Case study on the low improvement ratio deep mixing method utilizing geosynthetics

Tsuyoshi Murashita & Masahiro Yoshikoshi
Takada River National Highway office, Hokuriku Regional Development Bureau, Ministry of Land, Infrastructure, Transport and Tourism, Japan

Masanori Ohtani & Toshiya Kaneko
KITAC Corporation*, Japan*

Mikio Kubo, Seiki Murakami & Yoshinao Kurachi
Eternal Preserve Ltd., Japan

ABSTRACT: The Takada River National Highway Office of Ministry of Land, Infrastructure, Transport and Tourism Hokuriku Regional Development Bureau is developing Joetsu Sanwa Road. This project involves the construction of embankments of up to 10m in height over a distance of 7km on a vast lowland flood plain. Because this site has cohesive strata extending to a depth of 60m, low improvement ratio deep mixing method utilizing geosynthetics was selected as the foundation improvement technique to ensure the stability of the road embankment and to control the deformation of the surrounding ground. To investigate the efficient ground improvement specifications, several test embankments with field observations (surface settlement gauges, displacement piles, differential settlement gauges, earth pressure gauges, strain gauges, and etc.) were constructed. Since conventional design methods tend to design very conservatively (very much on the safe side), it was decided to use finite element method (FEM) analysis in the design. By using the results of field observations, the soil parameters of the ground were obtained by back analyses in an elasto-plastic consolidation model. From the soil parameters, the optimum improvement rate, the length and the strength of the improved bodies that satisfy the design verification reference value were determined. The main reinforcement geosynthetics were laid in grids over the head of the improved bodies and their specifications were selected through the usage of 3D FEM analysis. This paper describes the economic specifications of the ground improvement technique and the geosynthetics based on the results of test embankments with field observations.

Keywords: geosynthetics, Low improvement ratio deep mixing method, measurement

1 INTRODUCTION

Joetsu Sanwa Road is a 7km length of road that is a part of local high standard highway. The ground survey and schematic design of the entire road were conducted from 2000 through 2005. In 2006-2008, the performance and the economic potential of the road were decided mainly on the optimal countermeasure against the deformation of the surrounding ground. Low improvement rate deep mixing method (DMM) with geosynthetics was selected after other methods such as surcharge with vertical drain method, DMM with surface stabilization method, stress intercepting method, vacuum consolidation method and lightweight banking method were compared. And in the same period, a detailed design of the road with countermeasure against soft ground was completed. From 2008 to 2011 test embankments with field observation were constructed. The final countermeasures were selected based on the results from the field observation and the embankment with that countermeasure is currently being constructed.

2 GROUND CHARACTERISTICS

Thick alluvium of 60m depth is dispersed on the foundation ground of the embankment. The alluvium was divided into three different layers, top layer (Ac1), middle layer (Ac2) and bottom layer (Ac3), based on their measured characteristic and recorded Standard Penetration Test N values. The soil properties of each stratum are summarized in Table 1.

Table 1. Soil properties

Strata	D from GL	γ (kN/m^3)	$W_{natural}$ (%)	W_L (%)	W_P (%)	C (kN/m^2)	C_c
Ac1	10-15m	16.5	57.1	86.1	32.8	1.5z+30.8	0.600
Ac2	20-28m	16.9	52.5	74.5	28.9		0.580
Ac3-1	36-42m	17.1	49.7	62.2	27.9		0.490
Ac3-2	55-60m	16.9	49	72.7	30.7		0.600
Bottommost		Mainly composed with sandy gravel. It is the bearing strata for important structures.					

Ac1 and Ac2 are very soft so there was a risk of slip failure from the embankment construction and 75% of the total settlement was predicted to occur in these strata. Ac3 had a high N value and its main feature was the presence of irregular sand layers. This project required the construction of embankments, 7 to 10m in height over the soil strata given in Table 1.

3 TEST EMBANKMENT (PART 1: 2008)

3.1 *Specification of the countermeasure*

The specification of the improved bodies were as follow: The design height of the embankment was 9.1m but in 2008, 6.0m was already constructed. Geosynthetics were laid for countering differential settlement between the improved bodies and the unimproved ground. The improved bodies consisted of dry jet mixing method (DJM), with a diameter of 1m and an improvement depth of 23.7m (down to the bottom of strata Ac2). The center to center spacing between the improved bodies was 2.3m and beneath the embankment slope the spacing was 2.5m. A high strength web-shaped geosynthetics was placed over the treated area.

