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Compressibility of broken rock-fine grain soil mixture

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Abstract. Due to the enormous amount of fills required, broken rock-fine grain soil mixtures have been increasingly used in the construction of high-fill foundations for airports, railways and highways in the mountain areas of western China. However, the compressibility behavior of those broken rock-fine grain soil mixtures remains unknown, which impose great uncertainties for the performance of those high-fill foundations. In this research, the mixture of broken limestone and a fine grain soil, Douposi soil, is studied. Large oedometer tests have been performed on specimens with different soil content. This research reveals the significant influence of fine grains on the compressibility of the mixture, including immediate settlement, creep, as well as wetting deformation.

Keywords: compressibility; high-fill foundation; long term settlement; wetting deformation

1. Introduction

Due to the fast economic growth in western China since 1990s, there are mounting demands for improving the transportation infrastructures in those areas. More and more airports, highways, and railways are being or going to be built. However, in contrast to the vast plain area around Beijing and Shanghai in eastern China, most parts in western China are of mountain areas. As a result, many of the airports have to be built on high-fill foundations, the height of which could reach 50 meters and even 100 meters, while the maximum fill height for Panzhihua Airport in Sichuan Province even exceeds 123 m. Furthermore, construction of airports in those areas always involves extremely large amount of geomaterials being excavated and filled. For example, a new international airport has been under construction near Kunming, the capital of Yunnan Province, with a total volume of excavation and fills estimated to be 0.13 billion m^3 and 0.12 billion m^3 respectively, and a maximum embankment height of 52 m.

Since the amounts of fills needed are enormous, importation of fills by road over a long distance would become uneconomic and unrealistic. Therefore, most of the fills have to be acquired locally around the construction site, preferably by re-using the excavated materials. Although granular materials or broken rocks are always preferred for high-fill foundations due to their high permeability and low compressibility, their availability always becomes a problem due to such large quantity required. In contrast, these areas are usually covered with thick layers of fine grain soils.

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Those fine grain soils are mainly resulted from rocks being heavily weathered or eroded during their geological history in the warm and humid climate. Therefore, broken rock-fine grain soil mixture has been increasingly used for many high-fill foundations in recent years.

For high-fill foundations, settlement may continue over many years under the large self-weight of fills (Athanasiu 2005, Soriano and Sanchez 1999). There are deep concerns about the long term settlement for those high-fill embankments, because too large settlement will cause serviceability problems or even cause damages to the infrastructures above. For example, the Kunming Airport Design Guidance (2007) requires that the absolute settlement should not exceed 250 mm and the differential settlement should not exceed 0.18% during the 20-year service period after construction. Clearly, for those high fill foundations, it is essential that post-construction settlement should be predictable and within acceptable limits. However, there is little reported investigation on the compressibility behavior of broken rock-fine grain soil mixtures, as previous researches mainly involve studying the long-term behavior of rockfills (*e.g.* Charles 1990) as well as fine grain cohesive soils (*e.g.* Brandon *et al.* 1990). Therefore, uncertainties rise for using such mixtures in the construction of high-fill foundations.

This paper presents the findings from a laboratory research program on the immediate settlement, creep, and wetting deformation of broken rock-fine grain soil mixtures typically used in western China, with the aim of improving our understanding of those fills and providing guidance for construction.

2. Large oedometer testing

2.1 Test equipment

Due to the large particle size, conventional small test apparatus is not suitable for studying the behavior of rockfills. Therefore, a large oedometer has been developed during this research for investigating specimens with a diameter of 200 mm and a height of 200 mm. The maximum particle size of the specimen is 40 mm, which is 1/5 of the specimen height. The apparatus utilizes a 1:32 lever arm to apply vertical compressive pressure on the specimen. A special re-balance device is designed, so that the vertical position of the lever fulcrum can be adjusted smoothly to keep the lever arm horizontal during the test process. A maximum pressure of 1.34 MPa can be applied on the specimen, representing an overburden of 65 m approximately.

2.2 Broken rock-fine grain soil mixture specimen

The mixture of broken limestone and Douposi soil was studied in this research, which have been extensively used in Kunming airport construction. Douposi soil is a residual soil resulted from heavily weathered mudstone and sandstone, which is a typical soil in those areas with a tropic and humid climate. Douposi soil mainly consists of silty materials, and the particle size distribution for testing is presented in Fig. 1. Limestone is a typical rock in those areas. The particle size distribution of limestone is also presented in Fig. 1.

