A LABORATORY STUDY OF FRACTURE GROUTING TECHNIQUE IN SAND

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ABSTRACT

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In this study, fracture grouting technique of saturated, granular soils of different fine content were investigated. Model tests were carried out by using fluid particulate grouts namely micro fine cement and ordinary portland cement grouts. Basically, relationships were obtained between soil conditions (grain size distribution, relative density, overburden stress) and grouting parameters (type of grout, grouting pressure, amount of injected grout, rheological properties of the grout or water/solids ratio). At the end of the tests the soil specimens were exposed and the final grout shapes were observed and correlated with the grouting parameters. Response of soil specimens to grouting process under different grouting pressures and grout compositions was analyzed. Amount of heave occurred at the top of the specimen during injections was recorded at each test.

Micro fine cement grout and ordinary portland cement grout showed significant differences rheologically. Micro fine cement grout, with much higher Blaine fineness, lower specific gravity, lower viscosity and cohesion, lower bleed and filtration coefficients, made it possible to fracture the fine sandy soils of different fine content.

Results of tests performed with micro fine cement grouts show that fracturing pressure generally decreases with an increase in the water content of the grout but generally increases as the fine content of the soil increases.

A higher relative density of the soil increases the fracturing pressure significantly.

The volumes of grout injected into soil specimens until fracturing show an increasing tendency as the water/solids ratio decrease.

Ordinary portland cement grout, on the other hand, exposed to high pressure filtration during grouting in relatively clean sand and addition of some amount of kaolinite or fines is required to reduce the filtration percentages during grouting in order to fracture grout the sandy soil. Filtration due to high permeabilities results in accumulation of cement particles around the injection point and grouting tends to take a form similar to compaction grouting.

Keywords: Grouting, Fracture Grouting, Compaction Grouting, Sand, Water/Solids Ratio, Viscosity

ÖZ

ÇATLATMA ENJEKSİYONU YÖNTEMİNİN KUM İÇERİSİNDE UYGULANMASINA YÖNELİK DENEYSEL BİR ÇALIŞMA

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Bu çalışmada farklı inceler oranına sahip, suya doygun, gevşek granüler zeminlerin çatlatma enjeksiyonu teknikleri ile iyileştirilmesi araştırılmıştır. Hem mikro ince çimento hem de normal portland çimentosu bazlı, süspansiyon tipi enjeksiyon karışımları kullanılarak model deneyler gerçekleştirilmiştir. Temel olarak zemin koşulları (dane çapı dağılımı, rölatif yoğunluk, jeolojik yük) ile enjeksiyon parametreleri (enjeksiyon karışımı tipi, enjeksiyon basıncı, enjekte edilen hacim, enjeksiyon malzemesinin reolojik özellikleri veya kısaca su/katı oranı) arasındaki ilişkiler belirlenmiştir. Deneyler sonucunda enjekte edilen numuneler dikkatli bir şekilde açılarak elde edilen şekil enjeksiyon parametreleri ile ilişkilendirilmiştir. Numunelerin farklı enjeksiyon basınçları ve enjeksiyon karışımları altında enjeksiyon işlemine karşı gösterdikleri davranış incelenmiştir. Enjeksiyon sırasında numune üzerinde meydana gelen kabarmalar da her deney süresince kaydedilmiştir.

Mikro ince çimento ve normal portland çimento karışımları reolojik açıdan önemli farklılıklar göstermiştir. Daha yüksek Blaine inceliğine, daha düşük özgül ağırlığa, daha düşük vizkozite, kohezyon ve çökme değerlerine ve yine daha düşük filtrasyon oranlarına sahip olan mikro ince çimento ile ince kumlu zeminlerin çatlatılması mümkün olmuştur.

Mikro ince çimento karışımları ile yapılan testler çatlatma basıncının enjeksiyon karşımındaki su miktarının artmasıyla düştüğünü fakat numunenin inceler oranının artmasıyla yükseldiğini göstermiştir.

Numunenin rölatif sıkılığının artırılması çatlatma basıncını önemli ölçüde arttırmıştır.

Çatlatma anına kadar numune içerisine enjekte edilen hacimler, enjeksiyon karışımının su/katı oranının artmasıyla yükselmiştir.

Diğer taraftan, normal portland çimentosu enjeksiyon karışımları, inceler oranı düşük temiz kumlarda enjeksiyon işlemi sırasında yüksek oranda filtrasyona maruz kalmış ve numunelere çatlatma enjeksiyonu ile girilebilmesi filtrasyon oranlarına düşürebilmek maksadıyla belli bir miktar kaolin katılmasını gerektirmiştir. Yüksek geçirgenliklerden kaynaklanan filtrasyon çimento daneciklerinin enjeksiyon noktası etrafında toplanması ile sonuçlanmakta ve numune içerisinde enjeksiyon kompaksiyon enjeksiyonuna benzer bir şekil almaktadır.

Anahtar Kelimeler: Enjeksiyon, Çatlatma Enjeksiyonu, Kompaksiyon Enjeksiyonu, Kum, Su/Katı Oranı, Vizkozite

To My Parents and Wife

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CHAPTER 1

INTRODUCTION

Turkey, being one of the countries most susceptible to earthquake motions, is under the threat of liquefaction due to the vast presence of loose saturated sands and non – plastic silty sands / sandy silts both beneath existing buildings and new sites and around underground structures. The 1999 Kocaeli earthquake also revealed that many buildings are founded on loose, liquefiable soils. When these soils are subjected to seismic excitation, liquefaction will cause loss of strength, lateral ground spreading, and vertical ground settlement resulting in foundation distress and instability of building supported on such soils. Therefore, critical facilities such as hospitals, lifelines, and transportation systems founded on such soils need to be protected from such failures. Also these critical facilities should maintain their operations during the improvement of their foundation soils.

In recent years, grouting techniques for improvement of liquefiable soils are considered as an alternative to other improvement techniques such as dynamic compaction or vibro compaction since these techniques cause high vibrations under existing buildings and can not be performed within the buildings. With its small, compact equipment grouting techniques make it possible to reach the target soils from the basement or outside of the existing buildings.

However, it is still not known clearly how basically granular soils are improved by grouting. Jet grouting is a special category. It is a mixing technique at very high pressures. It is known for long that coarse sands and sand-gravel mixtures can be permeation grouted and micro fine cements can permeate to medium sands. Literature on fracture grouting of granular soils are very scarce and believed to be in the files of grouting companies. Variables like grout pressure, water/cement ratio, type of cement, type of soil, additives are seldom discussed in detail in fracture grouting. Compaction grouting techniques, on the other hand, have been used recently for the improvement of granular soils against liquefaction. Because of various functions of grouting, the differences between grouts themselves and the differences between the materials to be grouted, generalizations about how the grout behaves within the ground under certain circumstances are difficult to make. It is not clear which grouting method could be used with which grouting parameters and in which soil conditions for the best efficiency.

In this study, fracture grouting techniques of saturated, granular soils with different fine content was investigated. Both micro fine cement and ordinary Portland cement grouts were tried. Basically, relationships were obtained between soil conditions (grain size distribution, relative density, overburden stress) and grouting parameters (type of grout, grouting pressure, amount of injected grout, rheological properties of the grout or water/solids ratio). At the end of the tests the soil specimens were exposed and the final grout shapes were observed and correlated with the grouting parameters. Response of soil specimens to grouting process under different grouting pressures and grout compositions was analyzed.

It is aimed to define the behaviors of micro fine cement grout and the ordinary Portland cement grout in granular soils with different soil and grouting parameters.

CHAPTER 2

LITERATURE SURVEY

2.1 Definition and purposes of grouting

Grouting usually refers to the controlled injection of suspensions, solutions and emulsions under pressure into certain parts of soil or rock to improve their geotechnical characteristics. For this reason, grouting is a process where the remote placement of a pumpable material in the ground is indirectly controlled by adjusting its rheological characteristics and manipulating the placement parameters (pressure, volume and the flow rate).

Grouts can be classified into two basic categories: suspension or particulate grouts (Bingham fluids); and solution or non-particulate grouts (Newtonian fluids). Particulate grouts consist of cement-water, clay-cement or cement-clay-water mixes. The most common classes of chemical grouts are silicates, lignins, resins, acrylamides and urethanes. The silicate grouts are the most widely used chemical grouts.

The technology of grouting, like the several other ground improvement methods, is far from new. However, it is in a constant state of progress, with new materials and construction techniques being developed constantly. Modern grouting began in the mining industries for seepage and strength control in mines, tunnels and shafts, then in civil engineering it was used in the construction and maintenance of a subbase, for stabilizing loose soils or fragmented rocks during tunnel driving, reducing the liquefaction potential of loose saturated granular deposits during earthquakes and as an underground cut-off wall for strengthening and impermeation of dam foundations. In addition to these applications, grouting technique is particularly valuable in foundation work before, during or after construction. Before construction, it may be used to control water problems during boring, sampling, etc., to infill voids in order to eliminate possible future settlement and to increase permissible soil pressures relative to the untreated soil for new structures and extensions to already existing ones. During construction, proper grouting can control groundwater flow, prevent loose sand densification below adjacent structures due to pile driving and increase the stability of granular soils below existing structures so as to reduce the need for lateral support. After construction, the possible applications include underpinning, reduction of machine foundation vibrations and the elimination of seepage through openings.

Grout used to reduce the permeability of the ground must be able to develop sufficient strength to withstand the hydraulic gradient imposed. Normally a cement or clay-cement grout is used in coarser soils and clay-chemical or chemical grouts are used in finer grained soils. Micro fine cement is used to grout fine sands. On the other hand, when grouting aims to enhance strength, a cement-based grout is preferred if the particle size distribution of the soil permits, because this gives a higher strength compared with other grouts. If this is not the case, chemicals must be used since clay-based grouts add little strength to the soil. Resin or polymer based grouts are usually stronger than silicate grouts.

Determination of the most suitable grout to be injected under certain conditions requires data about the size of the voids to be grouted. The arrangement of the soil profile and the thickness of the overburden should also be known since they influence the maximum grout pressure that can be used. As far as the grout is concerned, it must be easy to mix and inject. It must be capable of flowing readily through the voids under reasonable pressure and must form a substance with adequate strength when set so that it is not displaced. It must set with a minimum amount of shrinkage and remain stable. The grout should not contaminate the groundwater (Bell, 1993).

2.2 Categories of grouting

Depending on the method used to introduce the grout into the ground grouting techniques can be subdivided into permeation grouting, fracture grouting, compaction grouting, jet grouting and compensation grouting. Compensation grouting is not a grouting process but may include grouting processes such as fracture and compaction grouting. The types of grouting are illustrated in Figure 2.1.

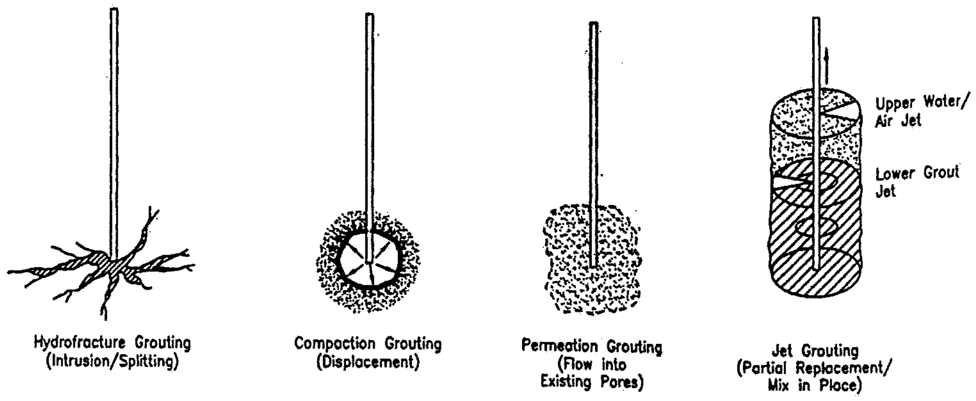


Figure 2.1 Types of basic grouting techniques (CIRIA, 2000)

2.2.1 Permeation grouting

In Xanthakos et al. (1994), it states that 'the techniques involved in permeation grouting are the oldest and best researched'. The intent of permeation is to introduce grout into soil pores without any essential change in the original soil volume and structure. The grouts are initially fluid, but harden in the course of time in the pore space, thus changing the properties of the mass of ground.

For genuine permeation the invasion of pore volume by the grout is continuous from the source and does not disturb the structure of the ground. Moreover the distribution of grout in the pore volume is uniform, but the grout need not necessarily completely fill the space. In practice some disturbance of the ground may be acceptable within the definition, providing there is uniform distribution of grout.

The requirement of uniform distribution of grout is important in relation to the objective of grouting. For reducing the flow of fluids through soils, it is usually sufficient for practical purposes to fill only the larger pore spaces in which flow concentrates. Blocking only these, leaving finer voids untreated is sufficient to cause a significant reduction in volume of flow. The target of treatment, in which uniform filling must be achieved, is thus only the coarser strata or lenses.

However, if the objective is to strengthen the ground, a large majority of particles must be surrounded by grout and mutually bonded together throughout the soil. Relatively small localised volumes of untreated particles can remain without significant effect, but they are unacceptable in layers or agglomerations. Unless the particles are mutually bonded to a uniformly high degree, improved strength and stiffness will not be realised. There is thus a clear distinction between the way the uniformity of treatment is perceived and the degree to which it can be effective when the objective is water-stopping rather than strengthening. Strengthening demands much more grouting skills.

It is appropriate to review the natural physical constraints controlling the penetration of fluid grout into porous ground or other porous media by permeation. These are addressed in a paper by Scott (1963). It identifies three main restrictions offering resistance to grout penetration:

1. Filtration of particles contained in the grout which are too big to pass through the void spaces in the ground.
2. Internal shear resistance due to the interaction of grout particles as the grout flows through the tortuous soil pore spaces.
3. Viscosity of the grout which restricts the rate at which the liquid flows into the soil pore spaces. Filtration and shear are absolute limitations and stop the grout flow. Change of fluid viscosity represents an increasing resistance as gelation progresses. High initial viscosity offers high resistance. Flow does not stop strictly, but for practical purposes it can do.

The effect of meeting resistance from these physical constraints is to raise the injection pressure. This can continue only until either the capacity of the pump is reached, or the ground splits in a hydrofracture. When this happens the grout forms and fills fissures which develop under physical controls determining the direction and thickness of the fissures. The direction depends on the existing state of stress in the ground, and the thickness on the interaction of the rheological properties of the grout and the dimensional and strength properties of the ground. However by definition, this form of interpenetration is irregular and is not permeation.

Permeation in uniform soils follows a very regular form which may be correctly represented by simple mathematical models. These models are usually based on either radial spherical or cylindrical flow volumes (Raffle and Greenwood, 1961). From these regular models and by using the properties of the grout and the ground it is possible to estimate the time for penetration to a given radius from which the injection hole spacing can be derived. The models also allow prediction of the limiting radius of penetration controlled by grout shear resistance (Greenwood, 1992). In the case of injection of chemical gels which have a relatively short gel time measured in minutes rather than hours, the effect of changing viscosity during injection, which is again linked with the selection of hole spacing, can be estimated.

The mathematical model employed to relate the value of the permeability coefficient (resistance) of the soil to the rheological properties of the grout is that of Kozeny-Carman (Kozeny,1927). This model equates the flow resistance of a bundle of parallel circular tubes to that of the ground, and uses Hagen-Poiseuille's law to derive the corresponding diameter of the tubes. It is this diameter which is used as a measure of the nominal diameter of the soil pore spaces. It is assumed that this dictates the maximum particle size in the grout which will not filter in the soil. Table 2.1 gives the results of such calculations. This pore dimension also governs the resistance developed by internal shear in the grout and by viscosity.

Table 2.1 Filtration of grout particles in soils (Kozeny-Carman:parallel tube model)

Nominal soil type	k (m/s)	Maximum particle size to pass (mm)
Fine gravel	10^{-2}	0.330
Coarse sand	10^{-3}	0.103: ordinary Portland cement
Medium sand	10^{-4}	0.033
Fine sand	10^{-5}	0.010 : unfiltered silicates

Regarding potential for filtration, the particle size distributions of typical cements are shown in Figure 2.2. Comparing the maximum sizes of these with the equivalent diameter of the soil pore space, it is clear that cements will not permeate soils in the range of permeability shown in Table 2.1. Since they correspond to sands, there is no possibility of Ordinary Portland cements permeating a sand. Even the specially finely ground cements can permeate only the coarse sands.

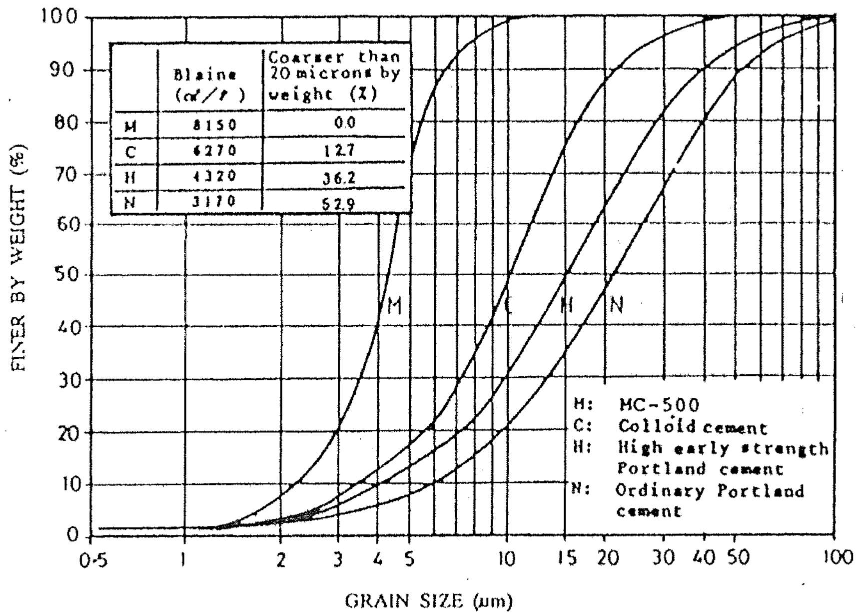


Figure 2.2 Typical particle size distributions for cements

Moreover filtration blockage occurs rapidly. Figure 2.3 illustrates by spherical models how few fine particles are needed to block pore spaces. A simple calculation (Greenwood, 1992) shows that for a typical injection hole dimension, only 1 or 2 litres is needed to choke a hole if filtration occurs. If cement will not go, fracturing or zero flow is inevitable. This result is supported by filtration tests with bentonites which have coarse particles, a similar size to that of cement, but finer particles are much smaller: they block by filtration in exactly the same manner (Greenwood, 1963).

