposal location for this ash. Consequently, expanding the beneficial uses of the coal ash as a civil engineering material has become an important issue.EPDC has been developing the deep mixing soil stabilization method with blended slurry containing fly-ash(F)(coal ash),gypsum(G) and cement(C).(Hereafter FGC-DM).Self supported earth retaining wall from FGC-DM was built at the construction site of coal thermal power station. This earth retaining wall, which is one kind of the gravity retaining wall, is 12m high and contains a volume of improved soil of approx. 40,000m^3. Measurements were made to study the stability of the wall. This paper presents the results of the stability and design method of an earth retaining wall..

1 INTRODUCTION

The Tachibana Bay Thermal Power Station is a power station with a power generation output of 2.1 million kW currently under construction by the EPDC in Tokushima Prefecture. The FGC-DM method[1)] was employed in the civil works for the earth retaining walls at the pump site which belongs to the condenser cooling water channel for the power station.

This paper outlines the design and execution of the earth retaining walls based on the FGC-DM method, the movement of the wall is also reported.

2 OUTLINE OF WORKS

The base ground around the pump site is a rock bed with intensive surface irregularity, deployed in the shape of a horse-back. A soft clay layer of a thickness of 15 m is deposit on the rock bed . On top of this land-fill soil consisting of earth containing gravel (SPT N-value: approx. 10) is deposited. In the present work, the FGC-DM method was used to improve the ground to build the earth retaining walls for the excavation work in preparation to the structure construction.

In the FGC deep mixing method (FGC-DM), total slurry volume to be mixed with soil as a soil improvement material could be increased by adding high volume coal ash to cement. By this method, uniform soil improvement of low strength zone became possible which, in the usual cement deep mixing method, was difficult to mix uniformly because of too less slurry.

FGC-DM is applicable to the soil improvement of wide range of strength of soil from low up to high value.

Generally, cement only is used as the mixing method, in the present work, the necessity of increasing the cement content became known. This was because, according to a mixing test on the cohesive soil sampled at the area under study, it was impossible to secure the slurry injection ratio (ratio of slurry in 1m3 of the ground soil) above 20%, which is needed for uniform mixing, with the cement content that satisfies the strength requirement. To solve this problem, the adoption of the FGC-DM method, that will secure the slurry injection ratio by mixing coal ash instead of increasing cement, was decided. The standard section is shown in Figure 1.

3 IMPROVED SOIL AND MIXING DESIGN

3.1 *Strength design of improved soil*

The strength of the improved soil was set for the retaining wall on the basis of the structure design of exterior and interior stability calculation. The relationship between the allowable compression stress (σca) and the site strength (quf) of the improved soil is given by Eq.(1).

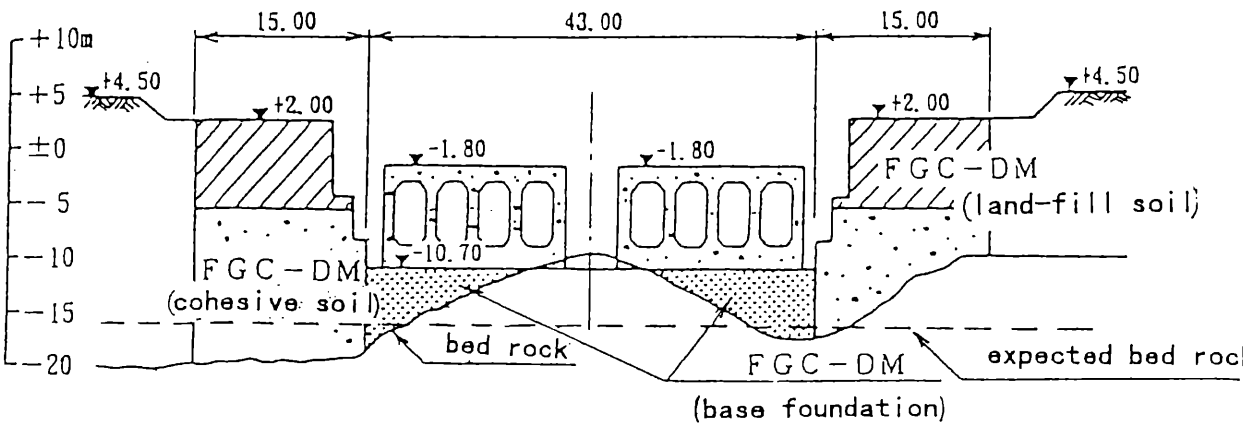

Figure 1. Standard section of pump pit.

$$\sigma ca = (1/F) \cdot \alpha \cdot \beta \cdot \gamma \cdot As \cdot quf. \quad (1)$$

where F = safety factor; $\alpha \cdot \beta$=reliability of lap section; γ =site strength coefficient; As = improvement ratio.

