TECHNICAL NOTE

FRACTURING PRESSURE OF SOIL GROUND BY
VISCOUS MATERIALS

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ABSTRACT

This paper investigates the fracturing phenomena in sandy as well as cohesive soil caused by injecting viscous materials such as the mortar. The fracturing pressure generally increases as the flow value F of injection material increases. In cohesive soils, the fracturing pressure P_f is equal to the cavity expansion pressure P_u (Vesic, 1972) of cohesive soils, irrespective of the value of F when F is more than 30 sec. This is because it is more difficult to mobilize the wedge action as the viscosity of injection materials increases. On the other hand, in sandy soils, the effect of flow value F on the fracturing pressure P_f becomes smaller as the permeability of sand increases. The fracturing pressure P_f' of sand of high permeability is equal to the cavity expansion pressure P_u' of sand soil, irrespective of the value of F .

Key words: backfill, cohesive soil, fracturing pressure, grouting, high viscosity, sandy soil, soil stabilization (IGC: K6/H5/D0)

INTRODUCTION

The local increase of pore water pressure in soil ground generates the failure of ground. This is so-called hydraulic fracturing. In geotechnical engineering, the fracturing occurs in in-situ permeability tests (Bjerrum et al., 1972) and in chemical grouting (Morgenstern and Vaughan, 1963; Karol, 1983). And also the failure of dam-core by seepage pressure (Vaughan, 1971) or the spouting of water or sand in liquefaction (JSSMFE, 1982) is probably resulted from the hydrofracturing. How-

ever, the injection material that gives rise to the fracturing of ground is not necessarily the low viscous liquid such as water or chemical grouts. In the case of backfill grouting to prevent the settlement of ground in construction of shield-tunnel, and in that of the injection for the anchor to fix to the ground, the viscous material, for example the mortar or the cement milk the viscosity of which is much larger than water, is used and the fracturing by excess injection pressure may occur. It is important to clarify the fracturing pressure and the mechanism of fracture initiation by viscous

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materials, because the fracturing brings about not only the loss of injection materials but also the damage such as the ground heaving.

Although few experimental study about fracturing of soil ground had reported, Jaworski had pointed out that the hydrofracturing pressure P_f of compacted soil by water was given by Eq. (1) (Jaworski et al., 1981).

$$P_f = m\sigma_3 + \sigma_{ta} \quad (1)$$

P_f : hydrofracturing pressure

m : constant ($1 \leq m \leq 2$)

σ_3 : confining pressure

σ_{ta} : apparent tensile strength

However, the physical meaning of σ_{ta} as well as the relation between the value of σ_{ta} and the soil type is unknown.

The authors had investigated the hydrofracturing phenomena of soils by low viscous liquid such as water or chemical grouts. The fracturing in soils is generated by the wedge action of low viscous liquid which penetrates into the crack around the borehole. In cohesive soil hydrofracturing pressure is given by Eq. (2). Eq. (2) is equal to Eq. (1) if $m=1$ and moreover if $\sigma_{ta} = q_u$ (Mori and Tamura, 1987a).

$$P_f = \sigma_{min} + q_u \quad (2)$$

σ_{min} : minimum principal stress

q_u : unconfined compressive strength

On the other hand, the hydrofracturing pressure of sandy soil is given by Eq. (3) (Mori and Tamura, 1987b, c).

$$P_f' = m\sigma_3' + \sigma_t + R \quad (3)$$

P_f' : hydrofracturing pressure $= (P_f - u_0)$

u_0 : initial pore water pressure)

m : constant ($1 \leq m \leq 2$)

σ_3' : effective confining pressure $(= \sigma_3 - u_0)$

σ_t : tensile strength of soil

R : resistance of crack progression

The resistance of crack progression R deeply depends upon the permeability of sand. The value of R increases as the permeability becomes large. And it is $1 \sim 2 \text{ kgf/cm}^2$ ($98 \sim 196 \text{ kN/m}^2$) in the range of $10^{-4} \sim 10^{-3} \text{ cm/sec}$ of the coefficient of permeability k . In addition, the sum of σ_t and R is equal to σ_{ta} in Eq. (1).

Although Eqs. (2) and (3) are the experimental equations using the water and chemical grouts, the fracturing phenomenon by the viscous materials such as the mortar is hardly known. Generally speaking, it is more difficult to mobilize the wedge action as the viscosity of injection material increases.

This paper investigates the relation between the fracturing pressure and the viscosity of injection materials, and we clarified the effect of soil type, for example the permeability, the strength, and the confining pressure, on the fracturing pressure. In addition, the effect of the pressurizing method on the fracturing pressure is investigated, too.