This research only investigated the mechanic behavior of limestone. However, it should be aware that the solubility of limestone in water and weak acid solutions (*e.g.* CO_2 environment) could cause degradation of limestone rockfills, resulting in additional long-term settlement.

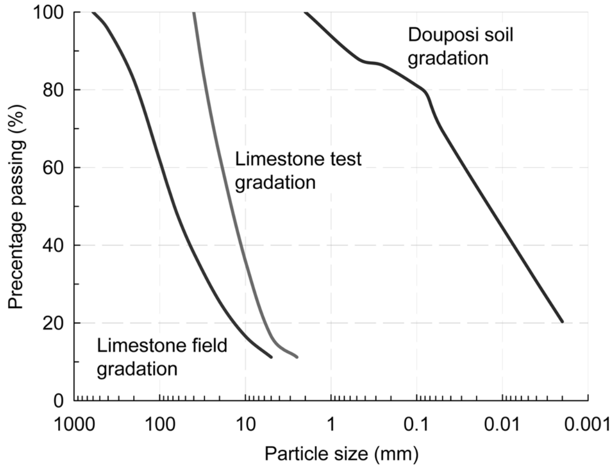

Fig. 1 Particle size distribution for broken limestone and for Douposi soil

The large particle size of rockfills could exceed 0.5 m in the field. To study the behavior of rockfills in the laboratory, the field gradation has to be scaled down (Parkin 1990) so that the fills can be tested in a laboratory apparatus, *e.g.* the large oedometer in this research. Previous researches (*e.g.* Marachi *et al.* 1972, Charles and Watts 1980, Indraratna *et al.* 1993) have demonstrated that this technique could generate reasonable results and could be used for predicting the field performance. To avoid large errors, the ratio of H/d_{max} should be kept less than 5 (BS1377, 1990), where H is the oedometer height and d_{max} is the maximum particle size. Therefore, the maximum particle size of broken limestone is 40 mm in this research.

Engineers tend to mix the broken rock with fine grain soil under a certain ratio by weight during construction. Therefore, a total of 7 specimens were tested, with different soil contents by dry weight as 0%, 10%, 20%, 30%, 50%, 70%, and 100%. Each specimen was prepared in 4 layers, compacted with a standard Proctor rammer and with the same compaction work of 281 kJ/m^3 for each layer.

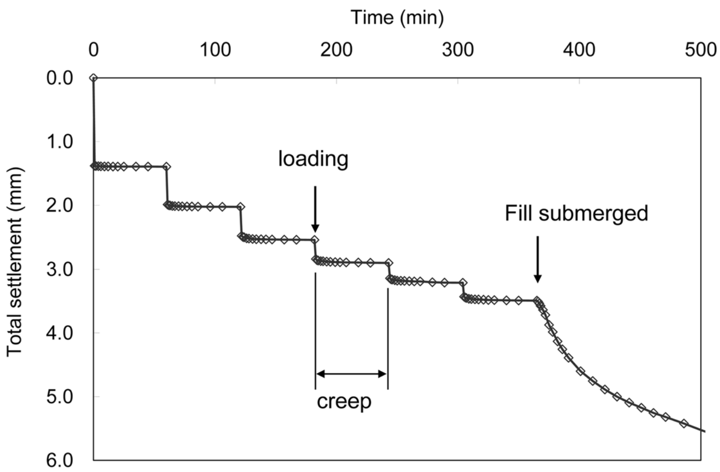

Fig. 2 Testing procedure for each specimen

2.3 Testing procedure

The testing procedure was kept identical for each specimen, as demonstrated in Fig. 2. The vertical pressure was applied in 6 stages for each specimen, from 0.14 MPa to 1.14 MPa. In each stage the pressure was increased by 0.2 MPa, and most settlement happened immediately after loading. Therefore, the measured settlement during the first minute after loading is taken as immediate settlement. Then approximate 1 hour was left for creep, during which the settlement and elapsed time was recorded frequently. At the end of each test, the specimen was submerged with water and the wetting collapse deformation is studied.

3. Results and discussion

3.1 Dry density of broken rock-fine grain soil mixture

The specimens with different soil contents were prepared with the same compaction work, as described before, and the dry densities were compared in Fig. 3. The rockfill has a density of 1.98 t/m^3 with a porosity n = 0.25 when the fine grain soil content increases, the density increases until reaching a maximum of 2.16 t/m^3 for the specimen with 20% soil content. However, further increase in soil content leads to a decrease in density.