For the retaining wall and the excavation part to be improved, the site strength was decided as shown in Table 1.

In this study, the curing time corresponding to the design strength was assumed to be 56 days respectively, in consideration of the work stage at which the design load is applied to the work stage at which the retaining wall last excavation is completed.

3.2 *Determination of mixing*

The mixing design is shown in Table 2.

The W/F+C of 80% was adopted to start the execution work, eventually, however the W/F+C of 100% was adopted because the W/F+C of 80% also caused penetration failure. The reason for this penetration failure is considered to be a large increase in penetration resistance caused by the gravel soil which consists of grains larger than ordinary soil and which is lower in water content, resulting from the inclusion of rock debris formed in the course of the original reclaiming.

4 EXECUTION

4.1 *Outline of execution*

The data of execution are the number of piles: 1660, total length driven: 27,000m, improved soil volume: 40,000m^3, worked with three deep mixing machines.

As previously described, the execution was started with the mix proportion with a W/F+C of 80%. However at the progress rate of approximately 15%, the ratio was switched to 100% for the subsequent operation

Table 1. Improved Soil Strength

σca kN/m^2	F	$\alpha.\beta$	γ	As	quf (56) kN/m^2	quf (28) kN/m^2
392	1.5	0.8	0.6	0.9	1361	990

Table 2. Mixing Design Of FGC-DM.

			Fill Soil	Cohesive Soil
Site Mixing (Initial)	W/(F+C)	%	80	
	Cement	Kg/m^3	120	130
	Fly ash	Kg/m^3	60	85
Site Mixing (Final)	W/(F+C)	%	100	
	Cement	Kg/m^3	100	150
	Fly ash	Kg/m^3	50	75

※W/(F+C):Water-stabilizer ratio (%)

4.2 *End point treatment of bottom contact portion*

The geological study conducted prior to the start of the FGC-DM operation clarified that the supporting base ground was steeply inclined like a horseback, this being greatly different from the initial expectations, and necessitating the both side of retaining walls to be separated. A stability check calculation indicated that the retaining wall width required very large increase if the conditions of the coefficient of friction 0.6 and the safety factor of 1.2 were to be left unchanged. However, from the viewpoint of pursuing economical performance, the width of the retaining walls was redesigned, and the required safety factor reduced to 1.0, on the conditions that the bottom contact portion be fully improved to increase its sliding resistance and the behavior of the retaining wall accompanying the excavation be strictly monitored.

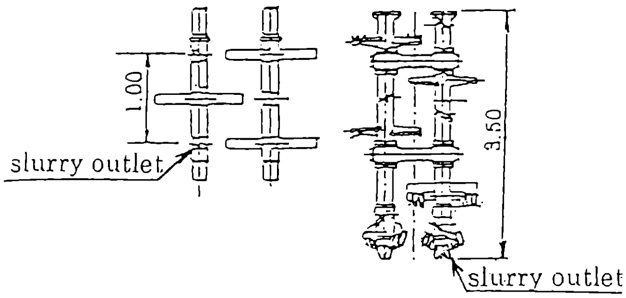

Figure 2. Improved Agitator Blades

Therefore, the careful treatment of the bottom contact portion of the end point became the key factor in the present execution work. For this reason, the end agitator blades were improved. (Figure 2)

Through these methods, the extensively weathered rock layer existed on the supporting base ground was improved, and at the same time, a favorable tight bonding with the weathered base rock (SPT N value above 50) was obtained.

4.3 *Strength characteristics of improved soil*

The unconfined compressive strength test results for the check boring are shown in Table 3.

The site strength coefficients are approx. 0.6, as initially set, regardless of the mix proportion, indicating the ordinary range deviation of the improved soil strength.