Grouts containing cements or bentonites will not permeate sands neither will they permeate gravels containing sand. Nevertheless, ordinary cements may adequately permeate gravels, and special forms of cement, such as colloidal cements or micro fine cements, extend the range of applications into the sand range as shown in Figure 2.4 (Clarke, 1984). According to Clarke, micro fine cements have become a substitute for more toxic chemical grouts and has been used on many projects for underground water control. Figure 2.5 compares the viscosity of micro fine cements with that of colloidal and ordinary Portland cements. Increasing the water/cement ratio does decrease the viscosity but also increase the gel time and reduces the strength of the grouted soil.

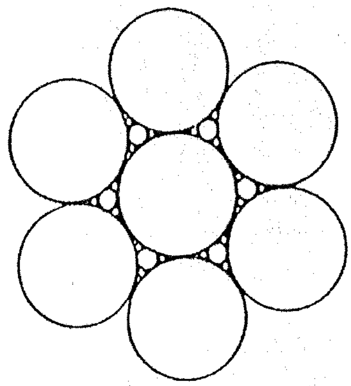


Figure 2.3 Illustration of how few particles will cause filtering blockage (Greenwood, 1992)

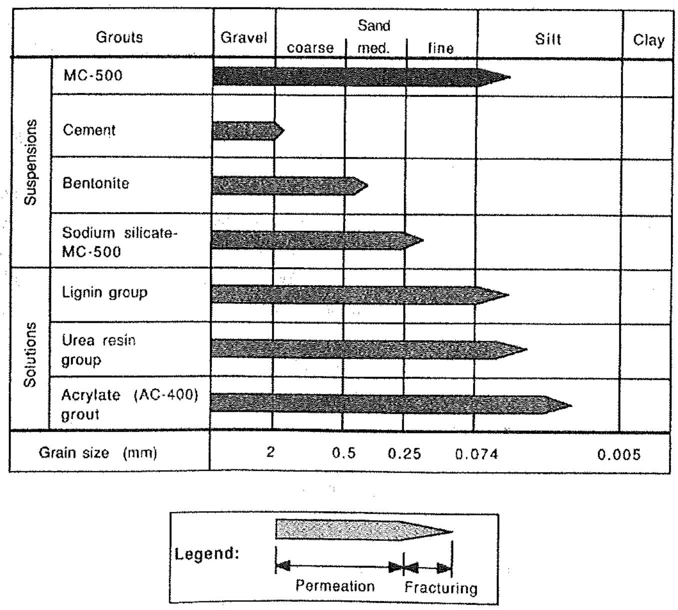
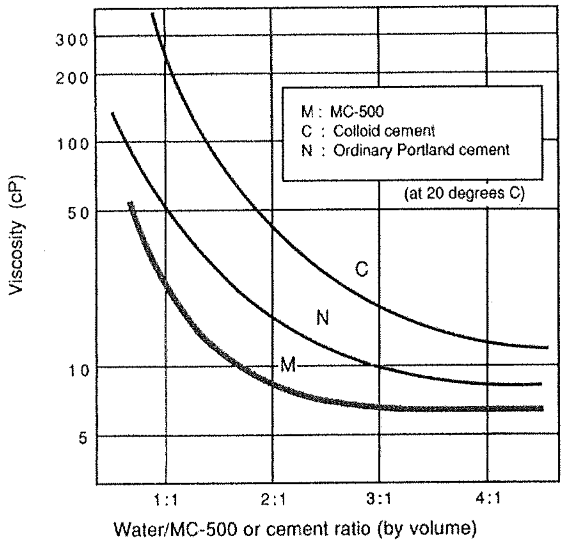


Figure 2.4 Comparison of permeation of grouts (Clarke,1984)



Note: 1 cP=10^{-3} Pa·s

Figure 2.5 Relationship between water/cement ratio and viscosity for different types of cement (Clarke, 1984)

The injection operations with chemical grouting are mostly based on sodium silicates diluted for sealing purposes with about 80 % water (with 20 % water when resistance is required). For sealing injections, many solutions based on acrylamides (e.g. AM-9) are also used; for strengthening injections, products based on phenoplasts (e.g. phenol-formaldehyde) are more usual.

With the original acrylamides, negative experiences caused by the strong toxicity were noticed in Japan. Nowadays (since 1980), these acrylamides are turned into 'AC-400', a less toxic monomer acrylate; or into 'injectite 80', a non-toxic polyacrilamide. However, the very high cost limits their general application.

Of importance with all chemical grouting materials is the initial viscosity and the variation of viscosity as a function of time at the moment of binding (Figure 2.6). Of course, it is advantageous to have an initial viscosity which is as low as possible, and an evolution in time as constant as possible. In this way, the injection can be easily performed.

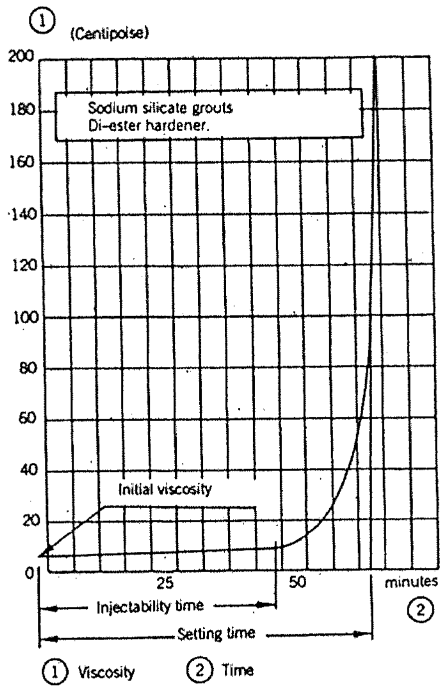


Figure 2.6 Example of viscosity evolution with time (AFTES, 1991)

Permeation grouting at shallow depths may take place at a single stage from a grout pipe, the grout-hole being sunk to full depth and then grouted upwards. Alternatively, grouting may proceed while the hole is drilled. The hole is extended a short distance using a hollow drill rod, it is then withdrawn this distance and grout is injected from the rod. The hole remains open over the exposed length since the soil will have been stabilized sufficiently by the migration of grout from previous injections. The cycle is then repeated. Lastly, grouting is continued as the rod is finally withdrawn from the hole.

Stage grouting is used when relatively high grouting pressures have to be employed to achieve satisfactory penetration of grout in deep holes or tighter sections of holes. In stage grouting the hole is drilled to a given depth and then grouted. After the grout has set the hole is deepened for the next stage of grouting (Figure 2.7). Stage grouting allows increasing grout pressures to be used for increasing depth of grout-hole and reduces the loss of grout due to the leakage at the surface.

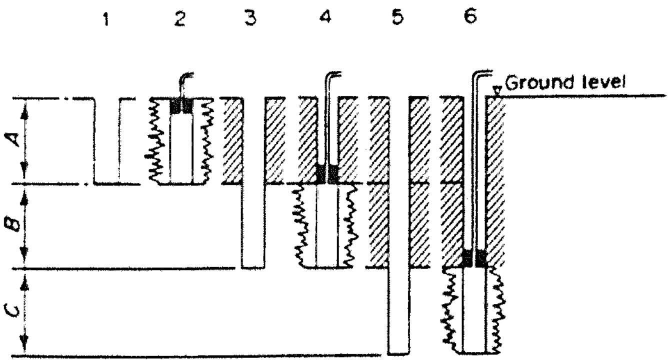


Figure 2.7 Stage-and-packer grouting (Bell, 1993)

Use of the *tube-a'-manchette* offers great flexibility since the same hole can be grouted more than once and different grouts can be used. In this way coarser soils can be treated first and finer ones later. Hence the *tube-a'-manchette* is often used to grout alluvial soils. The *tube-a'-manchette* consists of a steel tube, between 37.5 and 62.5 mm in diameter. This is perforated with rings of small holes (about 8 mm diameter) at intervals of approximately 0.3 m. Each ring of holes is enclosed by a tightly fitting rubber sleeve which acts as a one-way valve (Figure 2.8). A drillhole is sunk, with the aid of casing, to the full depth to be treated and the *tube-a'-manchette* is placed in it. The casing is withdrawn and grout, termed sleeve grout (clay-cement or bentonite), is poured into the annular space left behind. Grouting is then carried out through the *tube-a'-manchette* by lowering into it a small diameter injection pipe perforated at its lower end and fitted with two U-packers. The packers can be centered over any of the rings of injection holes. When injection starts the pressure in the grout pipe rises until the grout lifts the rubber sleeve, rupturing the sleeve grout and escaping through the small holes into the soil. The rubber sleeves stop any return of the grout into the *tube-a'-manchette* and the sleeve grout prevents any leakage of grout at the surface.

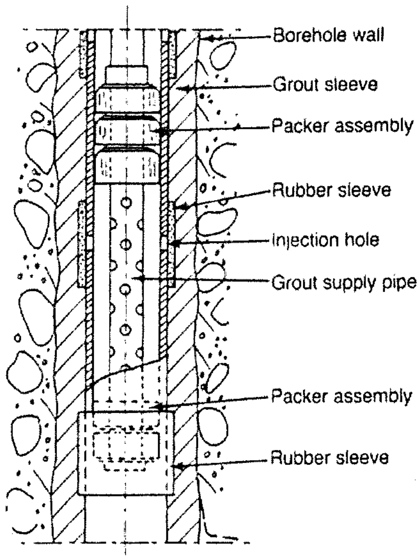


Figure 2.8 Detail of the tube-a'-manchette (Bell, 1993)

However, in congested urban areas, or where surface restrictions apply, the installation of TAM pipes from the surface, from shafts adjacent to the target grout zone, or from the tunnel face may not be viable options. In such circumstances, the recently-developed horizontal directional drilling (HDD) method of TAM pipe installation provides an alternative.

Directional drilling is used over great distances in the oil drilling industry and also, over shorter distances, for utility installation. For horizontal directional drilling for TAM installation, the addition of a steerable drill bit provides a precisely-located pilot bore over which a casing can be installed.

There are two basic drilling methods for horizontal directional drilling:

- The top drive system in which bit rotation speed and drill bit pressure is provided by a drill motor mounted on the drill mast.
- Use of an in-hole mud motor to rotate the drill bit. A mast-mounted drill head provides the steering rotation and drill bit pressure.

In the top-drive system, a slant-faced drill bit affixed to the drill string is rotated to advance the borehole. For steering, rotation is stopped while the bit is forced ahead into the soil. Knowing the orientation of the slanted face of the drill bit allows the direction of the bore to be altered as the bit is pushed into the soil. Alternating between pushing without rotation then rotating the drill string produces a curved bore alignment.

For the mud motor system, the drill bit is attached to a rotary motor that is driven by the drilling fluid forced down the drill string. The mud motor is attached to a precisely-curved piece of drill pipe (bent sub) that provides the ability to steer the string. Knowing the orientation of the bent sub allows control of borehole direction. Advancement of the borehole is continuous as the drill bit rotates and the drill string follows the direction of the bent sub. For a straight bore path, the entire drill string is slowly rotated at 20 to 30 rpm while the mud motor spins the drill bit at up to several hundred rpm. To change direction of the bore, rotation of the drill string is stopped when the bent sub is pointed in the desired direction. The mud motor spins while the entire drill string is pushed ahead (Blakita, 2000).

Although neither pipe installation system offers any distinct advantage over the other, the HDD system itself offers a number of advantages related to the soil stabilization activity, compared to drilling and grouting from the tunnel face or surface drilling:

- Fewer surface distruptions,
- Elimination of surface construction noise, dust and traffic distruption.
- Less environmental impact.
- Subsurface boring eliminates the risk of damage to underground structures and utilities.
- Pre-placement of TAM pipes is possible, allowing grouting to be performed ahead of the tunnel face after mining has begun, without detriment to subsequent mining operations.

Horizontal grout pipe placement also aligns the TAM pipe parallel to the treatment zone. This provides an efficient canopy of overlapping, horizontal grout cylinders over the tunnel arch. There are, however, a number of disadvantages also. In addition to the higher cost per installed unit of length of TAM pipe, this method requires:

- Stricter alignment tolerances, since hole lengths are typically 150 m or greater.

- The use of highly precise, and costly, borehole surveying techniques, such as gyroscopic-based systems, to ensure an accurate bore path so that the permeation program is meeting its objectives.
- Contingency planning for misaligned grout pipes. This could have a significant impact on cost and scheduling since additional pipes may have to be installed.

The higher cost of HDD for TAM installation limits its practical use. Nevertheless, this new technology has proven to be economically feasible for specific tunneling applications and may also be considered for bottom sealing under hazardous waste sites where surface drilling is prohibited.

High quality permeation treatment of the ground can be achieved only if all the points of the work are defined, understood and integrated. It is essential to be clear about what treatment is expected to achieve. Next the target grout has to be examined and its properties quantified. The important details are fabric, and the permeability of major elements of the fabric. Standard site investigations rarely yield this information since they are designed primarily for determining the strength and stiffness of the ground. Therefore special investigations designed for the grouting concept are needed. Permeation injections are governed by the detail of the ground adjacent to individual injection pipes. The smaller the volume of ground to be treated, the more critical an accurate description of the precise volume of the soil to be treated becomes.

Next it is important to choose a grout capable of permeating the target ground. The physical constraints on grout penetration described above must be satisfied. Grouts and ground must be properly matched. This also implies taking into account limits on injection pressures before fracturing occurs. At shallow depths injection pressures necessarily must be low and if exceeded fracturing may occur.

2.2.2 Compaction grouting

In 1980, the ASCE Grouting Committee, in its glossary of terms, defined compaction grouting as: 'Grout injected with less than 1 in. (25 mm) slump. Normally a soil-cement grout with sufficient silt sizes to provide plasticity together with sufficient sand sizes to develop internal friction is used. The grout generally does not enter soil pores but remains in a homogeneous mass that gives controlled displacement to compact loose soils, gives controlled displacement for lifting of structures or both (Figure 2.9).

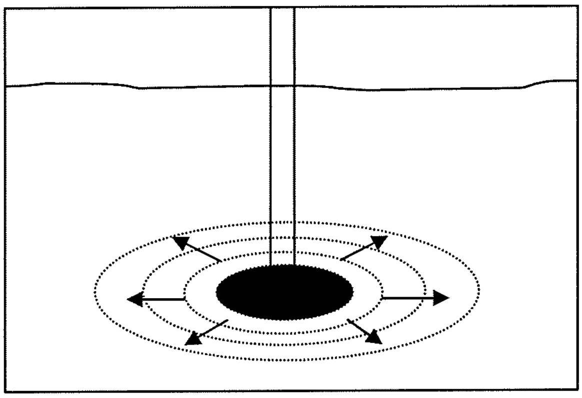


Figure 2.9 Schematic view of compaction grouting

Since its introduction over 40 years ago, compaction grouting has been commonly used for filling voids, jacking-up or underpinning slabs and footings, and controlling settlements (Mitchell, 1981; Welsh et al. 1987; Graf, 1992). Warner and Brown (1974) and Bandimere (1996), for example, described successful rehabilitation programs to correct detrimental settlement in residential and commercial buildings. Baker et al. (1983), and Harris et al. (1994) describe an application of compaction grouting to control loss of ground movements during tunneling operations for subway tunnel sections. McGovern (1996) described the use of compaction grouting to repair damage caused by the failure of an 84-inch diameter sewer pipe. Garner et al. (1998) reported on the remediation of sinkholes in a dam foundation. To demonstrate effectiveness in Karst terrain, Henry (1989) presented a case history describing the remediation of sinkhole development during the expansion of a landfill. Within the past 15 years, the procedure has also been used to mitigate soil liquefaction, although experience with this type of application is limited. However, interest in this use of compaction grouting is likely to continue to increase since the procedure offers the following advantages: effecient treatment of a localized zone or layer at a significant depth, compact equipment and limited work space requirements, limited construction spoils and waste water, and minimal vibration impact on existing structures. During site preparation for the construction of a power plant Boulanger and Hayden (1995) reported successful densification of deep, loose fill deposits using a combination of dynamic compaction and compaction grouting and summarized several other case histories that highlighted compaction grouting as an effective means for treating liquefiable soils. Baker (1985) investigated compaction grouting as an effective method for in-place densification of loose, liqueifiable soils below two existing dam embankments.

Fundamental to the success of the procedure, is deposition of the grout in such a manner that it remains in a globular mass in the vicinity of the point of injection. Regardless of the stiffness of the grout, if it is overly mobile, hydraulic fracturing of the adjacent soil can occur, resulting in loss of control of the intended compaction. Therefore, proper performance must prevent the occurrance of hydraulic fracturing, which will not only interferes with orderly compaction, but can also result in unacceptable displacement or other damage if it occurs in near proximity to overlying or underground structures. Compaction grouting has been successfully used in almost all types of soil, although special considerations must be provided for work in soft clays where slow drainage will likely cause high pore pressures that require special attention.

The first publication on the procedure (Graf, 1969), presented a theoretical description of the process hypothesizing that the injected masses would be generally spherical, and would densify the injected soil radially in all directions. The first report describing the actual mechanism and providing data derived from the excavation and removal of full scale test injections was made by Brown and Warner (1973). Since this early work, a large number of injected masses have been exposed and evaluated in research demonstrations in connection with actual or proposed projects.

A variety of other evaluations of the procedure are the investigation of the lateral forces exerted upon adjacent sub-structures, changes in the treated soil density utilizing standard penetration or cone penetrometer tests before and after grout injection, large scale load tests of foundations over treated ground and long term settlement monitoring of structures overlying treated soils.

Perhaps the single most important parameter to assure effective soil improvement, is obtaining a globular grout mass which will typically be either columnar or tear shaped. Cracking or hydraulic fracturing of the soil mass, with a resulting thin lens of grout should be avoided. One measure of the regularity of the mass is the Travel Index, (TI) of the grout. The TI is the maximum radial travel of the grout from the point of its injection, divided by the minimum radial distance to a grout-soil interface, and is thus representative of the grout's tendency to remain in a controlled mass and at the intended location. Grouts with low TIs, (less than about three) remain in relatively symmetrical masses that do not result in hydrofracture, whereas hydrofrcature is almost assured with a grout of TI exceeding about five.