3.2 Immediate settlement

Fig. 4 presents the relationship between void ratio e against applied vertical stress σ_v for each specimen. An approximately linear relationship is found between e and the logarithm of σ_v for each specimen with different soil content. The compression index C_c is defined as

$$C_c = -\frac{\Delta e}{\Delta(\lg \sigma_v)}$$

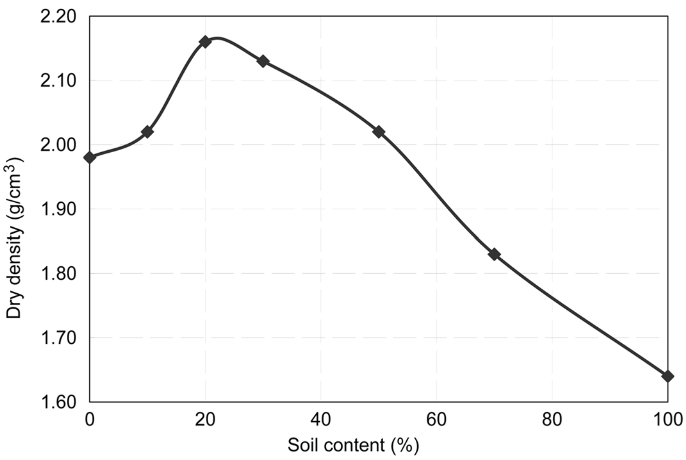

Fig. 3 Dry density for specimens with different fine grain soil contents

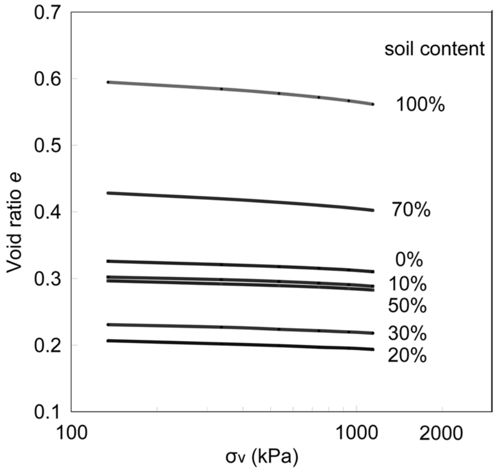

Fig. 4 e-log(σ_v) for specimens with different fine grain soil contents

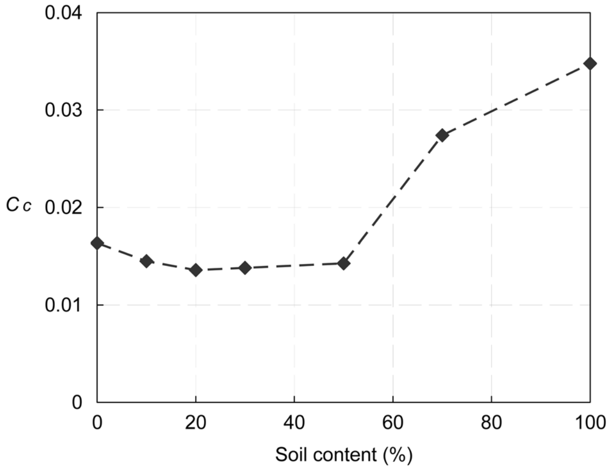

Fig. 5 Cc for specimens with different fine grain soil contents

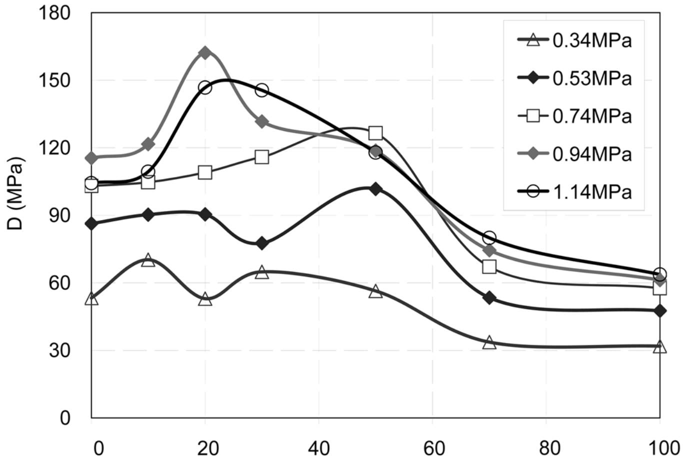

Fig. 6 Constrained modulus D for different specimens at each applied pressure level

A smaller C_c implies a lower compressibility under loading. As can be seen in Fig. 5, the compression index is the lowest when fine grain soil content is 20-30%, while the pure broken rock has a slightly larger C_c.