5 MOVEMENT OF STABILIZED COAL ASH SOIL WALL DURING EXCAVATION

5.1 *Outline of measurement*

The earth retaining cross section are shown in Figure 3.

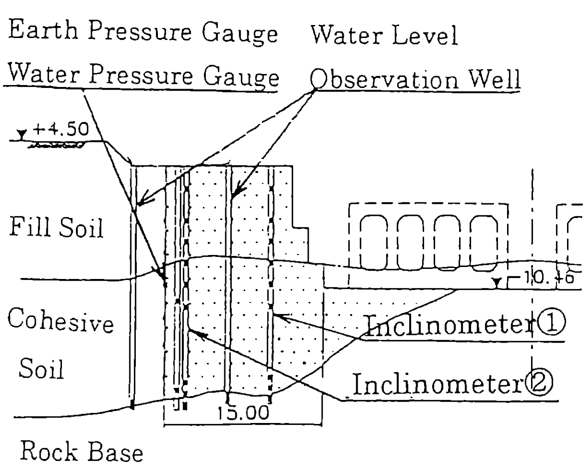

Figure 3. The earth retaining cross section

Table 3. Check Boring Results.

W/(F+C) (%)	Soil layer	Variation coefficient	Site strength coefficient (γ)	Mean strength quf (kN/m²)	No of specimens
80	Fill soil			2260	2
	Cohesive soil	0.21	0.65	2310	7
100	Fill soil	0.30	0.51	2680	9
	Cohesive soil	0.26	0.57	3220	16

The measuring items are the earth pressure gauge, water pressure gauge, inclinometer, the rock water level observation wells inside and outside of the wall and the slope horizontal displacement by survey.

5.2 *Measurement result*

5.2.1 *Working lateral pressure*

The lateral pressure working on the back side of the wall before the start and end of the excavation work is shown in Figure 4.

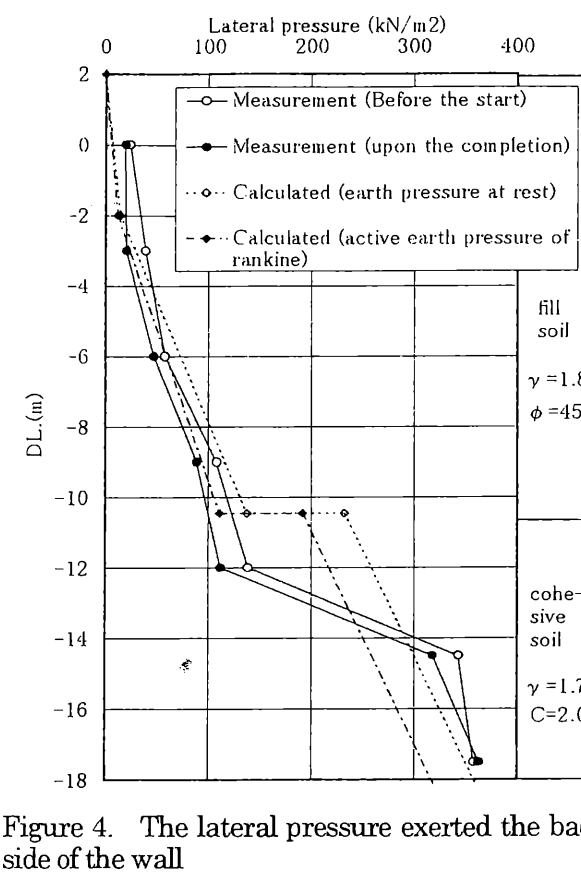

Figure 4. The lateral pressure exerted the back side of the wall

The lateral pressure of the fill soil is less than the design lateral pressure (calculated for ϕ =30° : ϕ =angle of internal friction). The coefficient of lateral pressure for the cohesive soil is equivalent to approx. 1.0. The lateral pressure excavation decreases with the progress of excavation, and decreases by 10 to 30 kN/m^2 when the excavation is completed.

In Figure 4, also the lateral pressure also as calculated approximate to the measured earth pressure (calculated with ϕ=45°, Ko =1−sinϕ, K_A=tan^2(45° − ϕ /2) for reclaimed soil, and Ko=K_A=1.0, c=19.6 kN/m^2 for cohesive soil) are entered alongside.