The importance of the resulting grout mass shape, was first reported by Brown and Warner (1973). A research program performed in the 1950s, which involved the injection and subsequent excavation of over 100 test grout columns. The work included a variety of different grout mixtures and consistencies, sand materials and injection rates. In the report, it was concluded that grouts containing fine sand combined with about 12 % cement and water to form a very stiff mortar like mixture were optimum. Also it was reported that a slower pumping rate results in significantly higher grout takes. The same authors, Warner and Brown (1974) emphasized the great importance of the grout composition and limitation of clay. An entire section was devoted to description of the 'sand material' which included a gradation envelope as shown in Figure 2.10. It should be noted that the gradation indicates zero allowance for clay size constituents. It was emphasized once more that 'it is preferable to use the least amount of water that will provide a very stiff plastic consistency grout. A rule of thumb is the stiffer the grout, the more effective its injection will be. They further warned of the risks of excessive pumping rates which could result in 'rupture' of the soil. The pumping rates must therefore be rigidly controlled. Rates exceeding 0.06 m^3 (2 ft^3) per minute should be prohibited. On sensitive works specifications of a much lower pumping rate will often be advisable.

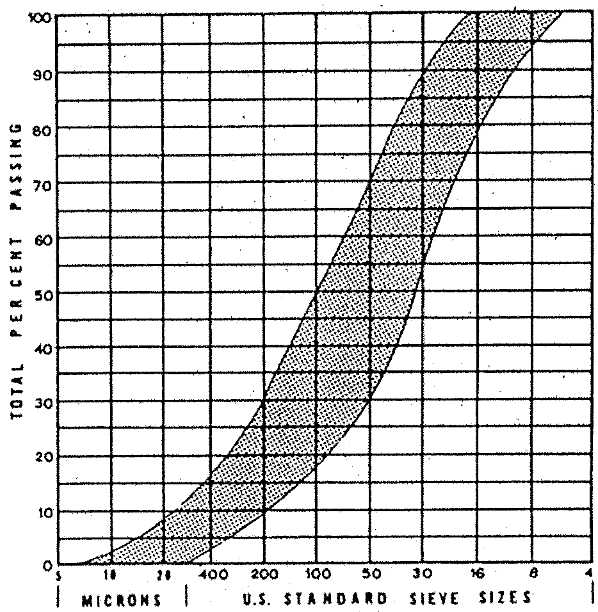


Figure 2.10 Prefered limits for sand used for compaction grout (Warner & Brown,1974)

Compaction grouting may proceed in stages from near the ground surface, downwards to a firm stratum, or from depth to the ground surface (Figure 2.11). According to Stilley (1982), if underpinning is the objective, then grouting downwards in stages beneath the building is more succesful. Grouting from the top downwards results in a greater grout – take per hole, and a more complete densification of the lower soils because higher pressures can be used after grouting the overlying soil. Indeed, when a problem soil extends to the ground surface, downward grouting is more or less mandatory in order to prevent grout escaping at the surface.

Typical pressures at the point of injection vary from 350 kPa to 1700 kPa when injection is within 2 m from the surface and to over 3500 kPa when grouting takes place at depths greater than 6 m. Pressures above 4200 kPa are seldom exceeded irrespective of depth, although pressures as high as 7000 kPa are sometimes needed to initiate grouting in a tight hole (Bell, 1993).

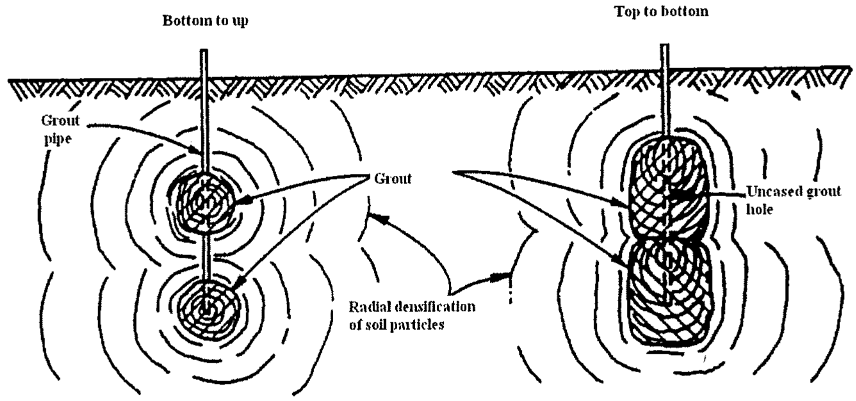


Figure 2.11 Performance of compaction grouting from bottom to top and top to bottom (Graf, 1992)

The spacing and the location of the injection holes depend upon the soils to be treated, the depth, the project requirements and judgement. Analysis and field observations indicate that the size of the grout compaction zone depends upon:

i. the restraining pressure of the soil
ii. the weight of the structure above the grout bulb
iii. the surface area of the grout bulb
iv. the grout pressure at the bulb (Graf, 1992)

In most cases, the horizontal spacing of the injection points is greater in zones with a high overburden pressure (deep injection points or beneath a structural load), as higher grout injection volumes and pressures can be used. Injection into loose soils generally produces a large grout bulb and the required horizontal spacing is thus greater than for denser soils.

A grout injection grid and sequence of injections are often designed with primary, secondary and tertiary injection phases (if required). Normally, the primary injection points are positioned on a regular grid and injected first. The secondary injection points are positioned midway between these to compact the ground further. In certain locations a further tertiary injection phase might be required. The injection points are positioned midway between the secondary and primary injection points (Figure 2.12)

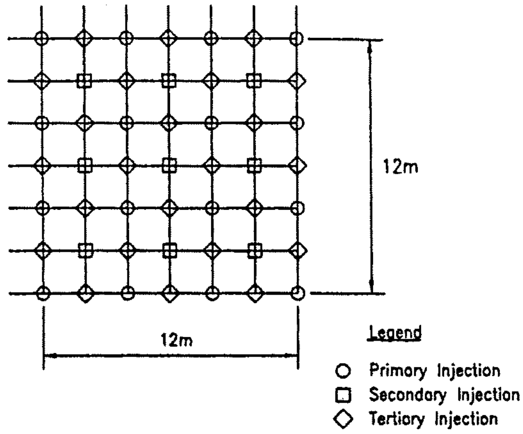


Figure 2.12 Compaction grouting injection grid (Baez and Henry, 1993)

Perhaps the most significant advance in the use of compaction grouting has taken place in seismic regions. Over the past decade, the technique has been :

- applied on relatively open sites in conjuction with other site improvement techniques, such as vibro replacement stone columns or dynamic compaction, for liquefaction mitigation,
- applied on restricted sites as a single site improvement technique to densify soils for liquefaction mitigation,
- applied as a remedial tool for seismic retrofit.

Open sites: Typically, on large, relatively open sites, vibro stone column construction or dynamic compaction is used to achieve the majority of the required improvement. However, such sites may also include surface or underground structures sensitive to the vibrations induced by these methods. Alternatively, overhead obstructions may preclude the use of large equipment. In these circumstances, compaction grouting, with its small, maneuverable and vibration-free equipment, is effective in achieving the improvement needed without adversely affecting surrounding features.

Restricted sites: More and more improvement for liquefaction mitigation is needed on smaller, urban sites completely bounded by existing structures, where vibratory methods or pile driving are not viable options. Compaction grouting may be used as a single site improvement technique, avoiding the cost of removal and replacement.

Seismic retrofit: Where existing structures are to be expanded, or their use has been revised, or building codes have been upgraded to meet new seismic considerations, compaction grouting is being increasingly used as a remedial tool for liquefaction mitigation. Of particular note is the ability of this technique to be applied to treat only the liquefiable zone, or zones, between competent material.

Boulanger and Hayden (1995) also present a comprehensive summary of published case histories involving compaction grouting for liquefaction mitigation. These case histories indicate that compaction grouting can be effective in increasing the standard penetration test (SPT) and cone penetration test (CPT) resistance of soils ranging from silty sands to silt. However, it should be noted that the experience available to assess the suitability of compaction grouting for liquefaction mitigation at a given site is limited. Furthermore, very few data are available on the effects of time on penetration resistance after compaction grouting.

Boulanger and Hayden (1995) presents a case history to illustrate the benefits of compaction grouting for mitigation of liquefaction potential in a clayey silt and a silty sand, and the effects of time on the penetration resistance of the treated soils. For this purpose CPT soundings one week, one month, and 18 months after treatment were utilized. Based on the results of the test section, it may be concluded that compaction grouting was successful in increasing the SPT and CPT resistance of the silt and sand. The level of improvement observed in the silt is particularly encouraging since few other improvement methods are as effective in this type of soil.

A large increase in the penetration resistance of the silt and sand was observed. However, up to about 30 % of the average increase was lost within the following 18 months. The available data suggests that the decrease in penetration resistance may be associated with a reduction in lateral stresses with time. Since such changes in lateral stress are likely to affect the liquefaction resistance of the treated soils, it is necessary to carefully consider the time effects following compaction grouting for liquefaction mitigation (Figure 2.13).

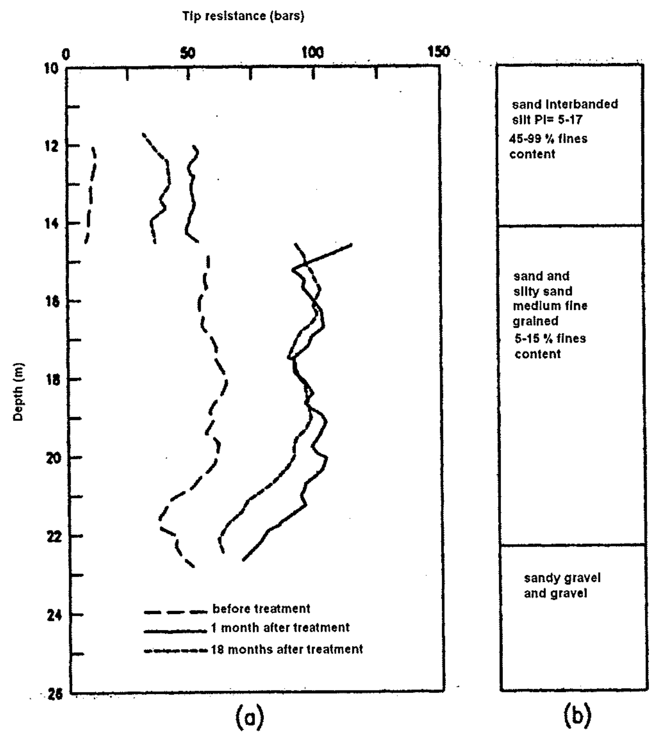


Figure 2.13 Average CPT tip resistances prior to and after compaction grouting (Boulanger and Hayden,1995)

2.2.3 Fracture grouting

Fracture (hydrofracture) grouting is a relatively new technique that originated in Europe. The method simply involves locally confined and controlled fracturing of a soil unit by injecting a stable but fluid, cement-based grout at high pressure, for example, up to 4 MPa (580 psi) (Bruce, 1994) (Figure 2.14). The process is applicable to stabilization and compensation grouting of fine grained soils with low hydraulic conductivity where permeation grouting is not effective or is limited. The original development of fracture grouting was to compensate for soil losses and subsequent settlement during tunneling or excavation activities (Figure 2.15). The process evolved into a soil stabilization technique for the purpose of preventing future settlements due to an unstable soil mass.

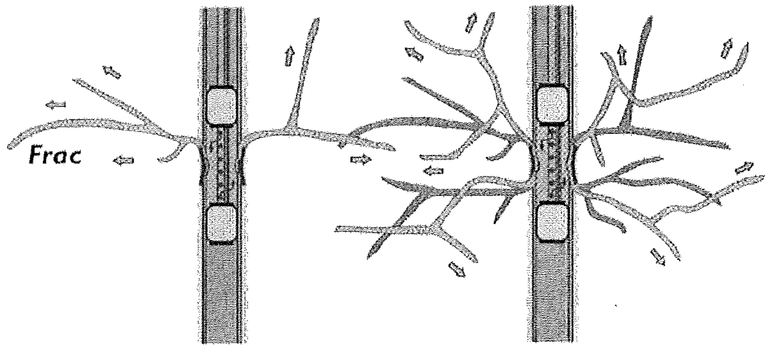


Figure 2.14 Fracturing of soil by injecting cement-based grout at high pressure (Keller, 2001)

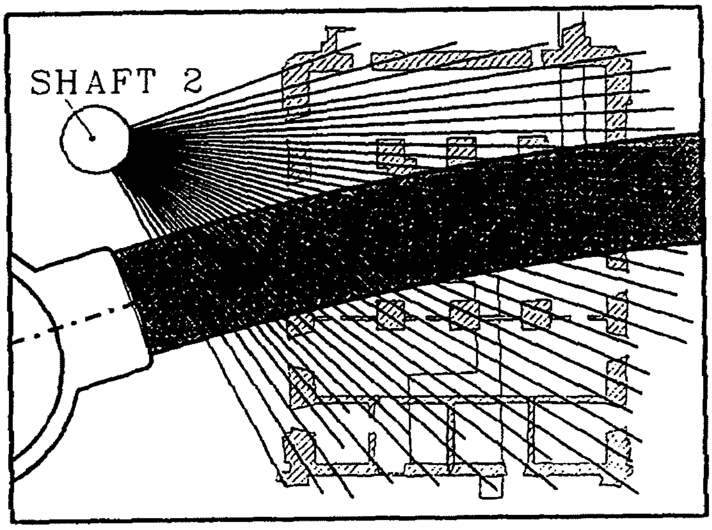


Figure 2.15 Fracture grouting to compensate for soil losses and subsequent settlement during tunneling (Falk, 1997)

The premise of fracture grouting is to inject a grout under pressure to hydrofracture or open pre-existing fractures within the in-situ soil. The grout filled fractures create grout lenses which densify and reinforce the soil mass. Grout is initially injected at pressures required to create soil failure, which in turn fills the fracture. The magnitude of the fracture, with regard to length, width, and volume is a function of hydraulic gradient established by the grout injection pressure and in situ soil overburden pressures.

Soil fracture grouting relies on the propagation of fractures by injection of a low viscosity grout at pressures higher than the hydrofracture pressure. At the initial stage, the grout pushes the soil outward and forms a ball as shown in Figure 2.16. When high viscosity grout is injected into normally consolidated clay, the clay deforms plastically and the grout ball continues to grow. As the grouting pressure is increased, the size of the ball increases rapidly until a plane of weakness is formed by hydraulic fracturing. The stress condition in the soil suddenly changes and the injection pressure drops dramatically. When a low viscosity grout is used, the grout will intrude into the plane of weakness to extend the fracture. When high viscosity grout is used, the grout may not be able to get into the plane of weakness and the grout ball may continue to expand so that the injection pressure starts to increase again (Jafari et al., 2001).

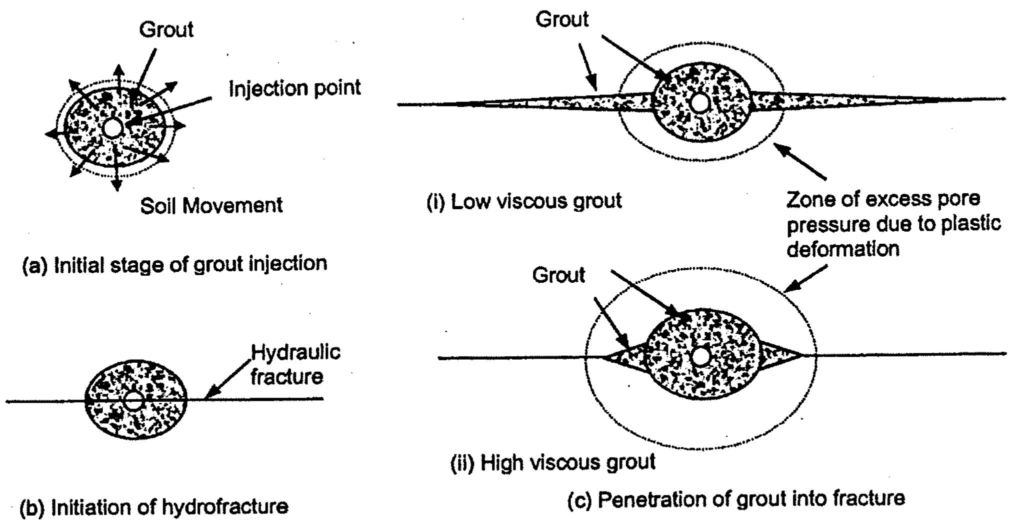


Figure 2.16 Entrance of grout and propagation of fractures (Jafari et al., 2001)

In theory, if a soil is normally consolidated, the first grout lens produced will propagate in the vertical direction (direction of major principal stress), which will cause horizontal pressures to increase (Figure 2.17) and hence cause compression in the soil (Raabe and Esters, 1990). If further injections are undertaken until the principal stresses are reoriented, horizontal fractures will be propagated and eventually (sometimes rapidly) lead to heave. In practice, the direction of the initial fracture is often controlled by the pre-existing defects, or weak zones within the soil or weathered rock mass, and not the existing state of stress (Rawlings et al., 1998).

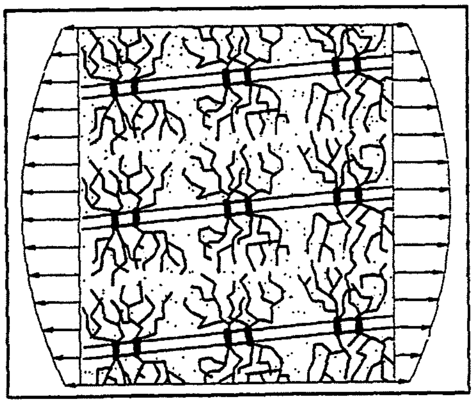


Figure 2.17 Formation of vertical fractures and horizontal prestressing (Falk, 1997)

Initially the grout forms a bulb that continues to grow, until the principal stresses prevailing in the soils are overcome. At this point a pressure drop occurs, and flow remains constant or increases, depending on the fracture that has developed (Figure 2.18). The dynamics of this process are complicated and often difficult to predict. The process, therefore, requires continuous real-time monitoring of structural movement and the grouting parameters to adequately and safely control the fracture grouting operation.