The behavior of fills in one-dimensional compression can also be described by the tangent constrained modulus D, which is the ratio of vertical stress increment $\Delta\sigma_v$ to the vertical strain increment $\Delta\varepsilon_v$ as $D = \Delta\sigma_v / \Delta\varepsilon_v$ (Fig. 6). Despite of the scatter nature of tangent modulus, it can still be found that D reaches its maximum magnitude when the soil content is 20% to 40%, while it reduces for pure broken rock specimen.

For gravels, *e.g.* broken rocks, the load is transferred through contact areas or points between particles, while the contact areas are usually very small. Previous researches (*e.g.* Santamarina *et al.* 2001) have also revealed that the load was not applied to each particle equally, but mainly transferred through particle chains which are approximately along the direction of the major

Table 1 Particle size distribution before and after test for the broken rock specimen

Particle size mm		40	30	20	10	5	2.5
Passing Percentage %	before test	100.0	82.6	63.1	35.9	16.6	11.2
	after test	100.0	88.0	67.0	37.5	17.6	11.2

principal stress, *e.g.* the vertical direction under one dimensional compression. This means that the local stresses at some particle contacts could be very high and well in excess of the average vertical pressure, leading to particle breakage and subsequent particle position change, *e.g.* slippage and rotation.

Table 1 compares the particle size distribution before and after test for the broken rock specimen. There are significant particle breakages, particularly for large particles. The percentage of particles larger than 30 mm decreases from 17.4% down to 12%, which implies that 1/3 of large particles breaks. Since the elastic deformation of rock particle is very small due to its high particle modulus, the deformation of broken rock specimen is therefore believed to mainly involve particle breakage and associated slippage and rotation.

When broken rocks are mixed with a small amount of soil, the fine grains would fill the voids among rock particles, leading to a more uniform distribution of stresses and less rock particle breakage. Furthermore, fine grains in the voids make rock particle slippage or rotation more difficult. As a result, less settlement happens under loading and the compressibility decreases (*e.g.* a lower *Cc* and a higher *D*). However, when the soil content exceeds 20%, the soil begins to separate rock particles and begins to dominate the behavior of the mixture, resulting in a higher compressibility.

3.3 Creep settlement

The creep strains under each applied stress for specimens with different soil contents are studied, and typical results are presented in Figs. 7~8. As can be seen, there is approximately a linear relationship between creep strain and the logarithm of the elapsed time.

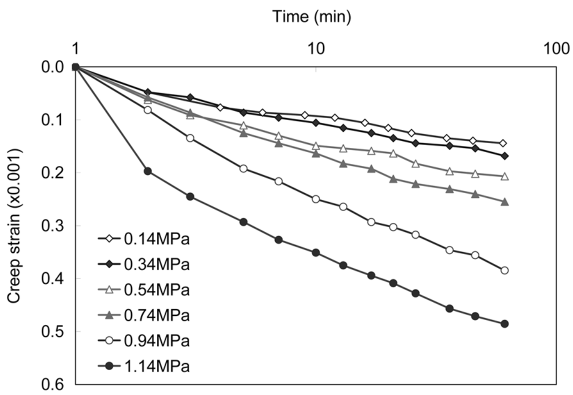

Fig. 7 Creep strain against the logarithm of time for the specimen with 50% soil

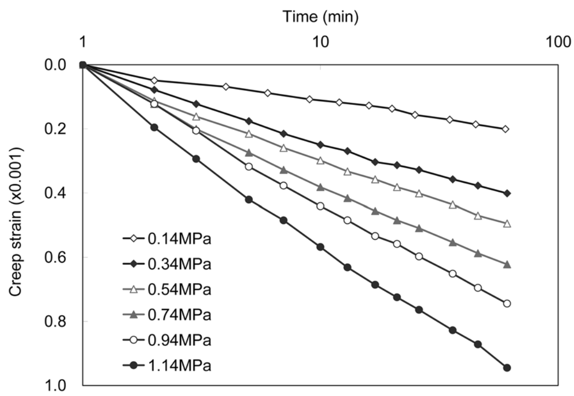

Fig. 8 Creep strain against the logarithm of time for the specimen with 100% soil

$$\varepsilon = \frac{\alpha}{1000}\lg\left(\frac{\Delta t_2}{\Delta t_1}\right)$$

where ε is the creep strain between time t_1 and t_2; α is a constant and is defined as creep compression rate parameter (Charles and Skinner 2001). The linearity becomes less significant when the soil content decreases.