5.2.2 *Displacement*

The depth distribution of the displacement is shown in Figure 5.

From the Figure 5, no slip is seen at the bottom contact portion (in design, the wall width is determined by sliding displacement) as feared in the beginning. Overall displacement was also minute.

5.2.3 *Verification of retaining wall width*

Based on the measurement results and the box shear test of samples from the contact surface (C=124.5kN/m^2, ϕ=31.4°), the external stability and the internal stability were evaluated, and verification of the retaining wall width was attempted.

5.3 *Method of examination*

5.3.1 *Design method*

The external stability was studied with respect to sliding and overturning. The schematic cross section of the earth retaining wall by FGC-DM is presented in Figure 6. The deformation modulus and strengths of the treated soil by FGC-DM are extremely larger than that of untreated soils. Therefore, it is reasonable to be assumed that the treated soil mass behaves as a structure buried into the ground. For the external stability, it is examined whether the soil mass is stable against sliding, overturning and bearing capacity, as the same manner of the gravity retaining wall. The examination of the sliding is done by the following equation:

$$FS = (Pp + S) / Pa. \qquad (2)$$

where Pp and Pa are the total passive earth pressure acting on the excavated side and the total active earth pressure on the back side of the treated soil mass, respectively, and S is the total shearing resistance mobilized along the bottom of treated soil mass. Safety factor against tilting can be obtained by eq. (3).

$$FS = (Pp.lp + W.lw) / (Pa.la). \qquad (3)$$

where W is the weight of the treated soil mass, lw is the distance of the acting W from the excavated side, and lp and la are distances of acting Pp and Pa from the bottom of the treated soil mass, respectively.

The internal stability was studied with the following simplified processes (refer to Figure 7).

- Horizontal shear stress: $\tau_h = \lambda \cdot P_h/B \leqq \tau_a$
- Contact pressure: $t_1 \leqq \sigma_{ca}$
- Bending tensile stress:
  $\sigma_{bt} = W/B - 6M/B^2 \leqq \sigma_{ta}$
- Bending compressive stress:
  $\sigma_{bc} = W/B + 6M/B^2 \leqq \sigma_{ca}$

where σ_{ca} = allowable compressive stress, τ_a = allowable shear stress, σ_{ta} = allowable tensile stress, λ = ratio between max. shear stress and mean shear stress (=1.5), B = wall width, W = wall weight (buoyancy considered), M = bending moment