3.2 *Field observation plan*

Field observations included surface settlement gauges and displacement piles (to record the differential settlement between the improved bodies and the unimproved ground and to verify the change of the surrounding ground and the stability of the embankment), differential settlement gauges (to record the settlement of each stratum), earth pressure gauges (to record the embankment load on the improved bodies and the unimproved ground). Figure 1 shows a schematic of the embankment and the locations of the monitoring instrumentation.

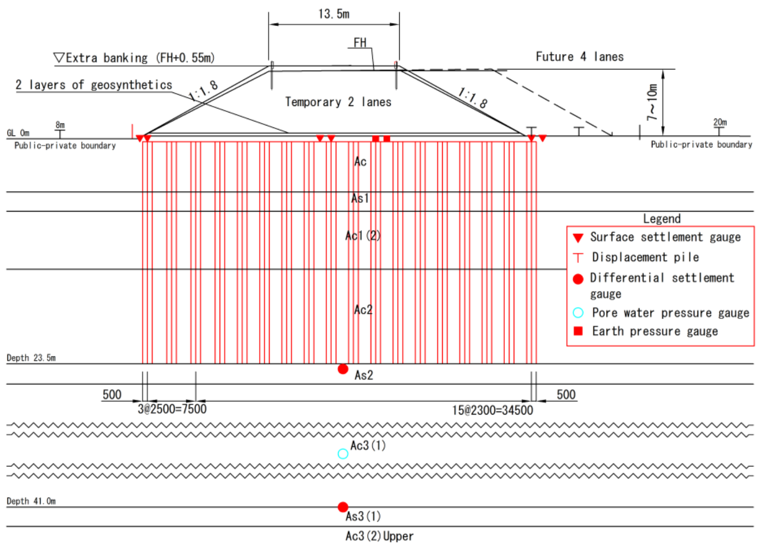

Figure 1. Shape of the embankment and gauge locations

3.3 *Field observation results*

In this section, the field observation results on settlement are described.

3.3.1 *Differential settlement between improved body and unimproved ground observed from surface settlement gauge*

The settlements observed at the top of the improved body and unimproved ground at the center of the embankment are shown in Figure 2.

At the center of the embankment, settlement at the unimproved ground was larger than the head the improved body. Considerably large settlements were observed and continued following the embankment construction, with differential settlement between the two bodies increasing for a period after construction. However, shortly after construction of the embankment was completed, settlements continued but the differential settlement remained largely constant.

3.3.2 *Results of consolidation settlement (differential settlement gauges)*

Figure 3 shows the consolidation settlement of the unimproved ground between improved bodies. The difference in settlement resulting from the embankment load at 0m depth and 23.5m depth (bottom of the improved body) was constant at about 9cm after the completion of the embankment (settlement of the layers with ground improvement converged soon after the completion of the embankment).

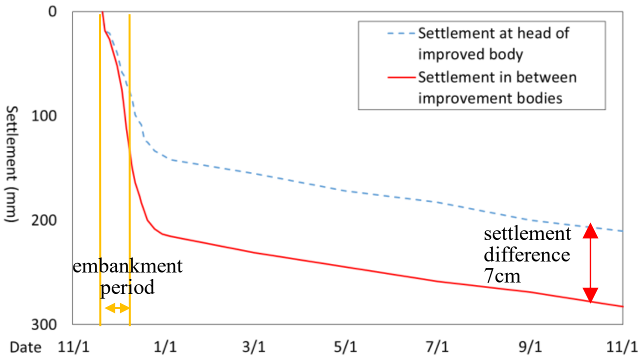

Figure 2. Surface settlement gauge

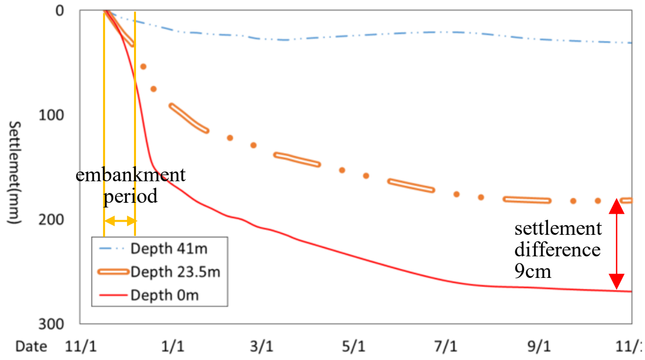

Figure 3. Consolidation settlement

4 TEST EMBANKMENT (PART 2: 2009 TO 2010)

4.1 *Specification of the countermeasure*

The specifications of the countermeasure were the same as Part 1:2008 and are summarized in Table 2.