The investigation of creep could be very time consuming. However, the linear relationship between creep and the logarithm of the elapsed time is also well observed in the previous laboratory compression tests and in the field. For example, Sowers *et al.* (1965) and Charles & Skinner (2001) reported the field monitoring results of a wide range of rockfill dams in the United States and in the United Kingdom over many years, *e.g.* decades. Approximately linear relations were observed between the long term settlement and the logarithm of the time since the middle of the construction period. Therefore, only one hour was left for creep at each stress level, aiming to derive the creep compression rate parameter α. Then the research mainly focused on investigating the influence of the vertical stress and the soil content on the creep behavior.

The logarithmic creep rate parameter α is found to be stress dependent for each specimen, and a linear relationship becomes evident between α and applied vertical stress σ_v, as demonstrated in Fig. 9.

$$a = k\frac{\sigma_v}{p_a}$$

where p_a is the atmospheric pressure. Charles & Skinner (2001) examines the long term settlement at different depth of several UK rockfill dams, with extensometers installed in the regions of those embankments not seriously affected by reservoir fluctuations. Similar relation between α and vertical stress was observed, despite of the scatter of monitoring data. At the 56 m-high Megget Dam constructed with sandy gravel, k was found to be about 0.04. In comparison, $k = 0.034$ was derived for the pure broken limestone rock specimen tested in this research. Comparable agreement is observed for those two different granular materials.

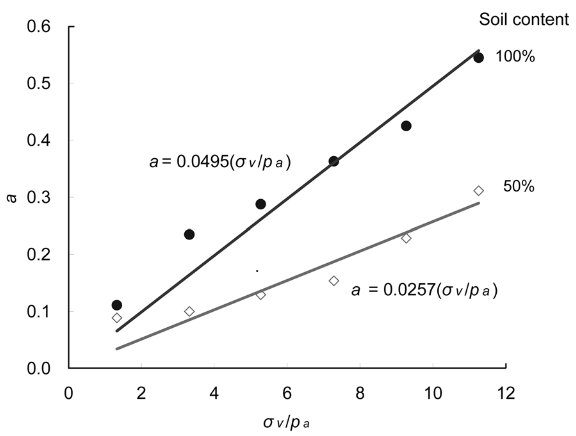

Fig. 9 Variation of α with pressure for the specimens with 100% and 50% fine grain soil

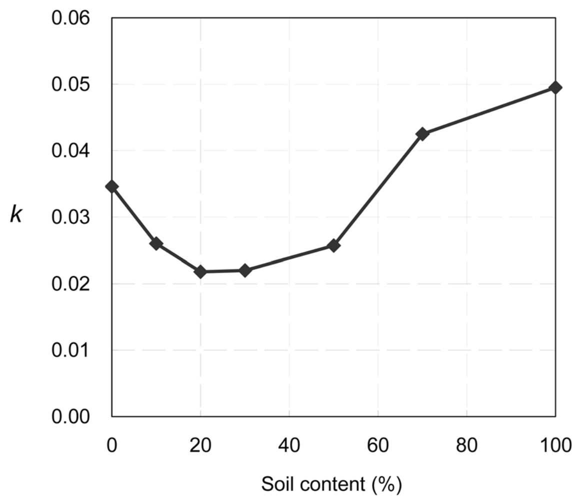

Fig. 10 Variation of k with fine grain soil content

The value of k was calculated for each specimen with different soil content, as shown in Fig. 10. Again, it can be seen that fills with about 20% soil content has the minimum k value, implying a least creep rate at a given stress level, while pure rockfill have a larger k value.

The creep of granular materials is believed related to the time dependent particle strength, *e.g.* reduction of particle strength with time (McDowell 2003, McDowell & Khan 2004). As explained in the previous section, additional fine grains in voids among broken rock particles would reduce the peak contact stresses. As a result, less particle breakage happens, not only during immediate loading but also during the rest period. Therefore, the creep rate decreases until the soil content increases to a certain ratio, e.g. around 20%. Further increase of fine grains will separate rock particles and prevent them from touching each other, and the soil tend to dominant the mechanical behavior of the mixture, resulting in an increasing creep rate.