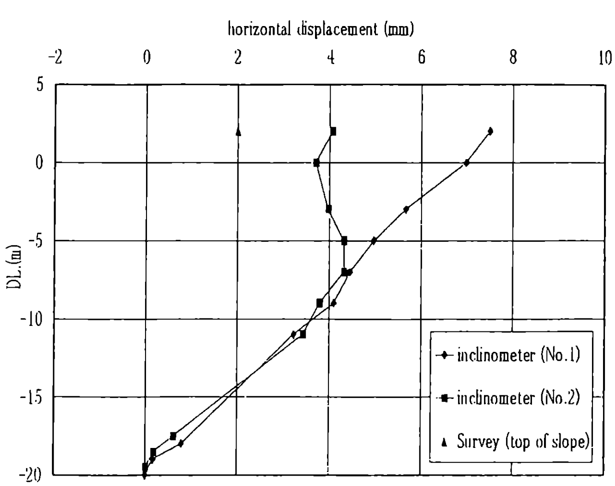

Figure 5. The depth distribution of the displacement

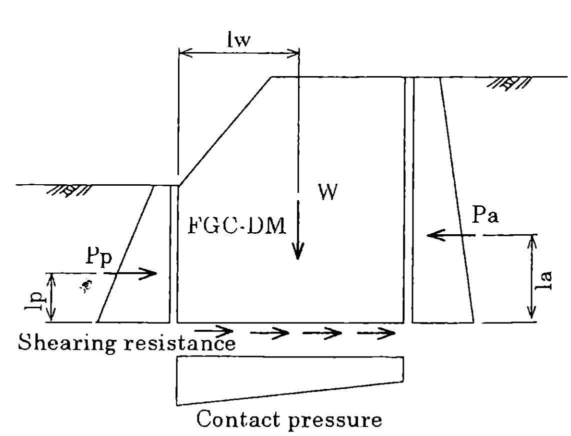

Figure 6. Application of external force

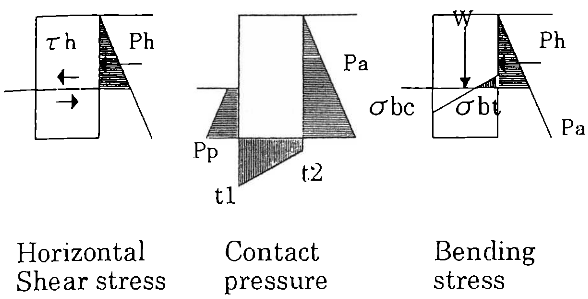

Figure 7. Design Method (simplified processes)

Four wall widths: 5.4 m, 7 m, 8.6 m, 10.2 m, 11.8 m, 13.4 m and 15 m were postulated. Note that the width of the actually executed wall was 15 m. In the verification, two allowable stress, as, one was the design value (τ_a=1/2 × σ_{ca}=196kN/m², σ_{ta}=0.15 × σ_{ca}=59 kN/m²) and the other being the site strength (mean quf≒3000 kN/m²), which is twice the design value were used.

5.3.2 *Setting of external force exerted to wall*

Based on the measurement result, the lateral earth pressure and uplift on the wall were set as shown in Figure 8.

The shear resistance of the wall bottom surface was calculated from the box shear test results (c=127 kN/m², ϕ=31°).

On the other hand, the improved soil seating on the inclined rock base on the excavation side was considered apart from the retaining structure.

Moreover, in the stability study, two methods, one considering only the ϕ (angle of internal friction) component of the wall bottom shear resistance force and the applied earth pressure (initial design way of consideration), and the other considering both c(cohesion) and ϕ, were used.

6 RESULTS OF EXAMINATION

6.1 *External stability*

The relationship between the sliding and overturning safety factor(Fs) and the wall width is shown in Figure 9. When only the ϕ component (coefficient of friction force μ= tanϕ= 0.61) was considered, the sliding Fs becomes 1.15. In the measurement results and observation of walls, however the working of adhesive force in practice is undeniable.

6.2 *Internal stability*

The relationship between the shear stress and the wall width is shown in Figure 10.

Note that with the bending compressive stress, it was represented as replaced by shear stress evaluation. With a 15 m wide wall, even when the design allowable stress was employed, the shear stress was less than the allowable value.

The relationship between the tensile stress and the wall width is shown in Figure 11. When the wall width is wider than around 10 m.

6.3 *Verification of wall width*

The necessary wall width and the governing factors for the respective evaluation conditions based on the above stability analyses are summarized in Table 4. From this, it was clarified that in the case of cohesion on the rock base and wall being taken into consideration, the wall width can be reduced to approximately 10 m, under these conditions, the internal stability, especially the tensile stress, becomes the governing factor.