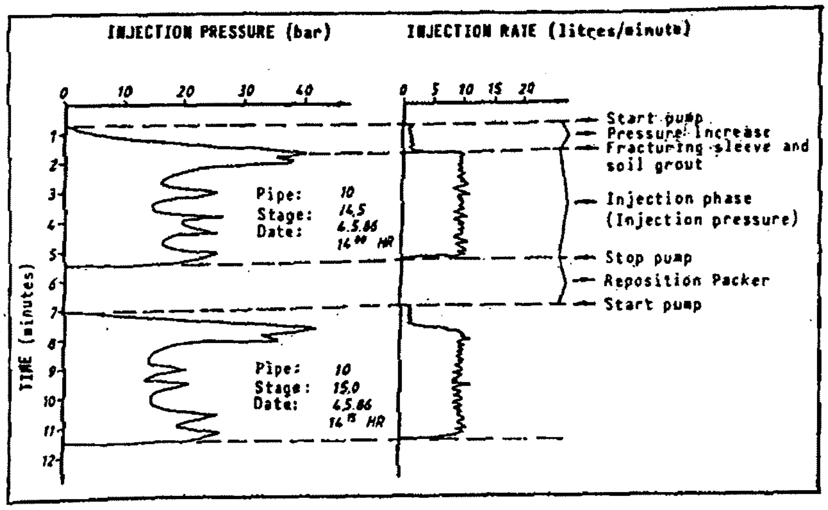


Figure 2.18 Drop in injection pressures means fracturing the soil (Raabe & Esters, 1986)

The grout delivery system typically involves the installation of sleeved pipes called tube-a-manchette's (TAM's) that are drilled, driven or jacked into the soil in some predetermined array designed to provide adequate grout zone coverage of the soil mass (Figure 2.19). TAM's consist of steel or plastic pipes with one-way sleeved valves spaced at 0.3 to 1.5 meters along the pipe alignment through which grout is injected and if necessary re-injected. A double pneumatic packer assembly is used to seal off the individual sleeved valves, through which grout is injected for the purpose of treating the surrounding soil.

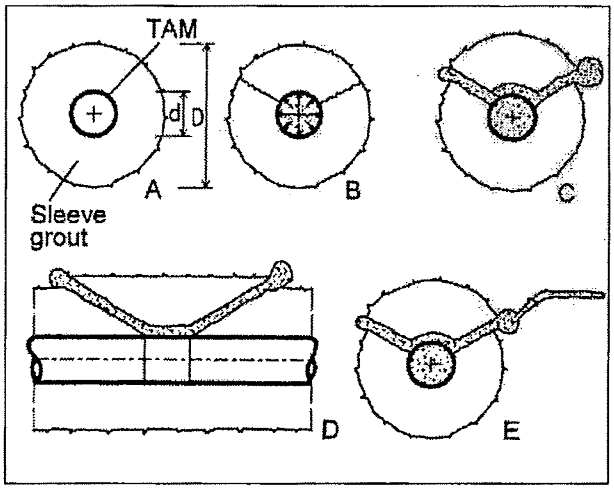


Figure 2.19 Application of soil fracture grouting using TAMs (Chambosse & Otterbein, 2001)

Soil fracture grouting by nature is a technique that has the potential to cause significant structural damage to adjacent structures and tunnel linings. Once the fracture grouting is initiated the in situ soil is subjected to internal pressure, and soil displacement will eventually propagate into surface heave. Controlled fracturing of soil requires real-time electronic monitoring of the ground surface or buildings to detect movement. Based on the monitoring data, informed decisions can be made and the fracturing of the foundation soils can be performed with confidence.

Until recently, the use of soil fracturing had been limited to the French grouting industry. The first successful application of soil fracture grouting in North America was on the St. Clair River Tunnel (Drooff et al., 1995). On the St. Clair project a soil-fracturing method trade-named 'Soilfrac' was used. The project consisted of excavating a 9.2 m (30 ft) diameter soft-ground tunnel using an earth pressure balanced (EPB) tunnelling machine. The tunnel allignment passed under an oil refinery research building approximately 7 m (23 ft) below its footings. The predicted settlements in the soft St. Clair till were in excess of 100 mm (4 in.). The project specifications required maintaining the research building within 10 mm (3/8 in.) of its original position.

The soil fracturing program consisted of two phases. In the first phase, called 'preconditioning grouting', soil fracture grouting was used to reinforce the soil below the research building and preheave or raise the building 5 mm (3/16 in.) to compensate for the anticipated settlement. The phase 1 holes were drilled horizontally under the research building from two work shafts installed specifically for the purpose. Two layers of fracture grouting holes were drilled from each shaft. A total of four layers of grout holes overlapped under the research building. The grout pumps for the project were developed to allow for a controlled injection of cement grout with a flow rate of between 4 and 30 L/min with a maximum operating pressure of 70 bars (Drooff et al., 1995).

Phase 2 of the program was to use fracture grouting during the actual tunneling operation beneath the research building to compensate for settlement. The building was heavily instrumented with electrolevels to measure and record settlement during tunneling. The instruments recorded a slight heave of 2-3 mm as the EPB machine mined underneath the research building. This heave disappeared when the mining operation passed beyond the building. Subsequently, after 24 hours, some settlement did occur. The settlement was on the order of 4-6 mm. However, the research building still had a net heave of 4-6 mm. Because of these favorable results it was not necessary to perform the phase 2 fracture grouting underneath the research building.

2.2.4 Compensation grouting

Compensation grouting has been defined by a number of sources (Rawlings et al., 1998) but it can be broadly described as the introduction of a fluid or semi-fluid material into the ground, increasing the local volume at the point of injection. This in turn causes movement by expansion of ground away from the area of injection, either compensating for movement in an opposing sense or causing movement to accumulate in the direction of expansion.

Compensation grouting can involve two different modes:

- Soil Fracture Grouting
- Compaction Grouting

When considering the use of compensation grouting for tunneling or other processes, due consideration must be given to the way the settlement is propagated. Figure 2.20 shows a typical settlement trough that will result when a 7-m diameter tunnel is bored through granular material at a depth of 25 m below the ground surface (Essler et al., 2000).

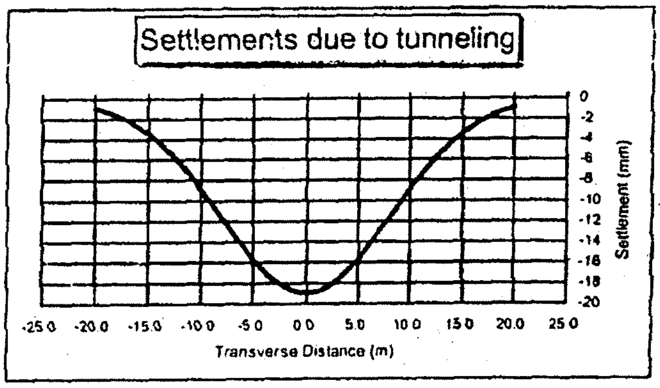


Figure 2.20 Settlement trough due to single tunnel (Essler et al., 2000)

Positioning of any compensation grouting zone needs not only to consider this transverse effect but also a similar effect in advance of the tunnel. Figure 2.21 demonstrates the dangers of ignoring this effect.

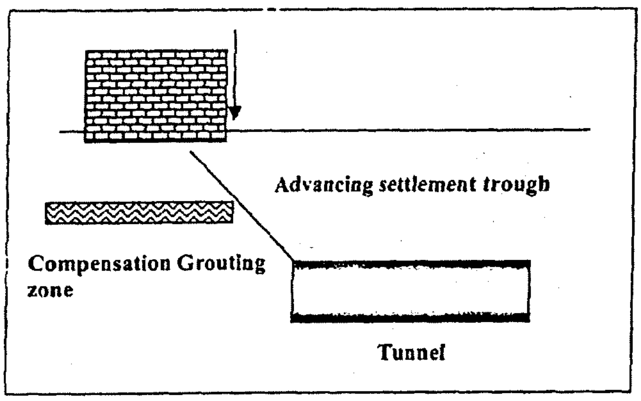


Figure 2.21. Position of compensation grouting zone (Essler et al., 2000)

Clearly the compensation grouting zone needs to be extended towards the advancing tunnel in order to intercept the forward settlement trough.

When a building is offset from the tunnel this effect still needs to be considered and the zonal placement shown in Figure 2.22 should be utilized.

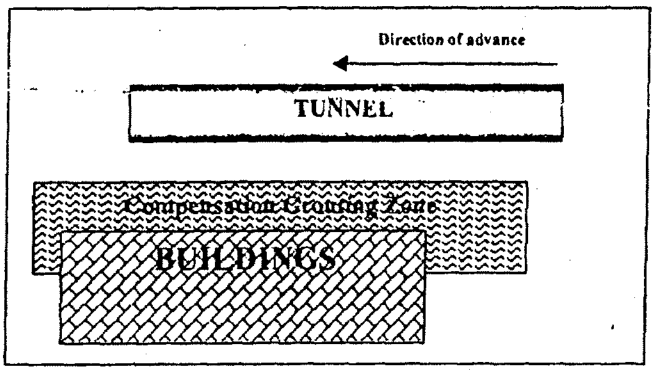


Figure 2.22 Plan view showing the relative position of compensation grouting zone (Essler et al., 2000)

The relative position of the foundations also needs to be considered as piled foundations may be close to the tunnel and hence may undergo both larger and more localized differential settlements. This is shown in Figure 2.23.

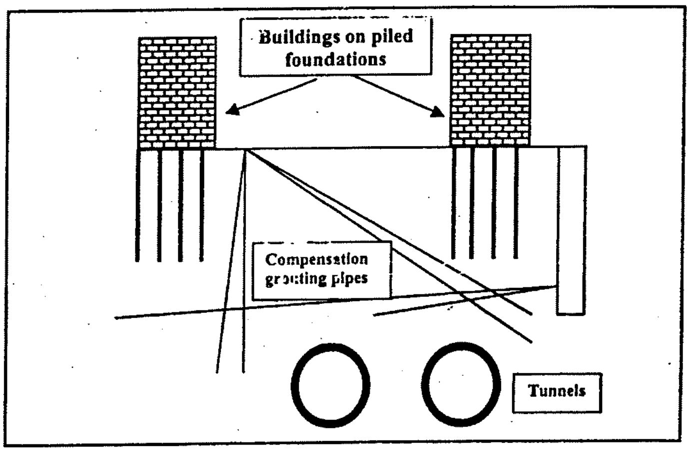
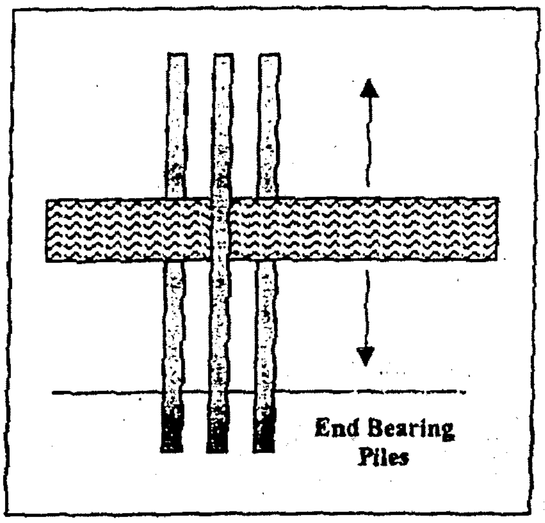


Figure 2.23 Section showing effect of piles on grouting zone (Essler et al., 2000)

In summary both building fabric, foundation level and relative tunnel position need consideration when designing a compensation grouting programme.

Compensation grouting in close proximity to piles needs careful consideration. Piles may be moved or not by the compensation grouting process depending on the relative position of the pile and the compensation grouting zone. Compensation grouting will cause the underlying ground to move during preconditioning (the injection phase in advance of tunnelling) and this needs to be designed with regard to the piles.

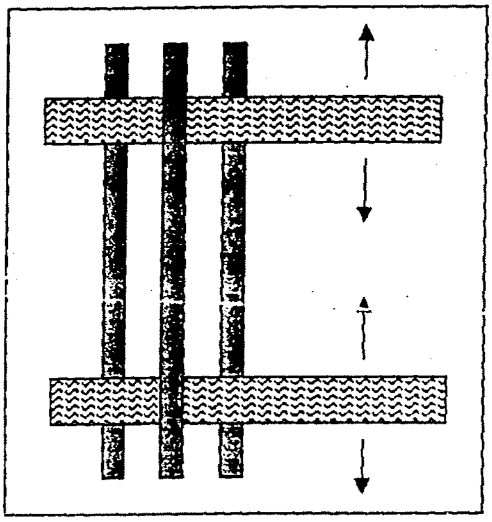
Generally if the compensation grouting zone is below the pile toes then the piles will move as the upward force is below the pile. If the zone is between the pile cap and the pile toe then the movement will be determined by the combination of ground conditions and pile design. If compensation grouting is carried out in very soft clays above end-bearing piles then skin friction support will be small and the ground will move relative to the piles. This is shown in Figure 2.24.



Side friction is low for end bearing piles and hence the relative movement of the ground will exceed the side friction on the piles and no movement of pile results.

Figure 2.24. End bearing piles (Essler et al., 2000)

If the piles are designed as frictional then they may or may not move depending on the relative position of the zone. The piles will move if the frictional support above the zone exceeds the frictional support below the zone. Figure 2.25 demonstrates this.



The zone near the top of frictional piles will not be able to move the piles as there is insufficient friction above the zone to fail the side friction below the zone. The zone near the toe however will move the pile as the side friction above the zone will exceed that below.

Figure 2.25 Frictional piles (Essler et al., 2000)

Dissipation of pore pressure when carrying out compensation grouting in clays needs careful consideration, particularly when working with soft clays. Figure 2.26 shows pore pressure dissipation at the end of trial injections in London clay, an overconsolidated very stiff clay. Dissipation was rapid, returning to original values within two weeks.

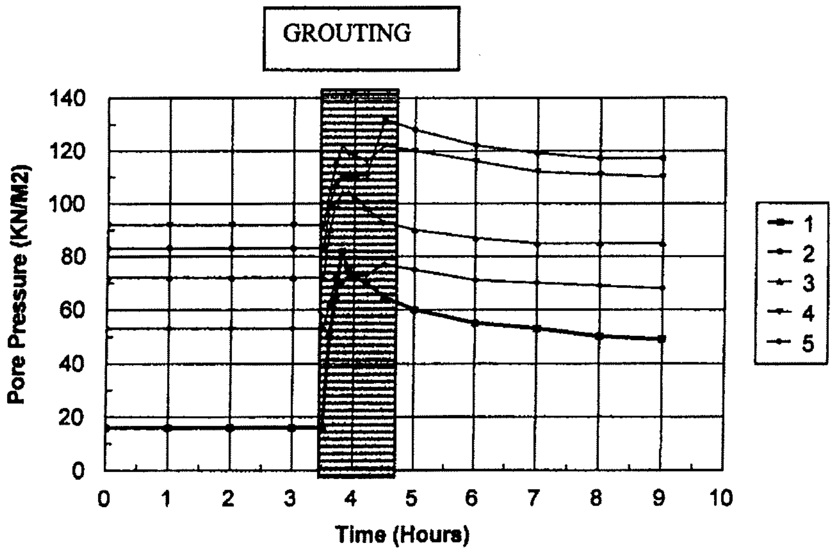


Figure 2.26 Pore pressure changes during injection in London clay (Essler et al., 2000)

This rapid dissipation of pore pressure in a stiff clay should be compared with the pore pressure response for injection in a soft clay which is shown in Figure 2.27. In this case, repeated injections were carried out through the summer, causing a steady build up in pore pressure. If the final section is replotted on a log scale (Fig. 2.28) it can be seen that around 1000 days may be required to completely dissipate the excess pressure (Essler et al., 2000). This is shown in Figure 2.27.

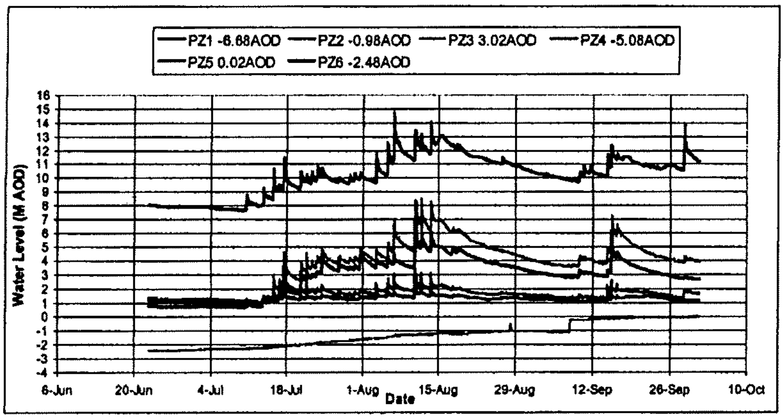


Figure 2.27 Pore pressure response in soft clays (Essler et al., 2000)

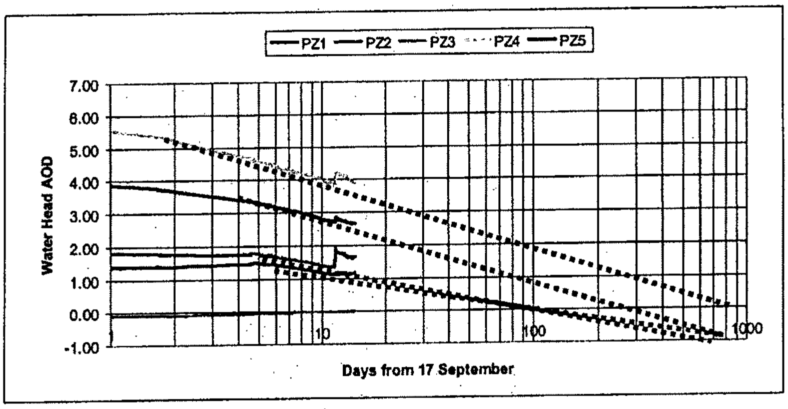


Figure 2.28 Pore pressure response in long term (Essler et al., 2000)

Pore pressure dissipation in soft clays will result in some long term settlement following tunneling and will probably require further grout injection to correct for this settlement.

2.2.5 Jet Grouting

Jet grouting is a relatively new technique developed since the 1960s (Welsh, 1991) in which thin high-pressure jets of cement grout are discharged laterally into a borehole wall. The cutting action of the jets can be enhanced by incorporating compressed air.

As drill rods are slowly turned and lifted (with a good deal of precision), the injection erodes and mixes the soil with a cementicious slurry to create columns or panels of a soil/cement product known as 'soilcrete'. Therefore, this technique involves *mixing* the soil rather than injecting. The high- pressure injection of the grout slurry through the nozzle allows changes in the geotechnical characteristics of the soil to suit the specific requirements of the project. The result is column of modified soil with low permeability and improved strength. Figure 2.29 illustrates the three basic systems of jet grouting.