EXPERIMENTAL PROCEDURE

Cohesive Soils

Fig. 1 shows the schematic diagram of triaxial apparatus for the fracturing tests of cohesive soils. The shape of specimen was the hollow cylinder. The height and the diameter were 180 mm respectively. The diameter of borehole was 13 mm in all cases (cf. Fig. 2).

After the borehole was filled with the injection materials, the plasticine was packet at

1. Axial pressure
2. Confining pressure
3. Pressurization
4. Injection pressure
5. Specimen (180 × 180 mm)
6. Borehole ($\phi = 13 \text{ mm}$)

Fig. 1. Schematic diagram of triaxial apparatus for cohesive soils

Fig. 2. Shapes of specimen and under-plate

the upper end of borehole to prevent the leakage of injection materials. The impermeable grease layer was formed at both ends of specimen. To prevent the borehole from broken by the confining pressure under the borehole pressure being zero, the confining pressure was applied stepwise at the same time with the borehole pressure. During the fracturing tests, the confining pressure was kept constant and only the borehole pressure gradually increased.

The facturing pressure was decided from the change of borehole pressure during injection. The sudden decrease of borehole pressure, measured by the pressure meter shown in Fig. 1, was accompanied with the initiation of fracturing.

The pressurization of borehole was done by the manual pump as shown in Fig. 3 in ordinary cases. However, the fracturing tests with the constant injection rate using the electric pump were performed in order to clarify the effect

1. Handle 2. Piston
3. Injection material
4. Valve

Fig. 3. Schematic diagram of manual pump

of pressurizing method on the fracturing pressure, too.

The pressurising rate P_f of the manual pump was $0.02 \text{ kgf/cm}^2/\text{sec}$ ($1.96 \text{ kN/m}^2/\text{sec}$). The constant injection rate q of the electric pump was 3.00 l/min .

Table 1 shows the mechanical properties of test samples. The tensile strength σ_t was determined with Brazilian tests.

All samples were the artificially prepared soils composed of Kibushi clay, gypsum, bentonite, sodium tetraborate, and water. The sodium tetraborate was used to delay the hardening by gypsum. After they were mixed well in the desired proportion with the soil-mixer, the soft mixed soils were packed into the mold the diameter of which was 180 mm. In the bottom of mold, the steel pipe the diameter of which was 13 mm was stood up.

Table 1. Mechanical properties of cohesive soils

Sample	Combination (g)	q_u (kgf/cm ²)	σ_t (kgf/cm ²)	E_{50} (kgf/cm ²)
C1	Kibushi-clay 3850, gypsum 1550 bentonite 500, water 2900	0.8~1.2	0.20~0.25	80~120
C2	Kibushi-clay 4100, gypsum 1350 bentonite 500, water 2900	0.4~0.8	0.12~0.15	35~60
C3	Kibushi-clay 4300, gypsum 1200 bentonite 500, water 2900	0.0~0.4	—	25~35

$$1 \text{ kgf/cm}^2 = 98.1 \text{ kN/m}^2$$

q_u : unconfined compressive strength
 σ_t : tensile strength
 E_{50} : Young's modulus

And then the sample was covered by the vinyl sheet to prevent the desiccation and was left alone for 20 hr. The borehole was formed by pulling out the steel pipe from the mold.

Sandy Soils

Fig. 4 shows the schematic diagram of triaxial apparatus for the fracturing tests of sandy

1. Axial pressure
2. Confining pressure
3. Back pressure
4. Pressurization
5. Borehole pressure
6. Injection material
7. Specimen
8. Injection pipe

Fig. 4. Schematic diagram of triaxial apparatus for sandy soils

soils. The air-dried sand was thrown into the molding box, in which the injection pipe was stood up on the bottom board, and was compacted to obtain the desired density in the apparatus. The diameter of molding box was 180 mm. A lot of small openings were formed in the lateral side of injection pipe and the steel mesh was covered around the pipe to prevent the sand from getting into the pipe. After the negative pressure was supplied to the specimen by the vacuum pump, the confining pressure was given and then the specimen was saturated with water. The initial pore water pressure in the fracturing tests was almost zero.

The pressurization of borehole pressure was done by the manual pump shown in Fig. 3.

The pressurising rate P_f was $0.02 \text{ kgf/cm}^2/\text{sec}$ ($1.96 \text{ kN/m}^2/\text{sec}$) in all cases.

Table 2 shows the permeability and frictional angle of sands. The types of sands were Keisa-5, Keisa-7, and the mixed sand composed of Keisa-7 and Kibushi clay. Fig. 5 shows the grain size distribution curves of samples.