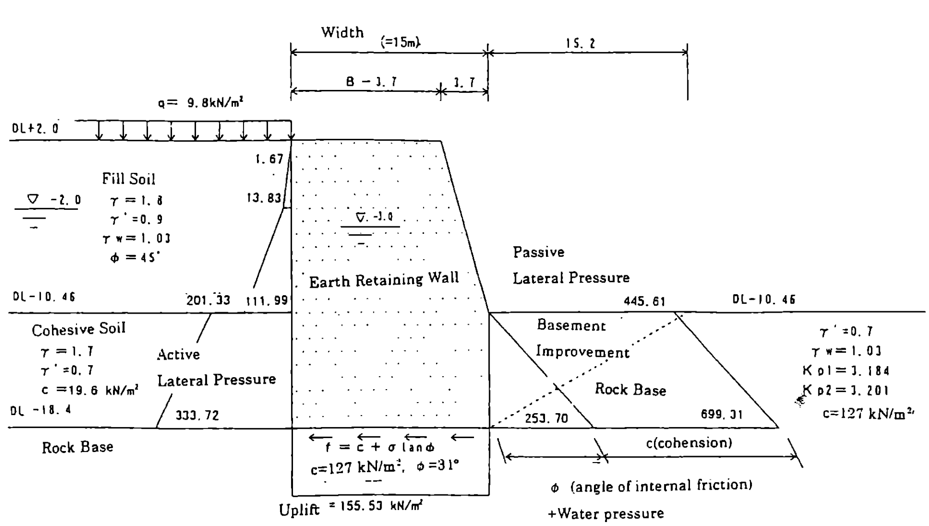

Figure 8. The lateral earth pressure and uplift on the wall

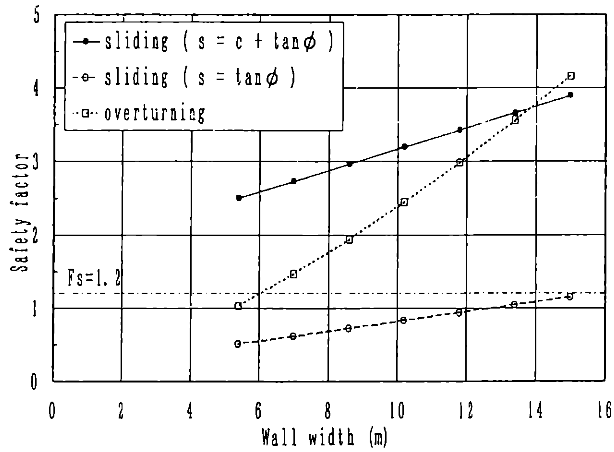

Figure 9. Relationship between the sliding and overturning safety factor (Fs) and the wall width

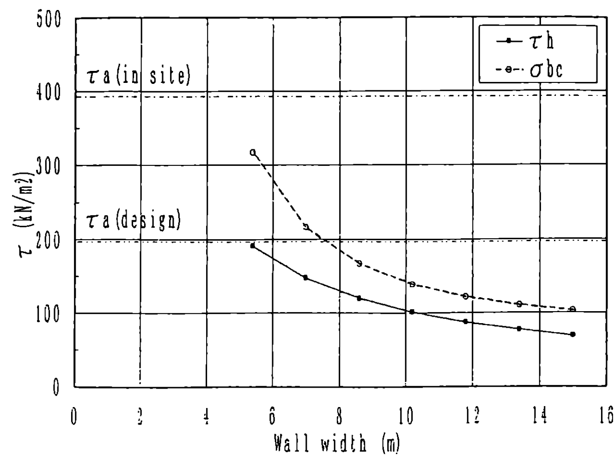

Figure 10. The relationship between the shear stress and wall width.

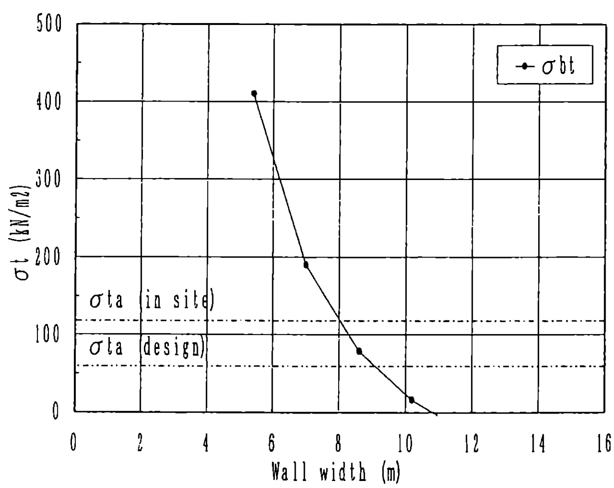

Figure 11. The relationship between the tensile stress and wall width.

Table 4. Wall width by stability analyses

Shear stress (contact rock base)	Wall width (design)	Wall width (in site)
s = σ tan φ (design)	> 15m 〔sliding〕	> 15m 〔sliding〕
s = c + σ tan φ	9m 〔σ bt〕	8m 〔σ t〕

※. 〔 〕: dependent factor

7. CONCLUSION

The retaining wall was constructed as the preparatory work for an unprecedented scale of excavation (excavation depth of approximately 15 m). As there were many design and construction problems due to the base ground having undulation, the behavior of the retaining wall during excavation had been measured. In addition, the wall thickness required for such retaining wall has been verified in reference to this measurement result as well as the result on the shearing test of the bottom contact portion.

REFERENCES

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