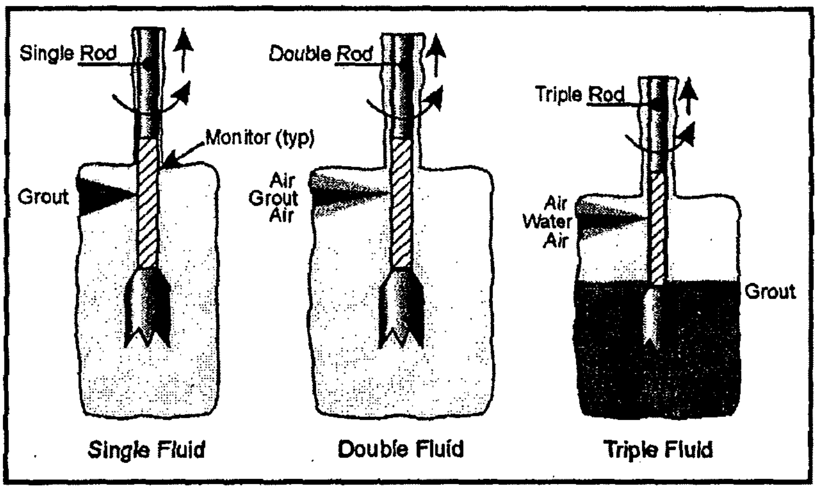


Figure 2.29 Basic jet grouting systems (Welsh and Burke, 2000)

Jet grouting is a very adaptable ground treatment technique that can be used for a wide variety of applications for either temporary or permenant works. Applications include:

- providing foundations for structures to be erected
- underpinning existing foundations and structures (Figure 2.30a)
- vertical and horizontal barriers to fluid flow (Figure 2.30b)
- formation of retaining or support walls
- solidification or containment of contaminants within the ground (Figure 2.30c)
- stabilisation of embankments (Figure 2.30d)
- remedial works to dam cutoffs, dam curtains or cofferdams
- break-ins or break-outs for tunnels

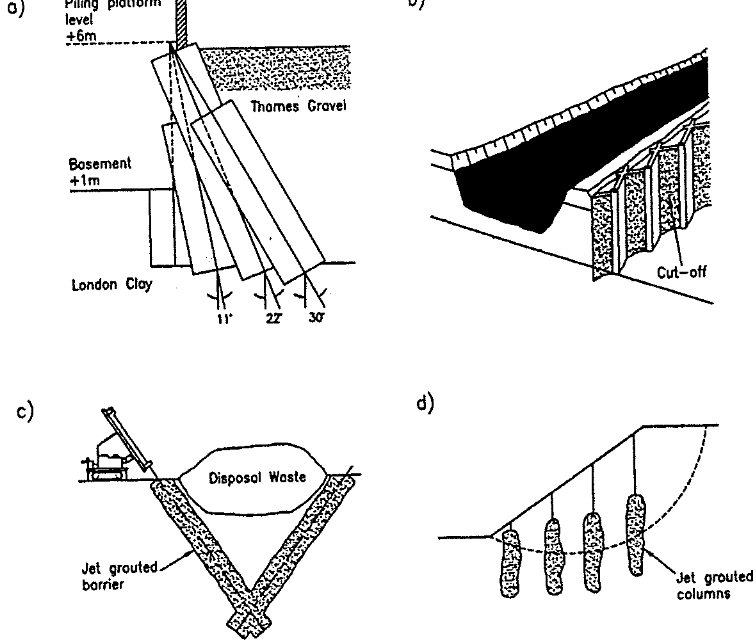


Figure 2.30 Applications of jet grouting: a) underpinning of foundations (Wheeler, 1996); b) cutoff wall; c) leachate barrier; d) stabilization of an embankment

However, there are three main limitations of jet grouting:

- It is important to maintain good control of the jet grouting process in regard to spoil return so as to minimise or eliminate ground heave. Excessive ground heave can damage underground services or adjacent structures or foundations.
- Spoil return can become blocked; if jetting persists, hydrofracture of the ground may occur.
- Large quantities of slurried spoil can be produced, e.g. a jet grouting unit typically generates up to 50 m^3 of spoil per shift. Of this up to 33 % by volume would be solid material, depending on the ground and jet grouting process selected.

The single jet process is the simplest technique. The grout is injected into the ground at high pressures (600 bars) from one or several 5-10 mm diameter nozzles on a hollow stem grout pipe. The grout jet erodes the soil and the grout mixes with the

spoil. Excess soil and grout slurry is forced up the annulus between the grout injection pipe and the borehole casing to the surface.
Double jet process is an extension of the single system process in which the grout jet is shrouded in compressed air at typically 2-15 bars. A greater volume of soil is treated. The air shroud is found to improve the cutting ability of the jet.

Triple jet process uses a water jet, shrouded in air to erode the soil and a separate nozzle for grout injection. A triple concentric pipe is required to inject the air, water and grout at typical presssures 2-15, 40 and 5-30 bars respectively.

The diameter of the jet grouted column depends upon the jet process, soil type and erodibility, pressure and injection rate of jet, rotation and the lift speed.

The strength of the final column is a function of the grout mix characteristics, degree of soil and grout mixing and the type of the soil.

2.3 Rheological properties of cement-based suspension grouts

The properties of cement-based grouts can be classified into two categories:

- Fluid characteristics: cohesion and its evolution with time, thixotropy, viscosity, pressure filtration coefficient, bleed, and initial and final gelation.
- Set characteristics: initial and final set-time, unconfined compressive strength, durability, resistance against chemical attack, and permeability coefficient.

Properties and characteristics of suspension grouts can be significantly altered by means of additives and admixtures. The grouting engineer must have a good understanding of the characteristics and the function of each component in the grout to formulate a balanced, stable, suspension grout with the desired rheology and set characteristics. It is obvious that the filling of a karst requires a different formulation than the filling of fine fissures in a rock formation or structure. Grouts placed in flowing water must have higher cohesion than grouts placed in stagnant water or in dry circumstances.

2.3.1 Fluid characteristics of cement-based suspension grouts

2.3.1.1 Viscosity, Cohesion and Thixotropy

Publications by Khayat and Yahia (1997), Skaggs et al. (1994), Naudts (1994) and Lombardi (1985) provide excellent background information on these matters.

The cohesion, apparent viscosity and thixotropy of a grout are related but impact differently on the rheology of a fluid. Cohesion, is a measure indicating how far and how easily the Binghamian fluid will flow through pore channels of a given size under the given pressure. Thixotropy relates to the time in which a certain cohesion value is obtained and the impact of the shear rate in obtaining this increase in cohesion. Viscosity is strictly a measure of flowability, and can be measured with a coaxial cylindrical viscosity meter or even with a Marsh cone. The latter two properties are closely related: the higher the cohesion, the higher the apparent viscosity. Ideally, admixtures only marginally impact on the cohesion or viscosity at moderate to high shear rates, but cause a major increase once the grout moves slowly or is at rest. These grouts are often referred to as 'stable suspension grouts with shear thinning rheology'.

It has been recognized that the behavior of suspension grouts can be characterized as a Binghamian fluid, as opposed to a Newtonian fluid. In a Binghamian model, interparticle forces between the solids result in a yield stress that must be exceeded in order to initiate flow. The shear stress 'τ' represents the transition between fluid-like and solid-like behavior. The cohesion represents the resistance to flow until a minimum force is applied. The cohesion 'c'- more so than the viscosity – is the property that prevents suspended particles from settling, or a grout from sagging in wide apertures.

It is clear from equation (2.1) that cohesion, plastic viscosity, and shear rate influence penetrability of a suspended grout. Therefore, the higher the cohesion and viscosity the higher the shear stress and hence the grouting pressure that will need to be applied to move the grout through the pore spaces.

$\tau = c + \eta_B\ (dv/dx)$ or $\eta(dv/dx)$..(2.1)

Where: τ: shear stress; c: yield value or cohesion; η_B: plastic viscosity; dv/dx: shear rate; η: apparent viscosity.

The ease of placement and susceptibility to wash-out of a grout are directly related to its thixotropic properties. Thixotropic agents impart a unique property to the cement-based suspension grout – as the shear rate applied to the grout falls there is a rapid increase in cohesion and viscosity. As the grout is being placed, the velocity and hence the shear rate drops at the fringes of the grout cylinder, and the cohesion and viscosity rise substantially. This in turn increases the resistance to washout but reduces the ability to flow. Even at very low concentrations (0.05 % by weight of cement or less) pre-hydrated biopolymers used in the base mix design provide a significant thixotropic effect. When these admixtures are properly prehydrated and neutralized, they are very potent to produce suspension grouts with shear thinning rheology.

2.3.1.2 Resistance against pressure filtration

If circumstances permit grouting with high pressures the grout's penetration can be enhanced if the grout has good resistance to pressure filtration. If an unstable grout is used, the grout will pressure filtrate (dry pack) causing a considerable reduction in penetration.

The pressure filtration coefficient is defined as the volume of water lost in the pressure filtration test divided by the initial volume of grout in the cylinder of the apparatus, divided by the square root of the filtration time in minutes (See Eq. 2.2).

$$K_{pf} = \frac{V_{WATER}}{V_{GROUT} * (t_{filt}(\min))^{0.5}} \quad \text{...(2.2)}$$

Neat cement grout mixes and unstable mixes have very poor resistance to pressure filtration. During grouting, unstable mixes will be subjected to substantial and almost immediate water loss where flow channels are narrow. The remaining grout forms a filtrate in the pores or fissures close to the injection point and refusal is reached quickly.

A balanced suspension grout refers to a grout that has a good resistance (low K_{pf}) to pressure filtration and a low to moderate cohesion. The relationship between pressure filtration and cohesion, based on recent in-house testing, is illustrated in Figure 2.31. Regular stable cement-based grouts have a low K_{pf} but high cohesion will limit their grout spread. By introducing high range water reducers the cohesion of the grout can be reduced without adding water, thus maintaining high resistance to pressure filtration.

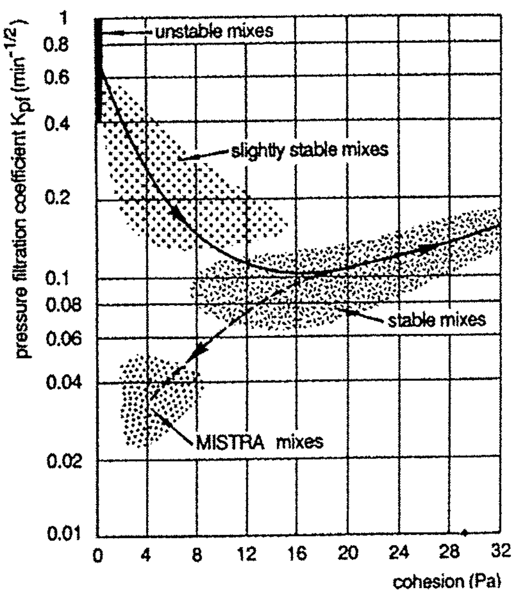


Figure 2.31 Resistance to pressure filtration vs. cohesion (De Paoli et al., 1992)

2.3.2 Set characteristics of cement-based suspension grouts

2.3.2.1 Gel-time and set-time

The gel and set time of grout are affected by numerous factors including: the use of fillers, particle size distribution of the grout, ambient grout temperature, the use of retarders and accelerators, the solids content of the formulation and the medium being injected. The use of some retarders and accelerators also affects the durability and strength of the grout.

2.3.2.2 Bleed

The residual permeability of a rock formation injected with grout can be estimated by using the amenability coefficient as previously described, but also the bleed of the grout has to be taken into account since bleed pockets will be formed.

In order for a grout to be considered 'stable' by the classic school of grouters, the bleed should be less than 5 %. Ideally, bleed should be less than 2 %. The use of superplasticizers masks the ultimate bleed in a suspension grout by slowing down the settlement of the cement particles. For this reason, bleed should also be determined from samples (kept covered to prevent evaporation) that have reached final set.

2.3.2.3 Unconfined compressive strength

The strength of grout is affected by numerous factors including : solids content, type and percentage of fillers, cement content, grain size distribution, water content, as well as the presence of certain retarders, accelerators, and viscosity modifiers.

The resistance against pressure filtration and the execution of the grouting operation have a major impact on the ultimate strength of the grouted medium. During an extensive laboratory testing program using the In-situ Soil Injection System (ISIS) in 2000 (Landry et al., 2000) using slag and non-slag based micro fine cements, the authors discovered that sands grouted with only one pass had considerably lower strength than the same sands grouted in several passes allowing pressure filtration to take place. The difference could amount to a factor six!

2.3.2.4 Durability and resistance to chemical attack

Although related to strength properties the durability of a grout can be considerably enhanced through the use of various admixtures that will react with excess calcium hydroxide to form secondary ettringite. Through the use of accelerated aging tests, whereby samples of cures grout are exposed to an environment that is harsher (at a higher temperature, exposed to dynamic attack by placing the samples in a shaker bath) than the expected field conditions. Furthermore, the ability of the grout to withstand chemical attack over time can be evaluated.

2.3.2.5 Matrix porosity and permeability

The matrix porosity and permeability of a set grout can be an important factor. A more pervious grout is subjected to faster deterioration that could, in turn, be accelerated if the environment has a low pH. Calcium hydroxide can be more readily leached out from the grout, if the permeability coefficient is high. Alternately, if a pervious grout is required in situations where strength is required but drainage is necessary, specially formulated cement foams can be used that have very high permeability and good strength characteristics.

2.3.3 Common additives and admixtures used in suspension grouts

2.3.3.1 Additives

2.3.3.1.1 Slag

Slag is a non-metallic by-product created during the smelting process of iron ore. In the smelter, layers of dolomite are placed between the ore. Slag is generally composed of silicates, aluminosilicates, calcium oxides and other base components. Slag is commonly used as an active component in the grout. It takes part in the hydraulic reaction between water and cement. It also reacts with other additives to create attringite. On its own, slag will cure very slowly and therefore must be used in conjunction with cement.

The low hydraulic reactivity of slag is ideal to delay initial set. This is particulary useful to control the set time of ultrafine cements. Most brands contain large amounts of slag and are blended with cement and other additives and admixtures before they are ground down to an ultra fine powder. Because of the large specific surface (Blaine fineness in excess of 8000 cm^2/g) and low matrix permeability and better chemical stability of ultrafine cement particles, the reactivity of the pure Portland cement is too high. Slag is commonly added to extend the set time of micro fine cement-based grouts and to create a reaction with the calcium hydroxide. This reduces the matrix porosity and enhances the chemical resitance.

2.3.3.1.2 Fly-ash

Fly-ash (Type F) is a rather inexpensive filler with pozzolanic characteristics. This product reacts with calcium hydroxide (free lime) that is generated by the hydraulic reaction between the cement and water to form secondary ettringite. This creates a more durable grout, especially in environments where the free lime could be easily leached out. Fly-ash also slightly reduces segregation and enhances the water repellant characteristics of the grout, as well as, its resitance to pressure filtration.

Fly-ash slows down the hydraulic reaction and strength development. Fly-ash also reacts with lime to form a low strength grout. Grouts containing high concentrations of type F fly ash (70-90 % of the total solids content in the mix) cure very slowly and can be used for backfill grouting operations, in compensation grouting applications and flowable fill.

Fly-ash is a valuable component used to make ultrafine cement by either wet milling for immediate use in micro fine cement-based grouts (Naudts and Yates, 2000). It is blended with Portland cement and plays the same role as slag but reaches a lower unconfined compressive strength (at the same concentrations and mixing ratios).

There are two common types of fly-ash available: type C and type F. The difference lies in their chemical composition due to the various types of coal used in the combustion chamber. If the concentration of type C fly ash exceeds 15 % by weight of cement, it could lead to rapid deterioration of the grout.

2.3.3.1.3 Natural Pozzolan

Natural pozzolan can be found in a natural state in different rock formations or they can be made from clay or schists burnt into cinder. Commonly known natural pozzolan such as pumice and trass have been effectively used to chemically react with the calcium hydroxide created during the hydration of cement to form secondary ettringite. By tying up the free calcium hydroxide, a more durable and competent end-product is achieved. The strength gain in the curing grout is slower and the exothermic reaction is less pronounced, which is beneficial when grout is placed in large quantities.

2.3.3.1.4 Trass

This natural pozzolan has been used over 2000 years. Trass-enhanced mortars have stood up for 20 centuries in marine environments. Trass is a powderised tuff stone mined in Andernavh (Am Rhein) in Germany. Trass is extensively used in grouts for grouting sewers because it enhances the chemical resistance of the end product. Because of its ability to react with the free lime, the bond to concrete of grouts containing natural pozzolan is considerably better compared to those that lack it.

2.3.3.1.5 Silica fume

Silica fume is a by-product of the production of silicon or alloys containing at least 75% silicon. The silica fume particles collected are spherical in shape with an average diameter of approximately 0.1 micron. The small particle size makes it act as small ball bearings, keeping the larger cement particles into position in the mix, enhancing the penetrability of the grout.

Silica fume is also useful to reduce the permeability of the cured grout and as a result enhance its durability. The stability and resistance against pressure filtration characteristics of a grout can be enhanced with the introduction of silica fume. Silica fume is also known to provide water-repellent characteristics to the grout.

Hooton (1990) researched and documented the qualities of silica fume enhanced grouts. During the last 15 years the authors have tested and used silica fume in numerous grouting applications. Especially for the grouting of structures, silica fume enhanced grouts are very suitable.

Typical concentrations of silica fume in cement-based grouts vary between 4 and 10 % by weight of cement. Because of its fineness, it is a most valuable additive to ultrafine cement-based grouts. It also enhances the wet milling of cement-based suspension grouts.

2.3.3.1.6 Bentonite

Bentonite is one of the most common additives used in cement-based suspension grouts. It enhances resistance to pressure filtration, reduces bleed, enhances stability and penetrability, and increases the cohesion and viscosity of the grout.

Bentonite is a claystone consisting of smectite (2-1 layer silicate) characterized by its great affinity for water. For grouting applications, sodium montmorillonite is recommended. They are composed of two tetrahedral sheets and one octahedral sheet. The weak Van der Waal's link between the 2:1 layers makes it possible for smectite particles to electrostatically attract other molecules of the opposite polarity. This phenomenon is particularly useful in grouting applications to curtail the segregation (minimize the bleed and reduce the pressure filtration coefficient) in cement-based suspension grouts.

Bentonite should be used as a pre-hydrated slurry. The authors found out that if the bentonite is not pre-hydrated first, it will lead to cracking of the cured grout. Even if bentonite is introduced as a slurry, it should always be entered first in the mix. The mixing order has a impact on the rheology and even the strength of the cured grout. When bentonite is introduced as the last component, its stabilizing effects are vastly reduced.

2.3.3.2 Admixtures

2.3.3.2.1 Dispersants, high range water reducers, and superplasticizers

These admixtures are used to reduce the viscosity of the grout and to reduce the speed of segragation of a given grout and consequently, enhance its stability.

Deflocculators enrobe the cement particles with a film of the same negative charge. Consequently, the particles reject each other and the formation of macroflocs is prevented. Typically deflocculators used in grouts are napthalene sulphonate or melamine based.