Table 2. Specification of countermeasure

Station number	Embank-ment height (m)	Bodies spacing (m)	Bodies design strength q_{uck}(kN/m^2)	Body length L(m)	Geosynthetics strength (kN/m^2)
No. 98	8.5	2.3	1100	23.2	$\sigma_{1.5} \geqq 16.5$

4.2 *Field observation plan*

Field observation gauges were the same as Part 1, but strain gauges were also added to the geosynthetics, as shown in Figure 4.

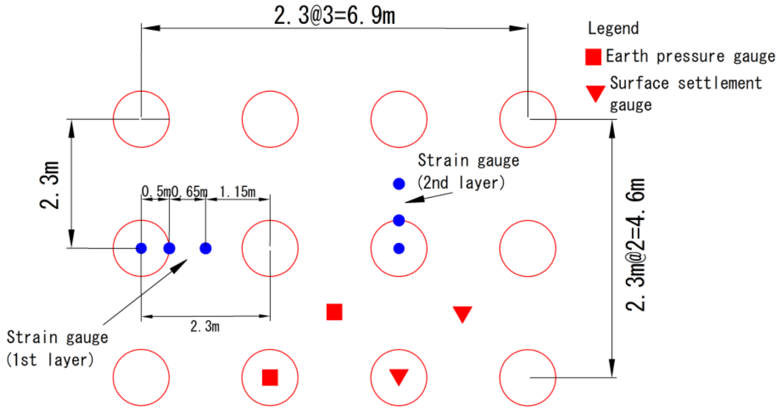

Figure 4. Locations of gauges

4.3 *Field observation results*

Results of the field observation and the respective design values are shown in Table 3.

Table 3. Observed values compared with design values

		Design values	Observed values
Settlement behavior	Improved bodies (cm)	9.0	11.3
	Ac3-1 (cm)	40.9	25.7
	Ac3-2 (cm)	20.8	3.8
	3years after its service (cm)	30.0	5.0
	Surrounding ground (cm)	3.8	App. 10
	Differential settlement (cm)	App. 10	6.7
Strain behavior	ε_{GTX} (%)	1.5	0.2(Body head)

Field observation results and the design results showed different behavior. Especially, the settlement of the Ac3-1 and Ac3-2 strata and the strain in the geosynthetics were smaller than the predicted design values. Therefore, it was concluded that the countermeasures needed revision to improve the performance of the embankment.

4.4 *Revision of specification of the countermeasure*

4.4.1 *Revision of design method*

For the conventional design, the area with the improved bodies was modeled as composite ground with traditional slip failure analysis and settlement analysis conducted separately. This approach resulted in a conservative design.

The revised analysis, a method that accurately simulates the behavior of the improved body and unimproved ground, used an elasto-plastic consolidation model. The optimal improved body was calculated based on the elasto-plastic consolidation back analyzed from the ground behavior data retrieved from the field observation during and after the embankment construction, Figure 5. The differential settlement between the improved body and the unimproved ground reduced due to the presence of the geosynthetics was analyzed using 3D finite element method analysis. The FEM analysis involved a quarter symmetrical model of an unimproved ground surrounded by 4 improved bodies, Figure 6. An example of the analysis is shown in Figure 7.

4.4.2 *Revision of countermeasure*

The specifications of the countermeasure were revised as follows:

i. Improved body diameters of 1m (current design, powder high pressure injection mixing method) and 2m (slurry mechanical mixing method) were selected. The improved body with 2m diameter was selected as this reduced the volume of cement required to improve the ground.