Injection Materials

Table 3 shows the viscosity of injection

Table 2. Mechanical properties of sandy soils

sample	sand type	$\gamma_d(\text{g/cm}^3)$	$k(\text{cm/sec})$	$\phi'(^{\circ})$	$E(\text{kgf/cm}^2)$
S1	Keisa-5	1.50	6.2×10^{-2}	38	250
S2	Keisa-7	1.40	6.3×10^{-3}	30	250
S3	Mixed sand	1.52	8.2×10^{-4}	30	150

$$1 \text{ kgf/cm}^2 = 98.1 \text{ kN/m}^2$$

mixed sand composed of Keisa-7 and Kibushi-clay (8:2).

γ_d : density

k : coefficient of permeability

ϕ' : frictional angle

E : Young's modulus

Fig. 5. Grain size distribution curves

Table 3. Types of injection materials

flow value F (sec)	combination
8~10	only water
15~20	10%-bentonite suspension 2500(cc) cement 250(g), delayer 200(cc)
25~30	10%-bentonite suspension 2500(cc) cement 700(g), delayer 200(cc)
60~	10%-bentonite suspension 2500(cc) cement 1100(g), delayer 200(cc)

Flow value F was measured by P -funnel test.

materials. The viscosity was expressed by the flow value F . F is measured by P -funnel method (JSCE, 1986).

These materials were colored by Rhodamine-B in the case of sandy as well as cohesive soils. After the tests, the specimen was cut down and the direction of fracture was investigated.

FRACTURING PRESSURE AND FLOW VALUE

Cohesive Soils

Fig. 6 shows the effect of flow value F on the fracturing pressure P_f in cases of sample C1, C2 and C3. The confining pressure σ_3 was 1.00 kgf/cm² (98.1 kN/m²) in all cases. On the whole, the fracturing pressure P_f increases as the flow value F becomes large. However, it is found from Fig. 6 that the critical value of P_f exists, i. e. P_f does not exceed the value.

Fig. 6. Effect of flow value F on hydrofracturing pressure P_f

In case of water, the flow value F is 8 sec and from Fig. 6 in all cases $P_f = \sigma_3 + q_u$. This equation corresponds to Eq. (2). And so Eq. (2) can be applied to the test results.

And the increment of P_f to flow value F becomes larger as the unconfined compressive strength q_u of cohesive soil becomes larger.

The direction of fracture was vertical and radial from the borehole in all cases and the specimen was divided into two parts perfectly.

Sandy Soils

Fig. 7 shows the effect of flow value F on the fracturing pressure $P_f' (= P_f - u_0)$ (u_0 : initial pore water pressure). The effective confining pressure σ_3' was 0.5 kgf/cm² (49.1 kN/m²) in all cases. The fracturing pressure P_f' of sample S3, the permeability of which is the smallest increases as the flow value F becomes large in the same way of cohesive soils.

Fig. 7. Effect of flow value F on hydrofracturing pressure P_f'

On the other hand, in sample S1 and S2 permeability of which are larger than sample S3, the effect of flow value F on P_f' is small. And it becomes more difficult to confirm distinctly the fracture surface as the permeability of sand increases. Especially, in sample S1, permeability of which is larger than 10^{-2} cm/s, the fracturing pressure P_f' does not depend upon the flow value F at all. In this case, the fracture surface was not seen and the injection materials seemed to permeate spherically from the borehole into specimen. This was probably because the general failure of sand around the borehole was initiated.

EFFECT OF INJECTION METHOD ON FRACTURING PRESSURE

Although all test results mentioned before were obtained using the manual pump, the fracturing pressure may be influenced by the pressurizing method. So the results mentioned above was compared to the results in the con-

stant injection rate using the electric pump. The injection rate was 3.0 l/min.

Fig. 8 shows the relation between the fracturing pressure P_f and the pressurizing method. The injection material was the mortar the flow value F of which was larger than 60 sec. In the case of constant injection rate, it is difficult to determine the value of P_f because the injection pressure rapidly increases immediately after the injection as shown in Fig. 9. But it is found from Figs. 8 and 9, P_f is hardly influenced by the pressurizing method.

MECHANISM OF FRACTURE INITIATION

The effect of flow value F on fracturing pressure P_f depends upon the properties of soils such as strength and permeability. Therefore,

Fig. 8. Relation between hydrofracturing pressure P_f and method of injection

Fig. 9. Relation between injection pressure P and injection time T

it is important to clarify the relation between the mechanism of initiation of fracture and the soil type.