One important reason water reducers, dispersants, and superplasticizers are used is to improve the mixing of all ingredients. Consequently, these admixtures should be introduced at the beginning of the mixing cycle and not at the end. In the case of mixes characterized by low viscosity, it is appropriate to introduce dispersants at the end of the mixing cycle. The latter saves money as the late introduction of dispersants requires smaller amounts.

2.3.3.2.2 Viscosity modifiers

Viscosity modifiers can be used to provide a wide range of flow properties. Viscosity modifiers with high molecular weight which are highly cross linked, when used in a fully neutralized solution or colloidal suspension, will provide a very 'short flow' rheology. Short flow rheology can be characterized by gelled, highly water repellent consistency similar to mayonnaise. On the other hand, viscosity modifiers with a low molecular weight, more lightly cross-linked, will produce suspension grouts with 'long flow' rheology, more resembling a 'syrup'. One of the key attributes of viscosity modifiers is that they impart 'shear thinning'. This means that when pressure is applied to the grout, it will flow. When the driving pressure is removed, the grout will quickly 'thicken up'.

Minute quantities of bio-polymers such as natural gums and starches, cellulose ester derivatives and hydrophobically modified polyacrylate polymers, polyethelene oxide or polyvinyl alcohol, make it possible to change the rheology of a suspension grout dramatically. In order to maximize the qualities of these products, they should be properly pre-hydrated and chemically neutralized.

Viscosity modifiers can increase the resistance against pressure filtration by a factor 2 to 10, depending on the grout formulation.

2.3.3.2.3 Retarders, grout stabilizers

In the past, lignosulphonate-based superplasticizers were used to slow down the curing of cement-based suspension grouts. This was done to facilitate multiple grouting passes for soil grouting or to prevent the loss of access to a given grout zone. Citric acid has been used for the same purpose. A major loss in strength was noticed, in spite of citric acid's minimal impact on initial and final gelation times.

CHAPTER 3

EXPERIMENTAL WORKS

3.1 Introduction

In this study, fracture grouting of saturated, loose granular soils of different fine content were investigated. Both micro fine cement grouts and ordinary Portland cement grouts were tried. Basically, a relationship was obtained between soil conditions (grain size distribution, relative density, overburden stress) and grouting parameters (type of grout, grouting pressure, injected volume of grout, rheological properties of the grout or simply water/solids ratio).

Model tests were carried out to simulate both fracture grouting and compaction grouting techniques by micro fine cement and ordinary Portland cement grouts, respectively. During the tests grouting pressures and the injected volumes were recorded and at the end of the tests grouted soil specimens were exposed carefully and final grout shapes were observed and correlated with the grouting parameters. Amount of heave occurred at the top of the specimen during injections was recorded at each test.

3.2 Tests performed with micro fine cement grouts

3.2.1 Properties of sand specimen used in the tests

Yumurtalık sand taken from 8 km west of Adana province in Turkey has been used in the experiments. Fine content of Yumurtalık sand is 6 % which is within the range of liquefiable soils. In order to investigate the effect of fine content on grouting parameters (fracture pressure, injected volume, etc.) a non-plastic silty material (ML) with a 73 % fine content has been mixed with Yumurtalık sand in different proportions. In this way, mixtures of Yumurtalık sand and silt material of different fine contents have been obtained. Fine contents of these mixtures are 6%, 24% and 32%. Particle size distributions of these three sandy soils are shown in Figure 3.1.

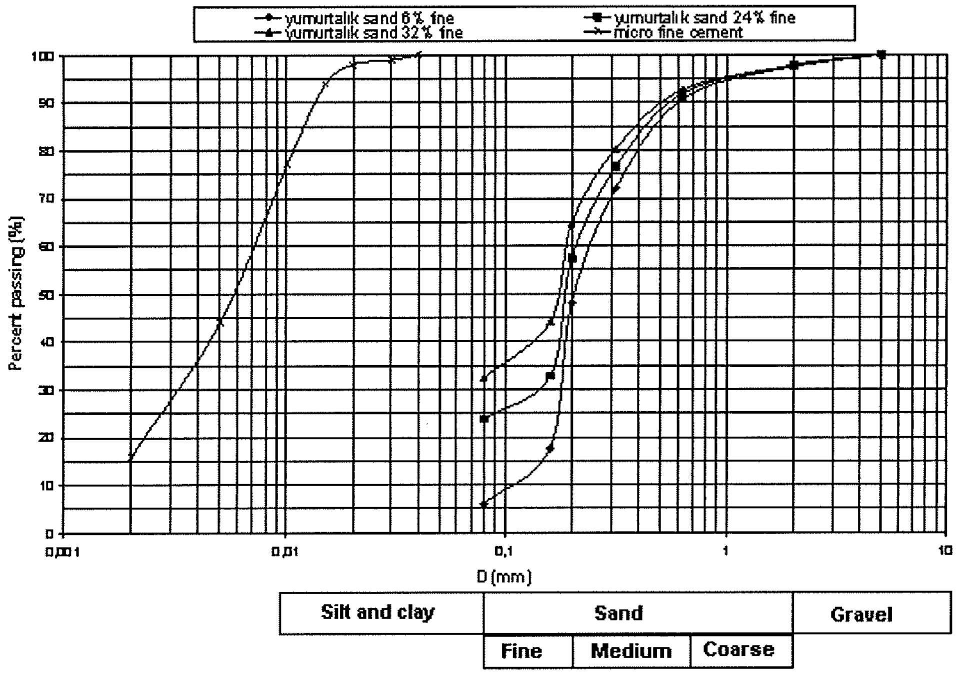


Figure 3.1 Particle size distributions of sandy soils used in the experiments

Density tests (BS 1377, 1990) have been performed on the sand specimen to determine its maximum and minimum dry densities. Specific gravity tests were carried out according to the Turkish Standards (TS1900, 1987) and the results are listed in Table 3.1.

Table 3.1 Maximum and minimum density test results

Soil	Maximum dry density, ρ_{dmax} (Mg/m^3)	Minimum dry density,ρ_{dmin} (Mg/m^3)	Specific Gravity (G_S)
Yumurtalık sand	1.732	1.489	2.70

3.2.2 Test set up

The test set-up consists of a pressure source, volume change measurement device, grout container, water-grout interface chamber, grouting pipes and valves, injection pipe and a soil test box as can be seen in Figure 3.2.

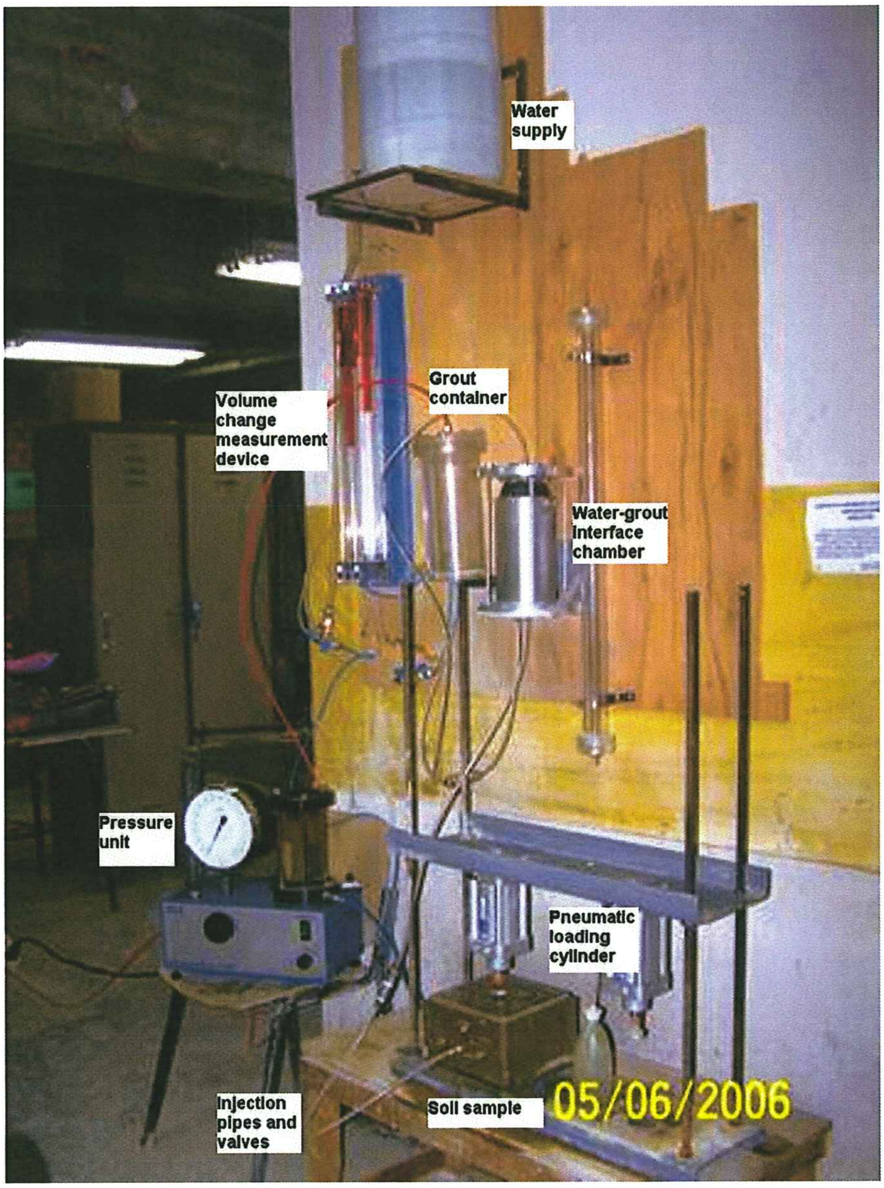


Figure 3.2 Laboratory set up

A pressure source of a conventional triaxial test apparatus supplying pressures up to 1700 kPa (~17 bars) has been utilized. This pressure source has gasoil-water interface. The pressurized water exiting from this source enters into the volume change measurement device with a capacity of 250 cc (ml) and from here it goes into the water-grout interface chamber of 22 cm height and 12.5 cm diameter. In this chamber, there exists an inflatable balloon which expands when the pressurized water enters inside and transmits the pressure to the underlying grout material which was previously filled in this chamber from the grout container by means of an air compressor. The grout material is then forced into the soil specimen through the injection pipe (5 cm in length and 1 cm in diameter) placed at the mid height of the test box of 20 cm in width and 20 cm in length. The height of the test box is 12 cm. However, the height of the soil specimen is kept as 10 cm. The volume of grout material injected into the soil at each pressure increment is equal to the volume of pressurized water entering into the water-grout interface chamber. And the volume of this water is recorded from the volume change measurement device. A schematic view of the experimental set-up can be seen in Figure 3.3.

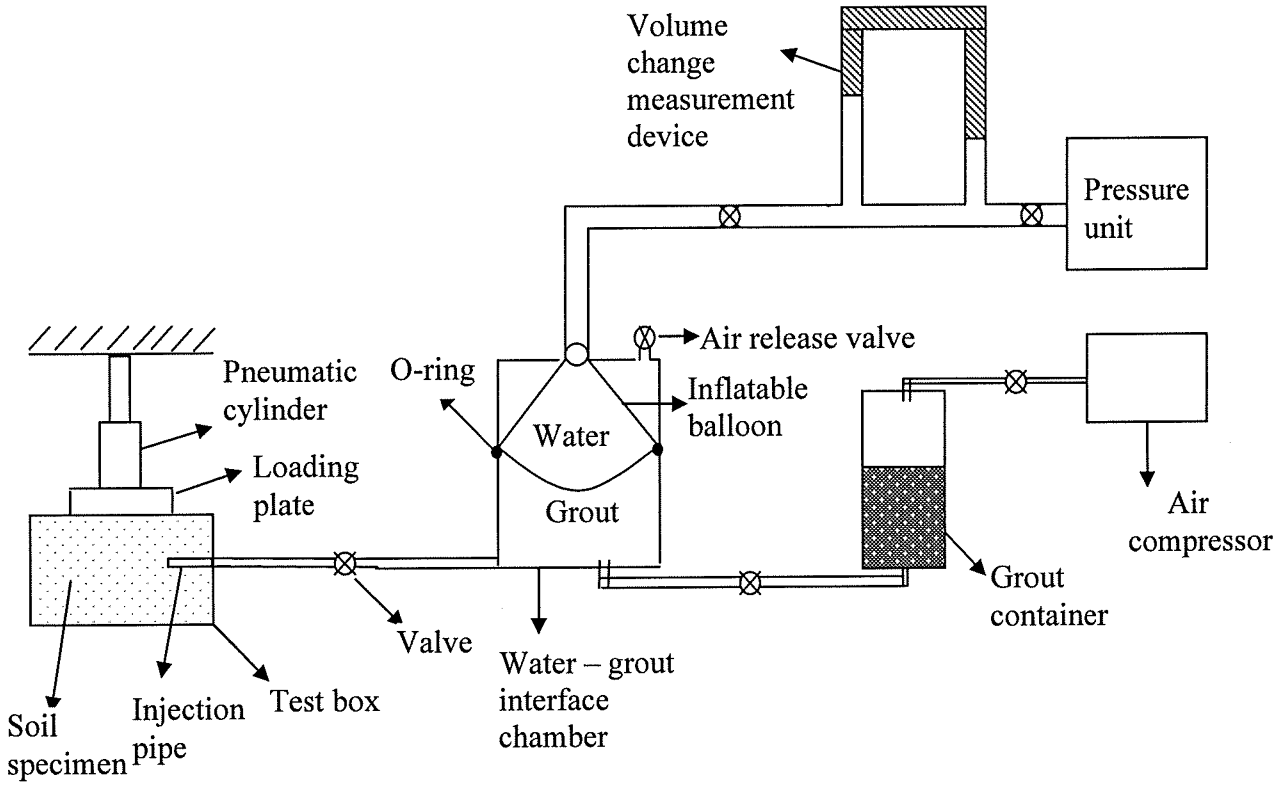


Figure 3.3 Schematic view of laboratory set up

3.2.3 Grout material

Both ordinary Portland cement (Type I with 95 % of clinker) and micro fine cement suspension grouts consisting of prehydrated bentonite, water and superplasticizer have been utilized. Significant difference in grain size distributions of the cements together with the test soils can be seen in Figure 3.3. Specific gravity and Blaine fineness values of the two cements are also significantly different. Specific gravity of ordinary Portland cement is 2.94 while that of micro fine cement is 2.24. Blaine fineness values are 3844 cm^2/g and 30446 cm^2/g for ordinary Portland cement and micro fine cement respectively. The so-called differences also showed their effects on the response of soil specimen to grouting.

In order to observe rheological differences between ordinary Portland cement and micro fine cement grouts, viscosity, initial viscosity and cohesion values were determined both for ordinary Portland and micro fine cement grouts. Using a cylindrical spindle type viscometer (Brookfield viscometer) and following ASTM D4016 viscosity tests were performed on grouts with water/solids ratio of 1.5. It should be noted that 5 % bentonite and 1 % superplasticizer were added into the grout mixture as a replacement for cement. The initial viscosity and cohesion values for ordinary Portland cement grout were 608 cp and 65 Pa, respectively. For micro fine cement grouts, these values were 15 cp and 25 Pa, respectively.

The Marsh cone having an opening of 4.76 mm was also used to determine the flow time of cement grouts. A 1200 ml sample of grout was introduced into the cone and the time of flow of one liter of grout was measured. The flow time for one liter of water through the cone at 20 ^{o}C is 32 s, and that of one liter of ordinary Portland cement grout is 42 s. For micro fine cement grout flow time is found as 33 being very close to the flow time of water.

The initial and final setting times of the grouts were determined as recommended by ASTM C191-92. The penetration of 1.0 mm diameter needle was monitored at different time intervals. The initial and final times of set correspond to a needle penetration of 25 mm and less than 1.0 mm, respectively. The initial setting time corresponds to a stage at which the fluidity of the grout decreases to a level that it can no longer be pumped, and the grout loses its plasticity. The final setting corresponds to a stage at which the grout hardens and increases in strength (Littlejohn, 1982). The initial and final setting times at 20 ^{o}C both for ordinary Portland and micro fine cement grouts can be found in Table 3.2. Water/solids ratios of the grouts were 1.5 and 5 % bentonite and 1 % superplasticizer were used as a replacement for cement.

Table 3.2 Initial and final times of set for ordinary Portland and micro fine cement grouts

Grout type (water/solids =1.5)	Ordinary Portland cement	Micro fine cement
Initial Setting Time (hours)	11	2
Final Setting Time (hours)	16	4.5

These rheological differences between ordinary Portland cement and micro fine cement grouts show that micro fine cement grouts are favorable with respect to their finer particle size (Figure 3.4), significantly smaller specific gravity and higher blaine fineness, higher groutability and very small amounts of bleed, water filtration and viscosity values.

The micro fine grout mixtures have been prepared to have water solid ratios of 1.5, 2.0, 2.5 and 3.0 so as to represent different characteristics of rheological properties. Bentonite was prehydrated before mixing with the other constituents of the grout. Amount of bentonite and superplasticizer added to the mixture was 5 % and 1 % of weight of cement, respectively.

For bleeding tests ordinary Portland cement grouts were placed in a 1000 ml graduated cups and the percentage of water on top of the mix was measured and related to the total volume of the sample at various time intervals. For micro fine cement grouts addition of 5 % bentonite was sufficient due to its cement grains of much smaller grain size and specific gravity. Amount of bleed for these grout mixtures was about 1 %.

All of the grout mixtures have been mixed with a high shear mixer for a duration of 10 minutes before injecting into the specimen.