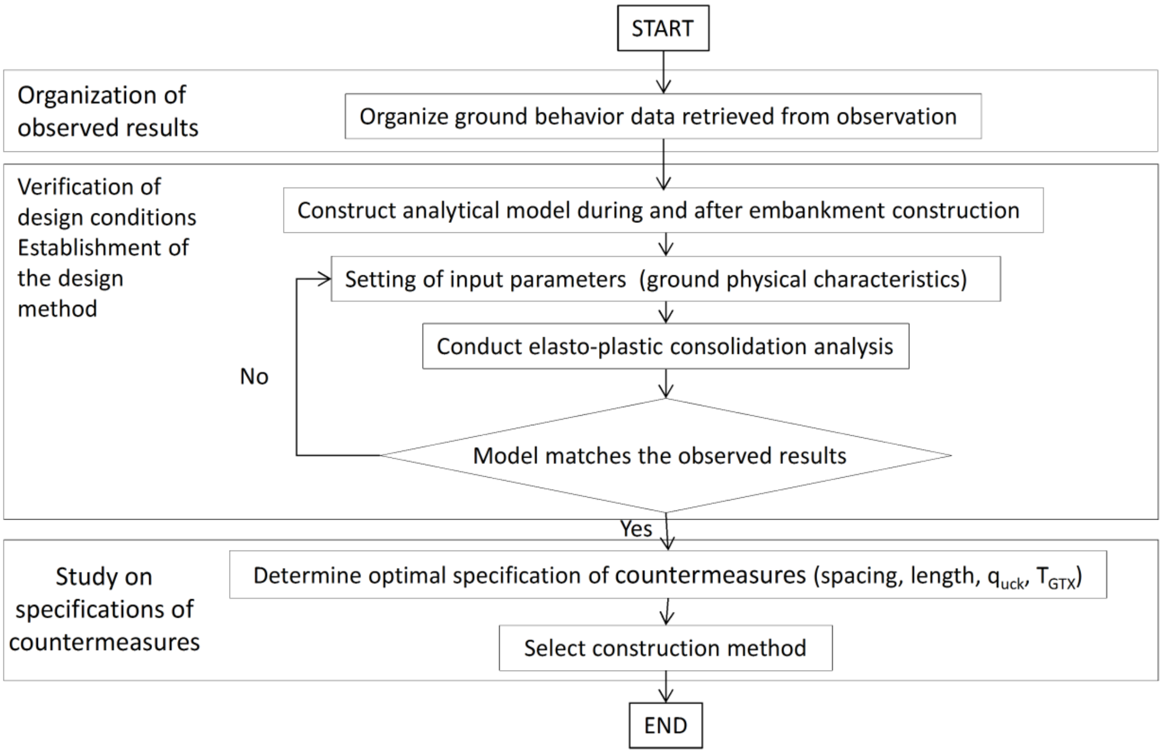

Figure 5. Flowchart

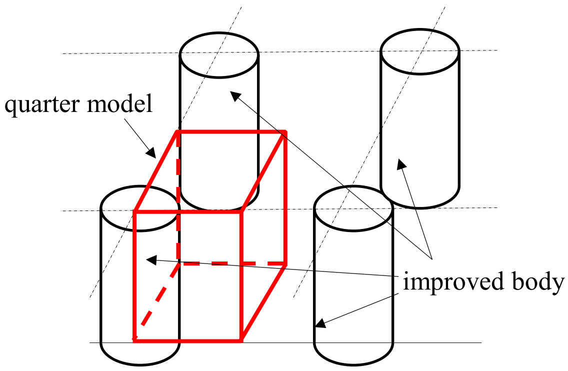

Figure 6. Quarter symmetrical model

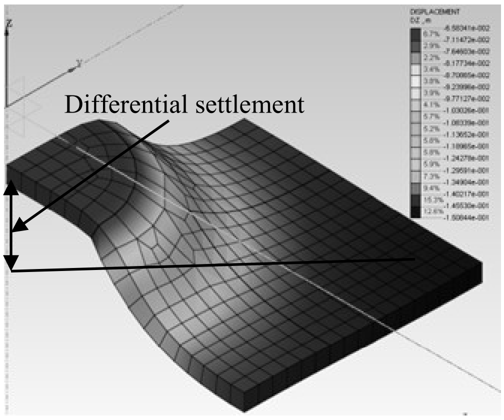

Figure 7. Example of FEM analysis

ii. The design standard strength was determined based on the ability of the mixing machine. The laboratory mixing test results found the maximum strengths to be quck=1,500kN/m^2 for the 1m diameter (powder injection mixing method) and quck=800kN/m^2 for the 2m diameter bodies (slurry mechanical mixing method).

iii. The spacing between the improved bodies was increased until the stress exerted on the body was almost equal to the design strength.