Figs. 10 and 11 show the results of cohesive soils. In any cohesive soil, the fracturing pressure P_f increases as the flow value F becomes large in the case when F is smaller than 30 sec. However, it seems that P_f becomes constant critical value if F exceeds 30 sec. The vertical axis in Fig. 11 shows the fracturing coefficient R_p . R_p is given by Eq. (4).

$$R_p = (P_f - \sigma_3) / q_u \quad (4)$$

It is found from Fig. 11 that the relation

Fig. 10. Relation between hydrofracturing pressure P_f and confining pressure σ_3

Fig. 11. Relation between fracturing coefficient $(P_f - \sigma_3) / q_u$ and flow value F

between R_p and F is not influenced by the value of unconfined compressive strength q_u of cohesive soil. When F exceeds 30 sec, the value of R_p becomes constant. The situation of pressurizing by injection materials with the very large viscosity means the expansion of cavity rather than the wedge action. The fracturing pressure P_f in this state is probably equal to the cavity expansion pressure P_u which was suggested by Vesic (Vesic, 1972). P_u in the case of cylindrical cavities given by Eq. (5).

$$P_u = c_u F_0 + \sigma_3 \quad (5)$$

$$F_0 = \ln \left\{ \frac{E}{2c_u(1+\nu)} \right\} + 1 \quad (6)$$

c_u : cohesion σ_3 : confining pressure
 E : elastic modulus ν : Poisson's ratio

The dashed line in Fig. 11 means the calculated value of P_u of cohesive soil used in the fracturing tests. It seems that P_u is almost equal to the fracturing pressure P_f in the case when the value F is larger than 30 sec. Therefore, the pressurizing condition in this state would be the same as the pressurization by the membrane such as the rubber balloon.

The relation between the fracturing pressure P_f and the flow value F is experimentally given by Eq. (7).

$$P_f = \sigma_3 + \left(-\frac{1}{0.1F - 0.3} + 2.8 \right) q_u \quad (7)$$

In the sandy soils, those relation is shown by Fig. 7. The dashed line in Fig. 7 demonstrated the cavity expansion pressure $P_u' (= P_u - u_0)$ of sandy soil which is given by Eq. (8). In the case of sample S3 permeability of which is larger than 10^{-2} cm/s, the fracturing pressure P_f' is almost equal to P_u' . On the whole, the effect of flow value F on the fracturing pressure P_f' becomes smaller as the permeability of sand increases.

$$P_u' = F_q \cdot \sigma_3' + (F_q - 1) \cot \phi \cdot c_u \quad (8)$$

$$F_q = (1 + \sin \phi) \times \left\{ \frac{E \sec \phi}{2(1+\nu)(\sigma_3' \tan \phi + c_u)} \right\} \left(\frac{\sin \phi}{1 + \sin \phi} \right) \quad (9)$$

ϕ : frictional angle

If the viscosity of injection material is as

small as water, "the wedge action," which means that the injection material penetrates into the crack around the borehole and enlarge the crack, can be easily mobilized. In this case, the minute crack around the borehole caused by the shear failure or tensile failure can develop into the fracturing. However, if the viscosity of injection material is large, the larger injection pressure is needed, because it is difficult for the injection material to penetrate into the crack. In addition, the pressure with which the injection material penetrates into the crack depends upon not only the viscosity of injection material but also the permeability of soil. If the permeability of soil is small enough, the injection material, which penetrated into the crack, cannot permeate into the soil from the crack inside surface. In this case, the crack easily extends and the fracturing is generated, because the pressure descent of injection material along the crack is very small and the pressure in the crack is maintained.

Thus the fracturing pressure of sandy soil permeability of which is small as well as cohesive soil is related to the flow value F .

On the other hand, in the sand permeability of which is very large, even if the crack around the borehole is initiated, the injection material permeates into the soil from the crack inside surface, pressure of injection material in the crack goes down, and it is hard for the wedge action to occur. Therefore, the fracturing would not be generated unless the injection pressure reached to "the cavity expansion pressure" which gives rise to the general failure around the borehole. For that reason, the effect of flow value F on the fracturing pressure P_f' decreases as the permeability of soil increases.

CONCLUSIONS

Through a lot of fracturing tests on sandy and cohesive soils using the viscous injection materials, the authors obtained the following conclusions:

1) The fracturing of cohesive soils is generated by the wedge action of the injection

material which was penetrated into the crack around the borehole. Therefore, the fracturing pressure P_f generally becomes large as the flow value F of injection materials increases. However, the fracturing pressure becomes constant when F exceeds 30 sec. The constant value is almost equal to the cavity expansion pressure which gives rise to the general shear failure around the borehole. And the relation between the fracturing pressure and F is given by Eq. (7).

2) In sandy soils, the effect of flow value F on the fracturing pressure becomes smaller as the permeability of sand increases. If the coefficient of permeability k of sandy soils is larger than 10^{-2} cm/sec, the fracturing pressure is equal to the cavity expansion pressure of sandy soils, irrespective of viscosity of injection materials.

3) The effect of pressurizing method of injection materials on fracturing pressure is small, because the fracturing pressure in the tests with the constant increment of injection pressure is almost equal to that with the constant increment of injection rate.

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