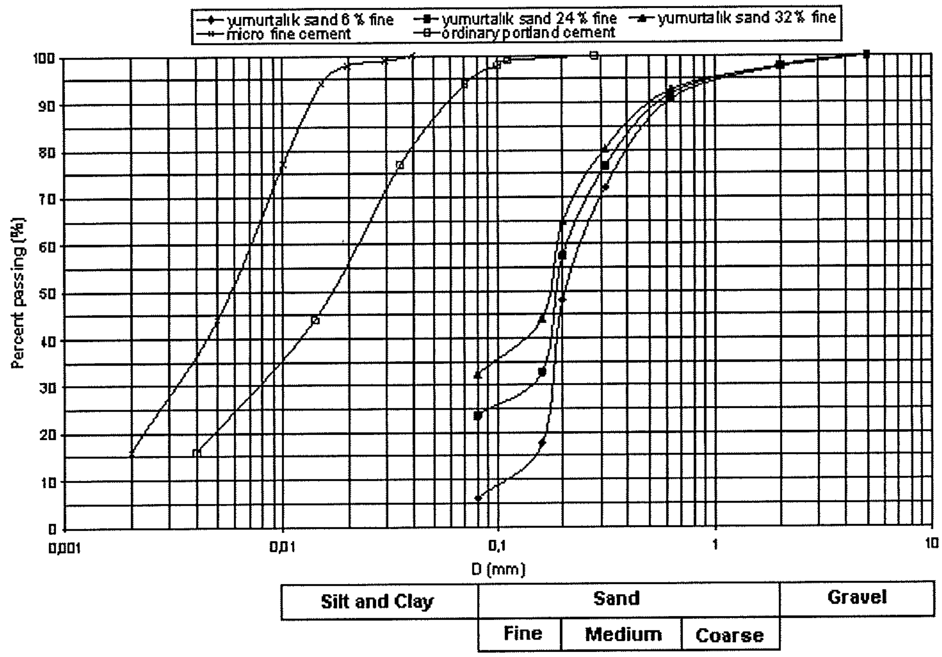


Figure 3.4 Grain size distributions of ordinary Portland cement and micro fine cement compared to test soils

3.2.4 Performance of tests

Soil specimens have been placed in the test box so as to have the required relative density. At the beginning, relative density of the soil specimens were 40 % and later on it was increased to 60 %. Corresponding dry densities for a certain relative density have been calculated and the required mass has been placed in the test box in dry condition. Then they have been saturated and a surcharge pressure of 50 kPa has been applied on the soil specimen and it was left to a relatively rapid consolidation.

After the grout material was prepared, it was poured into the grout container. The inflatable balloon (1500 cc of volume in its original size) was filled with de-aired water from the water tank and then the grout mixture was transmitted into the water-grout interface chamber by applying a small pressure (50 kPa) via the air compressor. As the grout is filled into the interface chamber the water in the balloon exits and goes into the volume change measurement device. Later on the system is reversed and the grouting pressure was increased gradually (pressurized water enters into the balloon and pushes the grout underneath) and the injection lines were filled with grout. From this point on, the corresponding volume of grout injected into the specimen with respect to the applied pressure was recorded via the volume change measurement device.

3.3 Tests performed with ordinary Portland cement

Tests performed with ordinary Portland cement grouts have shown that it is diffucult to fracture the fine silty sandy soils with an ordinary Portland cement. Because at the end of these tests the grouted soil has been exposed and it has been observed that a very small amount of grout has been accumulated at the tip of the injection pipe without fracturing the soil. This behavior somehow resembled to the compaction grouting mechanism where the grout material expands around the injection pipe without fracturing.

In all of the tests performed with ordinary Portland cement grouts (water/solids ratios varying from 1.0 to 2.0), same behavior has been observed. One reason for this behavior was at first thought to be the insufficiency of the pressure source. However, volume change measurement device recorded some amount of volume injected into the specimen during pressure increments. But when the test was stopped and the specimen was exposed, no significant amount of grout (in an amount equal to that recorded at the volume change device) was observed at the tip of the injection pipe. In addition, when the grout material in the injection pipe was investigated it was observed that the grout material had been exposed to pressure filtration leaving the solid cement particles inside injection pipe avoiding further injection. In other words, the water had separated from the grout. Then the measurements in the volume change device reflected the amount of water filtrated from the grout. Therefore, the problem is not a pressure problem but instead a water filtration problem.

Water filtration causes the water to be squeezed out of the grout and into the soil pores. This causes an increase in grout viscosity and prevents the grout penetrating into the ground. The grout forces a cavity within the soil to expand, compacting and displacing the adjacent soil. However, this is not a fracturing mechanism, but on the contrary resembles to compaction grouting which is sometimes refered as intrusion grouting. Therefore, grouting pressure is not continued to increase at this stage and the test is stopped.

Based on the results of these preliminary tests performed with ordinary Portland cement grout mixtures and in order to eliminate water filtration problem, it has been determined to add kaolinite into the test soil at a certain percentage as a replacement for Yumurtalık sand. It has also been considered necessary to design a new test set up that could provide higher grouting pressures of at most 50 bars in view of pressure filtration mechanism.

3.3.1 Properties of Yumurtalık sand – kaolinite mixtures

Mixtures of Yumurtalık sand and kaolinite have been prepared by adding 8 %, 15 % and 25 % of kaolinite by weight as a replacement for the sand. Fine contents of these mixtures are 14%, 21% and 31%, respectively. Particle size distributions of the mixtures can be seen in Figure 3.5.

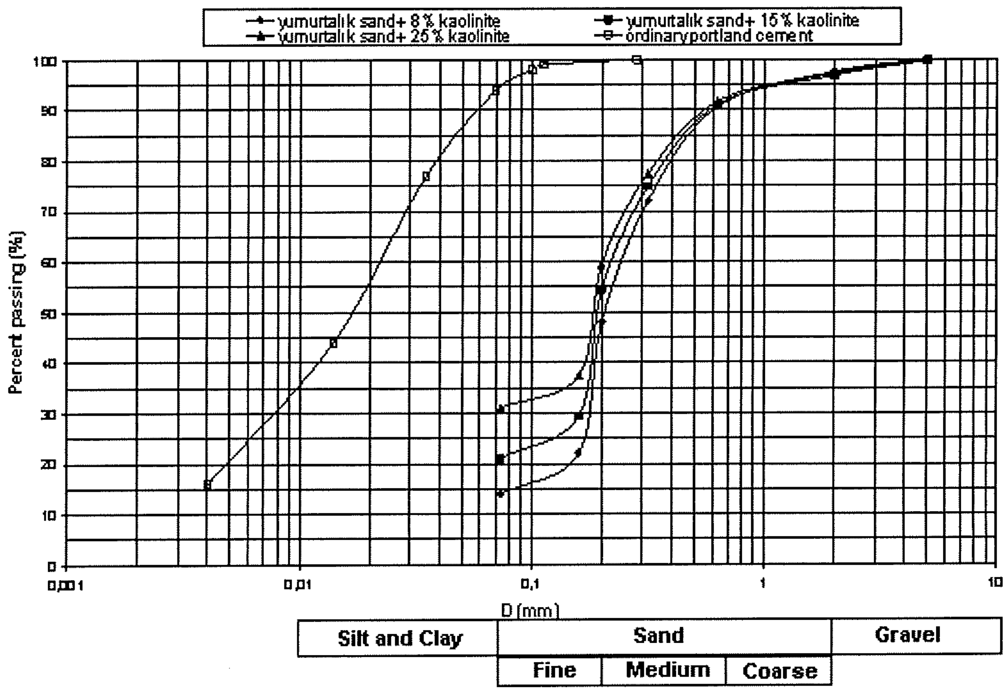


Figure 3.5 Particle size distributions of Yumurtalık sand and kaolinite mixtures used in the experiments

Density tests (BS 1377, 1990) have been performed on the mixtures to determine their maximum and minimum dry densities. Specific gravity tests were carried out according to the Turkish Standards (TS1900, 1987) and the results are listed in Table 3.3.

Table 3.3 Maximum and minimum density test results

Soil	Maximum dry density, ρ_{dmax} (Mg/m^3)	Minimum dry density, ρ_{dmin} (Mg/m^3)	Specific Gravity (G_S)
Yumurtalık sand + 8% kaolinite	1.75	1.47	2.69
Yumurtalık sand + 15% kaolinite	1.84	1.45	2.68
Yumurtalık sand+25% kaolinite	1.88	1.46	2.68

Atterberg limit tests show that mixture of 8 % kaolinite and Yumurtalık sand is non-plastic wheras mixtures prepared by 15 % and 25 % kaolinite have plasticity index values of 2 and 6, respectively.

3.3.2 Test set up

The test set-up consists of a piston type hydraulic jack as a pressure source, grout container, a dial gauge measuring the transitional progress of the piston for grout volume measurements, grouting pipe, injection hoses and valves and a cylindrical tank for the test soil. The test set up can be seen in Figure 3.6.

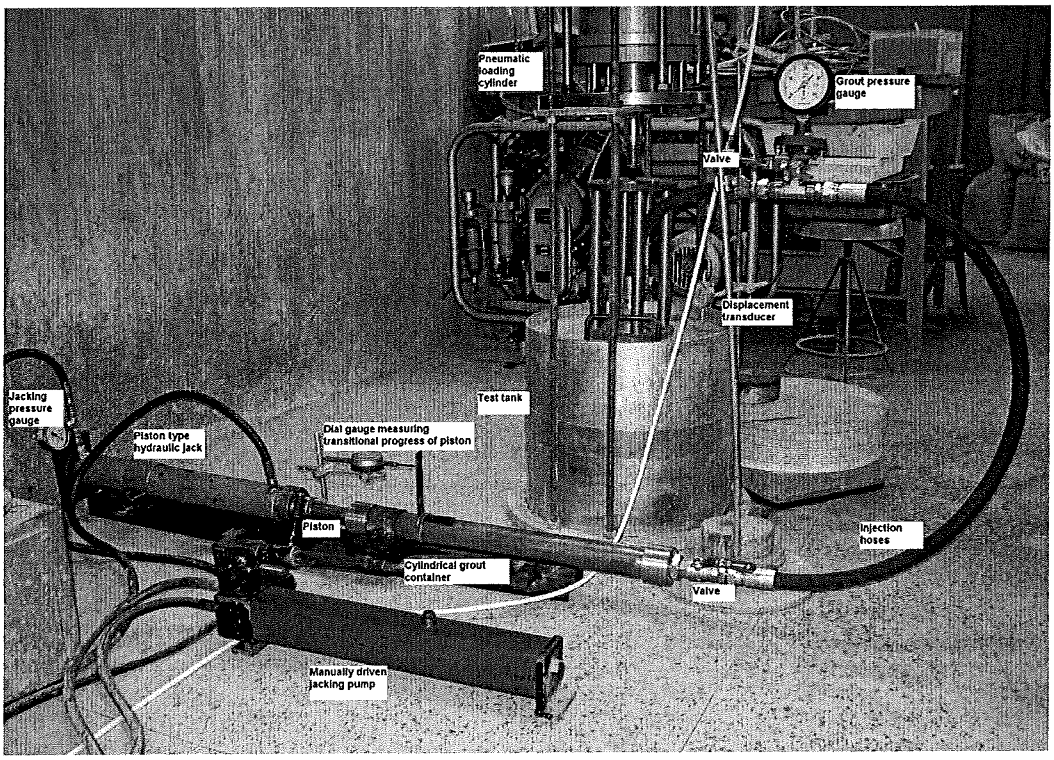


Figure 3.6 Test set up for ordinary Portland cement grouting tests

A piston type hydraulic jack with a capacity of 2 tons has been utilized as a pressure source. The piston of the hydraulic jack enters into a cylindrical grout container of 48 mm internal diameter and 55 mm length (approximately a volume of 1 liter). The grout container is made of brass and thus no corrosion problems occur. The piston pushes the grout which is previously filled into the container and transmits it to the open end grouting pipe of 20 mm internal diameter and 175 mm lenght through injection hoses of 20 mm internal diameter. Grouting pipe is embedded vertically into the soil specimen placed in a galvanized cylindrical test tank of 400 mm diameter and 350 mm height. The grouting pipe is placed at the mid height of the test tank during filling sand specimen into the tank. Grouting pressure is measured via a special diaphragm type grout pressure gauge at a location just before the entrance of grouting pipe into the soil specimen. The jacking pressure is also measured as a comparison. The volume of grout injected into the soil specimen has been measured through a transitional dial gauge fixed on the piston of the hydraulic jack. A schematic view of the experimental set up can be seen in Figure 3.7.

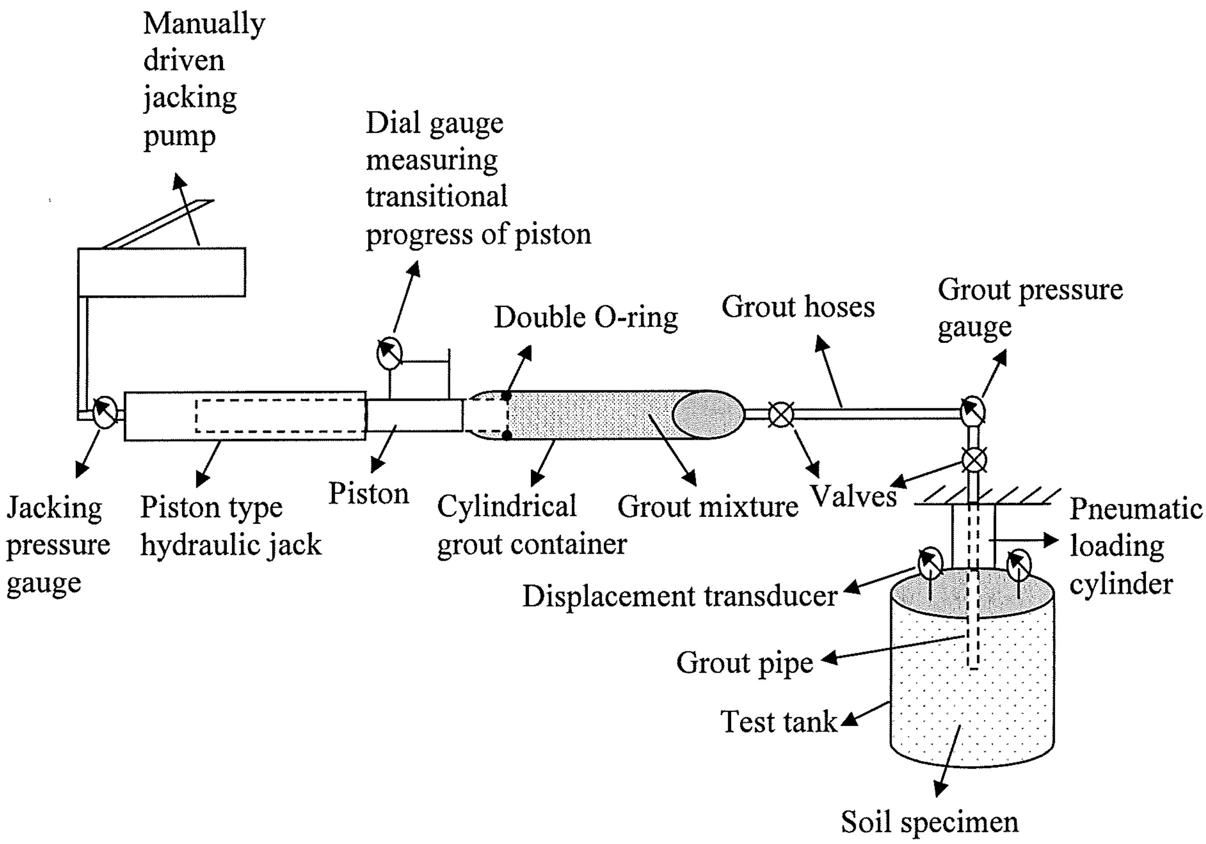


Figure 3.7 Schematic view of the experimental set up

3.3.3 Grout material

Ordinary Portland cement (Type I with 95 % of clinker) suspension grouts consisting of prehydrated bentonite, water and superplasticizer have been utilized. Grain size distributions of the cement together with the test soils can be seen in Figure 3.1. Specific gravity and Blaine fineness values of the cement are 2.94 and 3844 cm^2/g, respectively.

Using a cylindrical spindle type viscometer (Brookfield viscometer) and following ASTM D4016 (2002) viscosity tests were performed on ordinary Portland cement grouts with water/solids ratios of 1.0, 1.5 and 2.0. It should be noted that 5 % bentonite and 1 % superplasticizer were added into the grout mixture as a replacement for cement. Initial viscosity and cohesion values for these grout mixtures can be seen in Table 3.4.

Table 3.4 Rheological properties of grout mixtures

Water/solids Ratio	Initial Viscosity (cp)	Cohesion (Pa)
1.0	950	90
1.5	608	65
2.0	110	35

The Marsh cone flow times of the grout mixtures have also been measured having an opening of 4.76 mm was also used to determine the flow time of cement grouts. A 1200 ml sample of grout was introduced into the cone and the time of flow of one liter of grout through 4.76 mm diameter opening was recorded. The flow times for one liter of ordinary Portland cement grout with water/solids ratios of 1.0, 1.5 and 2.0 are 70 s, 42 s and 35 s, respectively.

The initial and final setting times of the grouts were determined as recommended by ASTM C191-92. The penetration of 1.0 mm diameter needle was monitored at different time intervals. The initial and final setting times for ordinary Portland cement grout mixtures at 20 °C can be seen in Table 3.5.

Table 3.5 Initial and final times of set for ordinary Portland cement grouts

Water/solids Ratio	Initial setting time (h)	Final setting time (h)
1.0	8	12
1.5	11	16
2.0	17	23

The grout mixtures have been prepared to have water solid ratios of 0.7, 1.0, 1.5 and 2.0 to represent different rheological properties. Bentonite was prehydrated before mixing with the other constituents of the grout. Amount of bentonite and superplasticizer added to the mixture was 5 % and 2 % of weight of cement, respectively.

For bleeding tests, ordinary Portland cement grouts were placed in a 1000 ml graduated cups and the percentage of water on top of the mix was measured and related to the total volume of the sample at various time intervals. Amount of bleed for these grout mixtures was about 2 %, 6 % and 11 %, respectively.

All of the grout mixtures have been mixed with a high shear mixer for a duration of 10 minutes before injecting into the specimen.

3.3.4 Performance of tests

Soil specimens have been placed in the test box so as to have a certain relative density. Corresponding dry densities for the selected relative densities have been calculated and the required mass has been placed in the test box in a dry condition. Then they have been saturated and a surcharge pressure of 50 kPa has been applied on the soil specimen and it was left to a relatively rapid consolidation. The grouting pipe was placed in the soil specimen during filling the sand into the test tank (Figure 3.8).

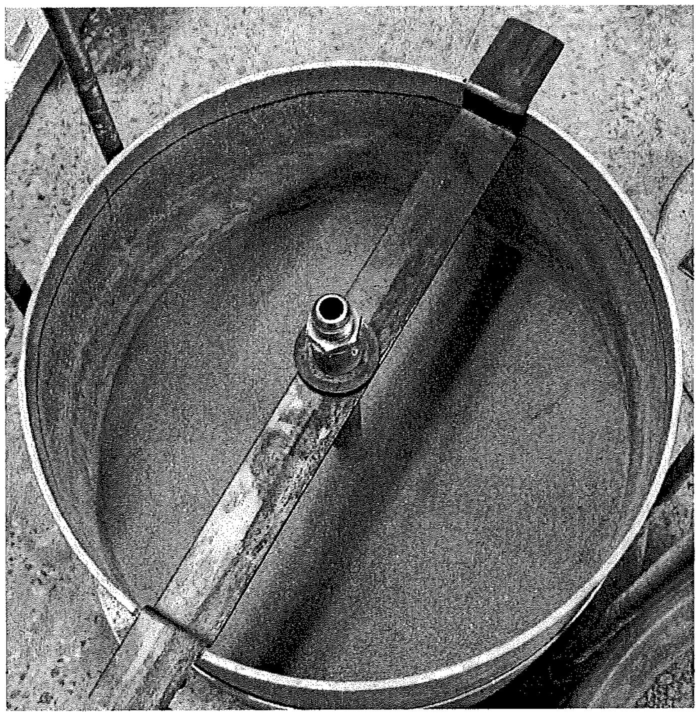


Figure 3.8 Placement of grouting pipe during filling sand into the test tank

The effect of relative density on the response of soil to grouting has been examined by repeating some of the tests at a higher relative density. Figure 3.9 shows the higher and lower placement densities of the Yumurtalık sand and kaolinite mixtures.