3.4 Wetting collapse

The wetting deformation was investigated, as the fills in the field may become wetted due to the seepage of surface water during heavy rain, or submerged due to the rise of ground water table. The latter is considered more likely to happen for high-fill foundations in western China, where the ground water conditions are very complex, especially in the karst areas. Construction of high fills over a wide area may change the ground water conditions, *e.g.* blocking underground rivers and causing underground water table to rise slowly.

Each specimen was submerged under a constant vertical pressure of 1.14 MPa at the end of the test, following by a rest period of about one hour for observing wetting deformation. The development of wetting deformation during inundation is demonstrated in Fig. 11. It can be seen that when the rockfill specimen was submerged, the compressive strain increased immediately by 0.5%. Even much larger wetting strains were observed for the specimens with 10% to 20% soil experience. The wetting strain was 0.8% for the specimen with 10% soil, while it even exceeded 1% and continued to increase after 1 hour for the specimen with 20% soil. Those wetting strains were much higher than the corresponding creep strains, *e.g.* by more than an order of magnitude.

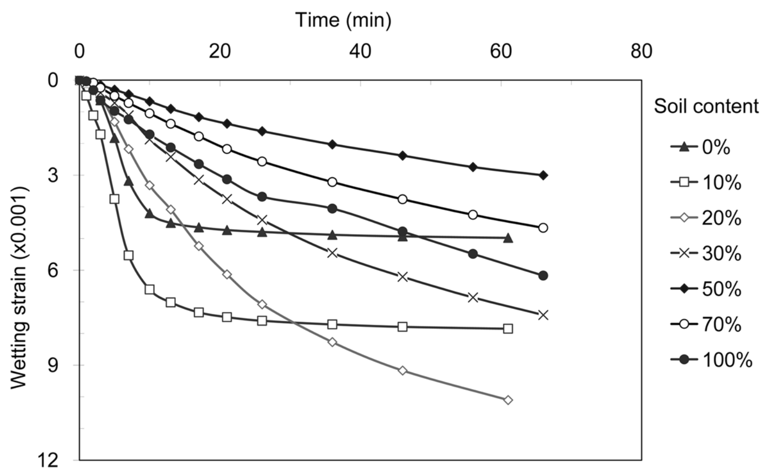

Fig. 11 Wetting strain for specimens with different fine grain soil content submerged under a vertical pressure of 1.14 MPa

For specimens with higher soil contents, the wetting deformation continued to increase without becoming stabilized after one hour due to the low permeability. However, the significance of wetting collapse already became evident, which should be treated seriously during the design and construction stage.

Previous researches (Charles and Watts 1996) suggest that loose fills without systematic compaction are susceptible to wetting collapse. Our work, however, reveals that significant wetting deformation could happen even for well compacted broken rock fills or broken rock-soil mixtures under high stresses.

The significant influence of fine grains on the wetting deformation also becomes obvious. For specimens with just only 10% to 20% fine grain soil, the wetting deformation becomes almost doubled compared with that of rock fills.

When dry soil particles become wetted or submerged, the strength of particles were weakened (Sowers *et al.* 1965), so that particles become much easier to break under the same stresses. At the mean time, water also reduces the frictions between particles, which facilitate particle slippage and rotation. Previous researches demonstrate that serious wetting collapse happens not only to coarse granular materials (*e.g.* Charles and Watts 1996), but also to fine grain cohesive fills, as observed by Brandon *et al.* (1989). This research reveals for the first time that significant wetting deformation could be induced to broken rock-fine grain soil mixture.

4. Conclusions

Due to the enormous amount of fills required, broken rock-fine grain soil mixtures have been increasingly used in the construction of high-fill foundations for airports, railways and highways in the mountain areas of western China. However, the compressibility behavior of those broken rock-fine grain soil mixtures remains unknown, which impose great uncertainties for the performance of those high-fill foundations after construction. In this research, the mixture of broken limestone and a fine grain soil, Douposi soil, is studied. Large oedometer tests have been performed on specimens with different soil content. This research reveals the significant influence of fine grains on the compressibility of the mixture, including the immediate settlement, creep, and wetting deformation.

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