iv. Because the settlement of Ac3 layer was small, the ground improvement targeted strata Ac1 and Ac2. The length of the improved body was selected so that the residual differential settlement after the start of its service and deformation at the public-private boundary fall between the allowable levels.

v. For a conventional design, two layers of high strength geosynthetics were laid (transversely and longitudinally) over the entire area of the embankment. In this design, high strength geosynthetics (primary) were laid directly over and between the improved bodies, forming a large grid pattern (the shaded area of Figure 8 and Photo 1) and low strength geosynthetics (secondary) were laid 30cm above the primary layer over the entire area of the embankment. In the UK, the fact that the stress concentrates on the grids is known and the first grid shaped placement (primary and secondary reinforcement) was developed in 2000. According to the test embankment, strain generated in the geosynthetics and soil pressure gauge results showed that the stress is low at the center of the unimproved area.

Photo1. Example of geosynthetics laid in grids

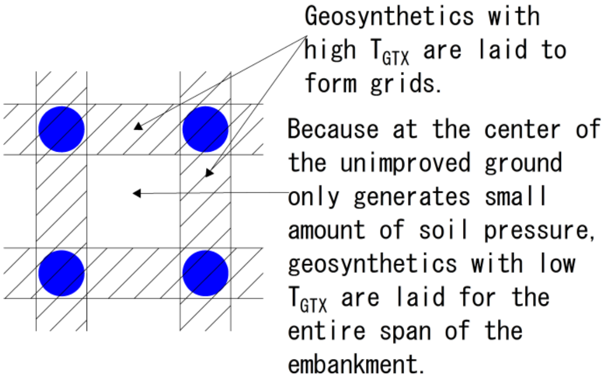

Figure 8. Geosynthetics laid in grids

5 TEST EMBANKMENT (PART 3: 2011)

5.1 *Specification of the countermeasure*

The specifications of the countermeasure determined from the test embankment (Part 2: 2009 to 2010) results are summarized in Table 4.

Table 4. Specification of countermeasures

Station number		No.120	No.129
Embankment	Height (m)	7.4	7.4
	L (m)	60.0	60.0
Improved body	D (m)	2.0	1.0
	L (m)	17.0	17.0
	Spacing (m)	4.0	3.0
	q_{uck} (kN/m^2)	650	1,400
GTX	Primary	$\sigma_{0.6} \geqq 6.0$kN/m	$\sigma_{1.7} \geqq 17.0$kN/m
	Secondary	$\sigma_{0.6} \geqq 0.6$kN/m	$\sigma_{1.7} \geqq 1.7$kN/m

5.2 *Field observation results*

The gauges for the field observation were the same as the test embankment (Part2: 2009 to 2010), and the field observation results and the respective design values are shown in Table 5.

Table 5. Observed values compared with design values

Station number		No. 120 (d=2m)		No.129 (d=1m)	
		Design	Observed	Design	Observed
Residual settlement after 3 years of service (cm)		8.1	8.7	11.3	6.9
Deformation at site boundary (cm)		9.2	9.3	10.7	10.6
Differential settlement (cm)		10	8 to 18	10	12 to 24
σ on center of body head (kN/m^2)		270	290	270	260
ε_{GTX}(%)	Primary	0.6	0.8	1.7	1.7
	Secondary	0.6	2.5	1.7	2.5

Because the differential settlement between the improved body and the unimproved ground and the strain in the geosynthetics exceeded the design values, the cause and the safeness of the embankment were analyzed. The strain in the geosynthetics was small at the center of the improved bodies and at the space between the improved bodies, but at the edge of the improved body, large strain was generated. This behavior was also observed by Russel & Pierpoint (1997).

For the initial design value, the embankment material was assumed to be silty sand but the actual material was cohesive soil thus the soil parameters were different. In addition, the first 1m beneath the ground, the cohesive soil was extremely weak and the soil parameters were lower than expected.

When the geosynthetic strain and differential settlement analyses were revised using the new soil parameters, the analyzed results were about the same as the observed values. Therefore, the observed results exceeded the design values because the soil characteristics were different to those assumed in the initial analysis.

Surveying of the embankment surface and the durability of the geosynthetics were checked, even with a 20cm of differential settlement, there was no problem with the safety of the embankment.

5.3 *Selecting the optimum countermeasures*

From the results of the test embankment the required level of safety and economic efficiency, the optimal improved body design was selected

Table 6 compares the safeness of the improved bodies used for the test embankments. Improved bodies with 2m diameter (Case I) and 1m diameter (Case II) acquired stability, but Case I reduced differential settlement and deformation at the public-private boundary than Case II.

Table 6. Stability verification

		Case I		Case II	
Improved bodies Specification		· d=2m · Spacing=4m · L=17m · q_{uck}=650kN/m^2	Score	· d=1m · Spacing=3m · L=17m · q_{uck}=1400kN/m^2	Score
Stability of embankment		Embankment was stable during and post construction	A	Embankment was stable during and post construction, but the controlled values were close to the unstable zone	B
Stability of body	Lateral displacement	Body heads displaced about 1mm. Bodies were in healthy condition.	A	Body heads displaced about 5mm. Bodies were in healthy condition.	A
	q_{uck}	Soil pressure of 490kN/m^2 was exerted on the bodies.	A	Soil pressure of 1,050kN/m^2 was exerted on the bodies.	A
Stability of GTX	Differential settlement	Differential settlement of 8 to 18cm was observed. This settlement did not occur at the embankment top.	A	Differential settlement was 12 to 24cm. This settlement did not occur at the embankment top.	B
	ε_{GTX}	ε_{max}=0.8% for strip reinforcement ε_{max}=2.5% for planar reinforcement. Rupture of GTX is not a problem.	A	ε_{max}=1.7% for strip reinforcement ε_{max}=2.5% for planar reinforcement. Rupture of GTX is not a problem.	A
Validity of body length	Overall settlement of embankment	Residual settlement of 9cm after 3 years of service.	A	Residual settlement of 7cm after 3 years of service.	A
	Settlement of surrounding ground	Final settlement of public-private boundary was about 9cm.	A	Final settlement of public-private boundary was about was 11cm.	B
Others		From the lab mixing test, maximum q_{uck} is 1,300kN/m^2. Soil removed due to improved bodies could be used as fill materials. Requires supplying of water.	A	From the lab mixing test, maximum q_{uck} is 1,400kN/m^2. Soil removed due to improved bodies could not be used as fill materials, unless it is conditioned by adding cement. Because it uses compressed air during the construction, air could flush from the surrounding.	B
Overall score		A		B	

To compare the economic efficiency of the construction cost, total cost for each embankment height of this area was calculated and Case I was cheaper than Case II. In addition, as the embankment height increased, the construction cost for Case I became much cheaper than Case II.

6 CONCLUSION

From the results of the test embankments, the optimal countermeasure specification was an improved body with a diameter of 2m and a spacing of 4m constructed by a slurry mechanical mixing machine (low improvement ratio cement deep mixing method (CDM)).

For the unconstructed area beyond 2012, low improvement CDM and grid geosynthetics basal reinforcement became the standard countermeasure and the length and strength of the improved bodies were adjusted according to the ground and embankment conditions for constructing the embankment.

In addition, the allowable differential settlement was changed to 20cm to facilitate further cost reduction.

REFERENCES

Kubo, M. & Okochi, Y. 2011. A rational countermeasure against differential settlement of low improvement ratio deep mixing method with utilizing geosynthetics. Proc. of Geotec Hanoi,pp.91-98.

Mizutori, K.,Kubo, M., Matsumoto, M. & Okochi, Y. 2010. Study of geosynthetics modeling for 3D FEM analysis. Japan Civil Engineering Society 65th annual conference.pp.573-574.(in Japanese)

Russel, D. & Pierpoint, N. 1997. A numerical investigation of the behavior of piled embankments, Ground Engineering , Thomas Telford..

Russel, D., Naughton, P., & Kempton, G. 2003. A new design procedure for piled embankments. Proc. of 56th Annual Canadian Geotech. Conf. CD-ROM.

Tsuchida, M., Watanabe, I. & Anayama, H. 2009. Effect of high embankment over soft ground on the surrounding ground. Hokuriku Regional Development Bureau Heisei 21-year project research presentation. (in Japanese)

Umemoto, H. & Wakimoto, N. 2011. Cost reduction of countermeasure against soft ground for Joetsu Sanwa Road. Hokuriku Regional Development Bureau Heisei 23-year project research presentation. (in Japanese)

Urushiyama H., Umemoto, H & Ohira, H. 2012. Cost reduction using the results of Joetsu Sanwa Road soft ground countermeasure. Hokuriku Regional Development Bureau Heisei 24-year project research presentation. (in Japanese)