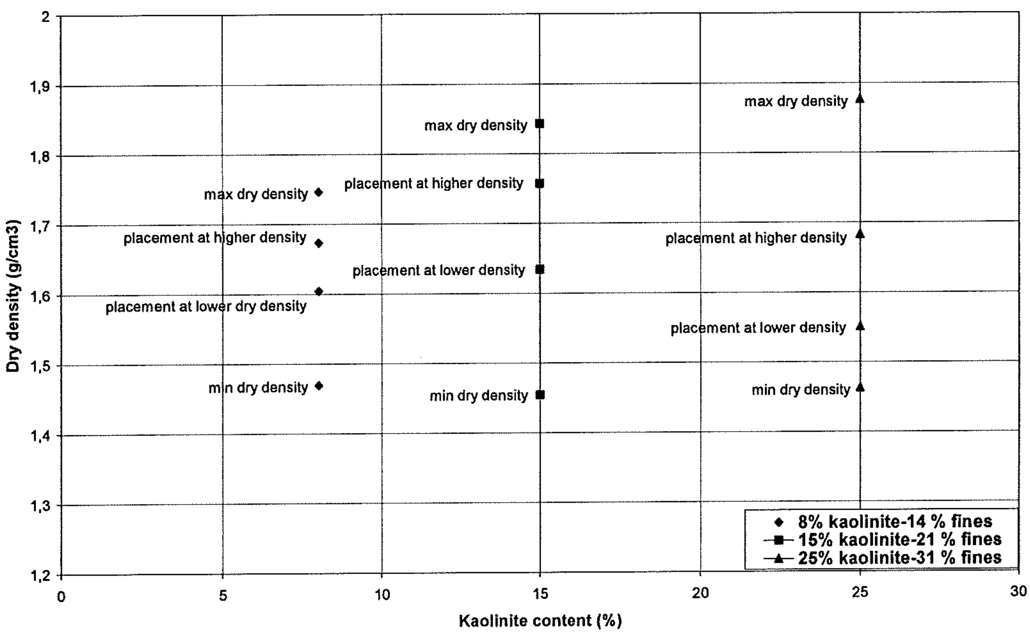


Figure 3.9 Higher and lower placement densities of the soil specimens

Once the consolidation of the specimens was completed and the grout mixture was prepared, it was poured into the cylindrical grout container with the injection hoses being filled with the grout as well. Thus the air in the hoses was removed before grouting into the specimens.

The grout material was injected into the soil at a constant rate (by driving the piston equal distances at equal time intervals) and the corresponding pressure build up was recorded at the grout pressure gauge. The displacement at the top of the specimen was also recorded by means of two displacement transducers.

At the end of the grouting operations and after waiting for the grout material to solidify, the specimens were mined through and the injected grout masses were exposed. The volume and the mass of the grout masses were measured for each test and the corresponding dry densities have been calculated. The measured volumes were compared with the injected volumes and thus the filtration percentages have been calculated. The obtained shapes of the grouting masses were correlated with the soil and grouting parameters.

CHAPTER 4

PRESENTATION OF TEST RESULTS AND DISCUSSIONS

4.1 Results of tests performed by micro fine cement grouts

Ordinary Portland cement and micro fine cement grouts of different water/solid ratios were tried in the tests. Effects of fine content and relative density of the soil and the effect of overburden stress on the fracturing pressure and pressure – volume intake relationship were investigated.

The tests performed with grouts consisting of ordinary Portland cement have shown that it is diffucult to fracture the fine silty sandy soils with an ordinary Portland cement. Because at the end of these tests the grouted soil has been exposed and it has been observed that a very small amount of grout has been accumulated at the tip of the injection pipe without fracturing the soil. This behavior somehow resembled to the compaction grouting mechanism where the grout material expands around the injection pipe without fracturing.

In all of the tests performed with ordinary Portland cement grouts, same behavior has been observed. One reason for this behavior was at first thought to be the insufficiency of the pressure source. However, volume change measurement device recorded some amount of volume injected into the specimen during pressure increments. But when the test was stopped and the specimen was exposed, no significant amount of cementitious grout (in an amount equal to that recorded at the volume change device) was observed at the tip of the injection pipe. In addition, when the grout material in the injection pipe was investigated it was observed that the grout material had been exposed to pressure filtration leaving the solid cement particles inside injection pipe avoiding further injection. In other words, the water had separated from the grout. Then those measurements in the volume change device reflected the amount of water filtrated from the grout. Therefore, the problem is not a pressure problem but instead a water filtration problem.

Water filtration causes the water to be squeezed out of the grout and into the soil pores. This causes an increase in grout viscosity and prevents the grout penetrating into the ground. The grout forces a cavity within the soil to expand, compacting and displacing the adjacent soil. However, this is not a fracturing mechanism, but on the contrary resembles to compaction grouting which is sometimes refered as intrusion grouting. Therefore, grouting pressure is not continued to increase at this stage and the test is stopped.

In all of the tests with ordinary Portland cement (water/solids ratios varying from 1.5 to 3.0) same behavior is observed.

Based on the results of these tests performed with ordinary Portland cement type of suspensions and to eliminate water filtration problem, it has been determined to utilize micro fine cement grouts which are favorable with respect to their finer particle size, significantly smaller specific gravity and higher blaine fineness, higher groutability and very small amounts of bleed and water filtration (See Figure 3.4 for comparison of particle size distributions of ordinary Portland cement, micro fine cement and the soil specimens) characteristics. This type of cement is not produced in Turkey but exported from abroad.

Micro fine cement grouts with varying water / solids ratios of 1.5, 2.0, 2.5 and 3.0 were used in the tests. Two tests were performed for each water/solids ratio in order to verify the test results. There was no water filtration and segregation problem. In all of the test, the main behavior is that the grout is forced against the soil when the pressure is increased and some amount of grout is accumulated at the tip of the injection pipe. Then at the tensile limit of the soil a brittle fracturing is observed. When the injected specimens are exposed, fracture patterns are observed in some of them (especially in those injected into finer soils with grouts of higher water/solids ratios), but in some others a soil – grout mixture has been observed at the tip of the injection pipe reflecting the compaction grouting mechanism together with a small amount of permeation at the initial low pressure applications.

The results of these tests performed with micro fine cement grouts are presented in Table 4.1. In these tests relative densities of the soil specimens are kept as 40 %.

In order to investigate the effect of relative density on the response of the soil specimens to grouting, some of the tests have been repeated by increasing the relative density of the soil specimens to 60 %. Results of these tests are presented in Table 4.2 and Table 4.3.

Table 4.1 Results of grouting tests on three soil specimens with different fine content, RD=40%

6% fine	Water/solid	1.5	1.5	2.0	2.0	2.5	2.5	3.0	3.0
	Fracturing Pressure (kPa)	1180	670	450	390	330	310	300	220
	Volume injected until fracturing (cc)	100.5	74.5	98	81	77	69	76	60
	Volume injected during fracturing (cc)	57.5	52.5	42	51	47	47	49	38
	Vertical heave during fracturing (0.01 mm)	53-44	40-47	31-45	27-43	33-23	31-19	24-18	17-15
24% fine	Water/solid	-	-	2.0	2.0	2.5	2.5	3.0	3.0
	Fracturing Pressure (kPa)	-	-	550	550	400	350	350	300
	Volume injected until fracturing (cc)	-	-	63	66	58	45	48	40
	Volume injected during fracturing (cc)	-	-	23	34	34	33	34	32
	Vertical heave during fracturing (0.01 mm)	-	-	26-31	39-45	17-24	23-30	14-21	18-25
32% fine	Water/solid	-	-	2.0	2.0	2.5	2.5	3.0	3.0
	Fracturing Pressure (kPa)	-	-	620	580	420	350	300	260
	Volume injected until fracturing (cc)	-	-	60.5	62	48	40	43	30
	Volume injected during fracturing (cc)	-	-	45.5	49	48	48	40	50
	Vertical heave during fracturing (0.01 mm)	-	-	35-27	40-29	26-17	25-33	23-12	25-17

Table 4.2 Grouting test results performed on soil specimens with 6% fine content and RD=60%

6% fine	Water/solid	2.0	2.0	2.5	2.5	3.0	3.0
	Fracturing Pressure (kPa)	>1640	1490	1020	930	630	510
	Volume injected until fracturing (cc)	50	39.5	45	36.5	41	25.5
	Volume injected during fracturing (cc)	-	50	47	53.5	54	49.5
	Vertical heave until fracturing (0.01 mm)	32	26	19	13	20	15
	Vertical heave during fracturing (0.01 mm)	-	35	39	44	23	30

Table 4.3 Grouting test results performed on soil specimens with 32% fine content and RD=60%

32% fine	Water/solid	2.5	2.5	3.0	3.0
	Fracturing Pressure (kPa)	1010	890	710	530
	Volume injected until fracturing (cc)	27.1	29.7	25.7	16.5
	Volume injected during fracturing (cc)	52.9	52.7	54.3	49.5
	Vertical heave until fracturing (0.01 mm)	27	21	17	13
	Vertical heave during fracturing (0.01 mm)	44	51	36	31

Effect of overburden stress applied on the specimen was investigated by increasing the vertical stress to 100 kPa. Some of the tests were repeated taking the relative density of soil specimens as 40 %. Results of these tests are presented in Table 4.4.

Table 4.4 Grouting test results performed on soil specimens with 6 % and 32 % fine contents taking RD=40% and overburden stress as 100 kPa

Water/solid	3.0	3.0	2.5	3.0	2.5
Fine content %	6	6	6	32	32
Fracturing Pressure (kPa)	490	510	1220	790	>1640
Volume injected until fracturing (cc)	42.1	35.2	49.5	23.6	31.3
Volume injected during fracturing (cc)	49.9	48.8	51.3	48.4	-
Vertical heave during fracturing (0.01 mm)	21-13	17-9	27-19	19-26	-

Figure 4.1 shows the relationship between the fracturing pressure and the water/solids ratio of the micro fine cement grout. Effect of relative density of the soil specimen can also be seen in this figure.

Volumes of grout material injected into soil specimens of different fine content with a relative density of 40% can be seen at each pressure increment in Figures 4.2 to 4.11.

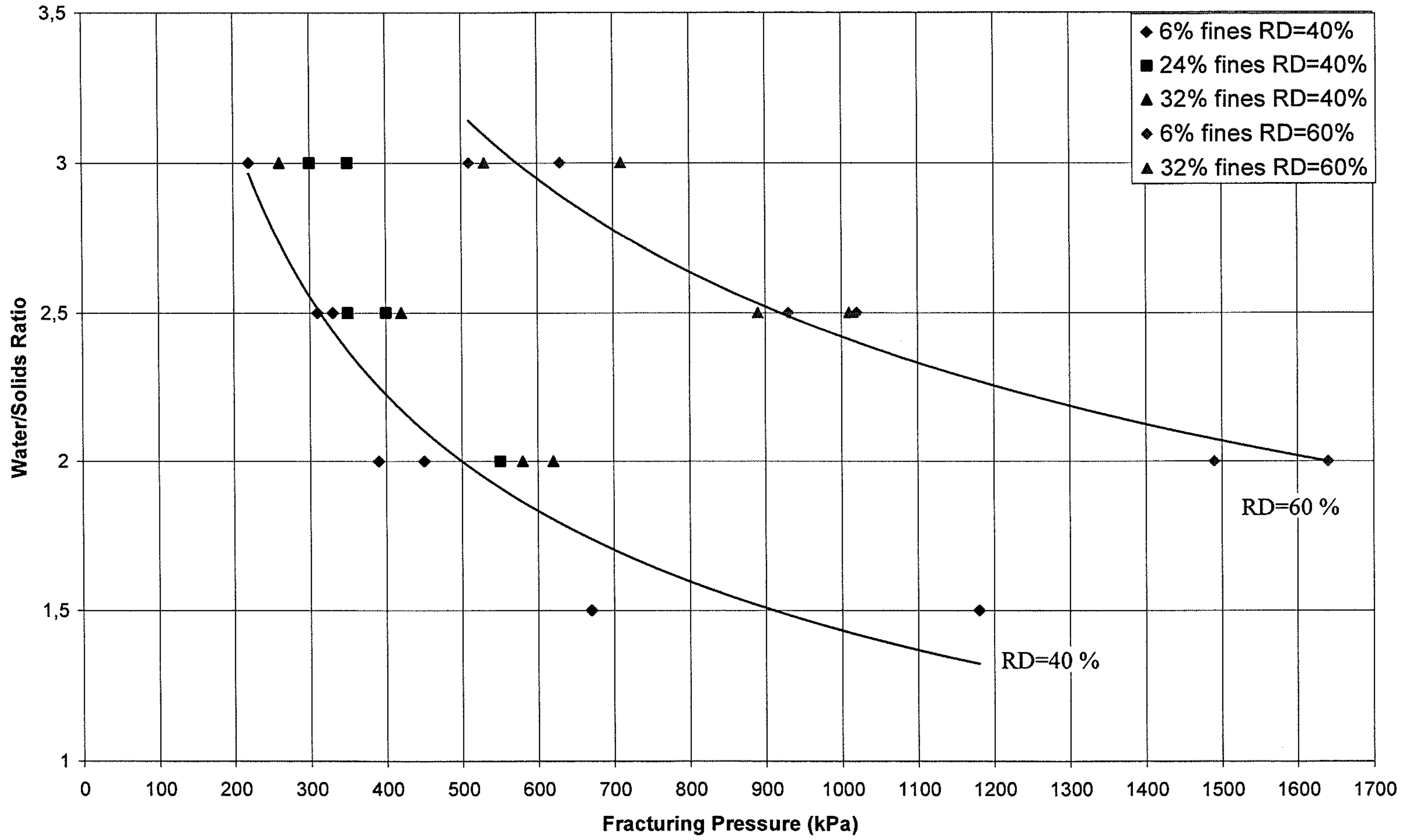


Figure 4.1 Effects of water/solids ratio and relative density on fracturing pressure

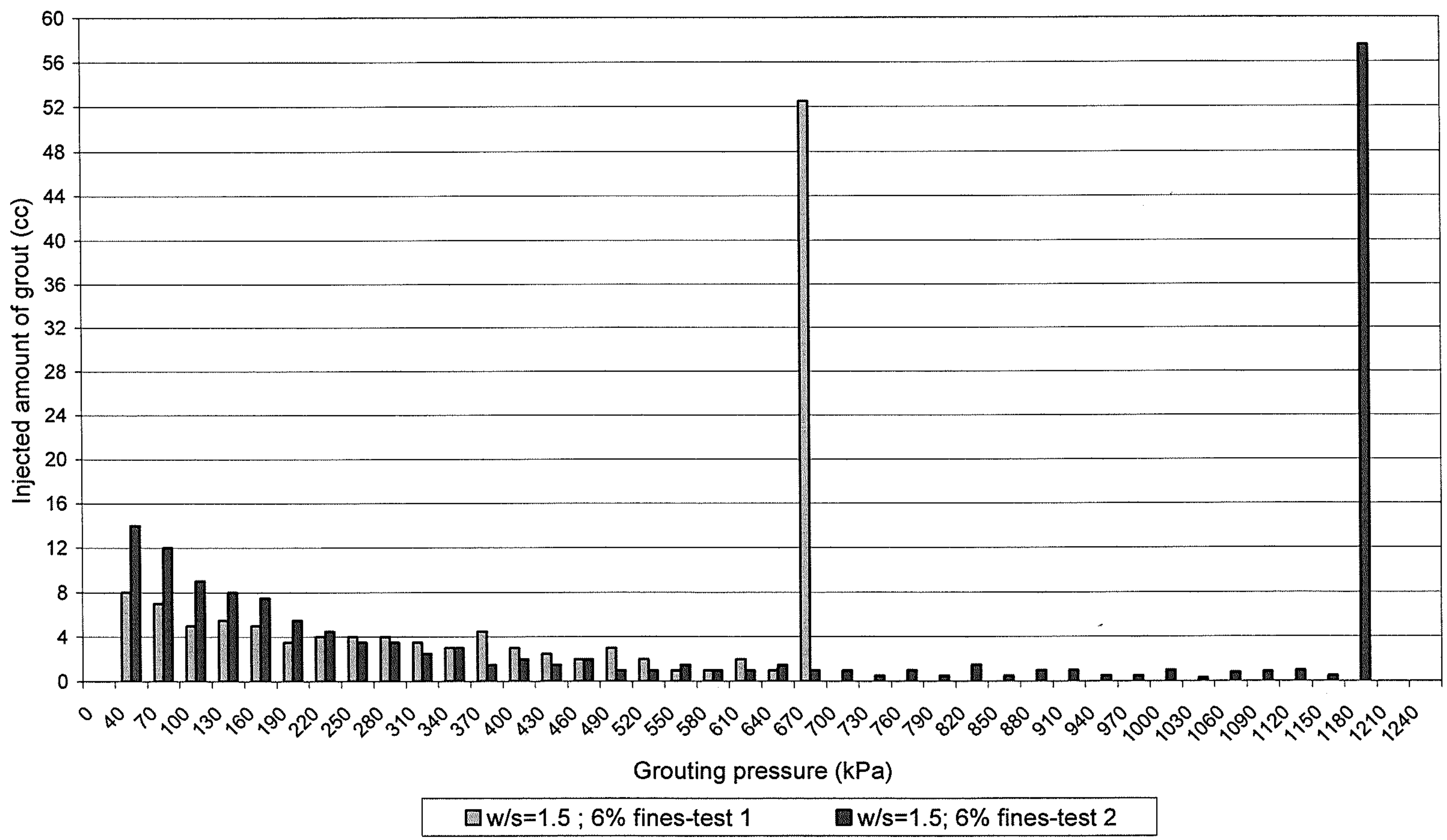


Figure 4.2 Grouting pressure versus injected grout volume; 6% fine, w/s=1.5

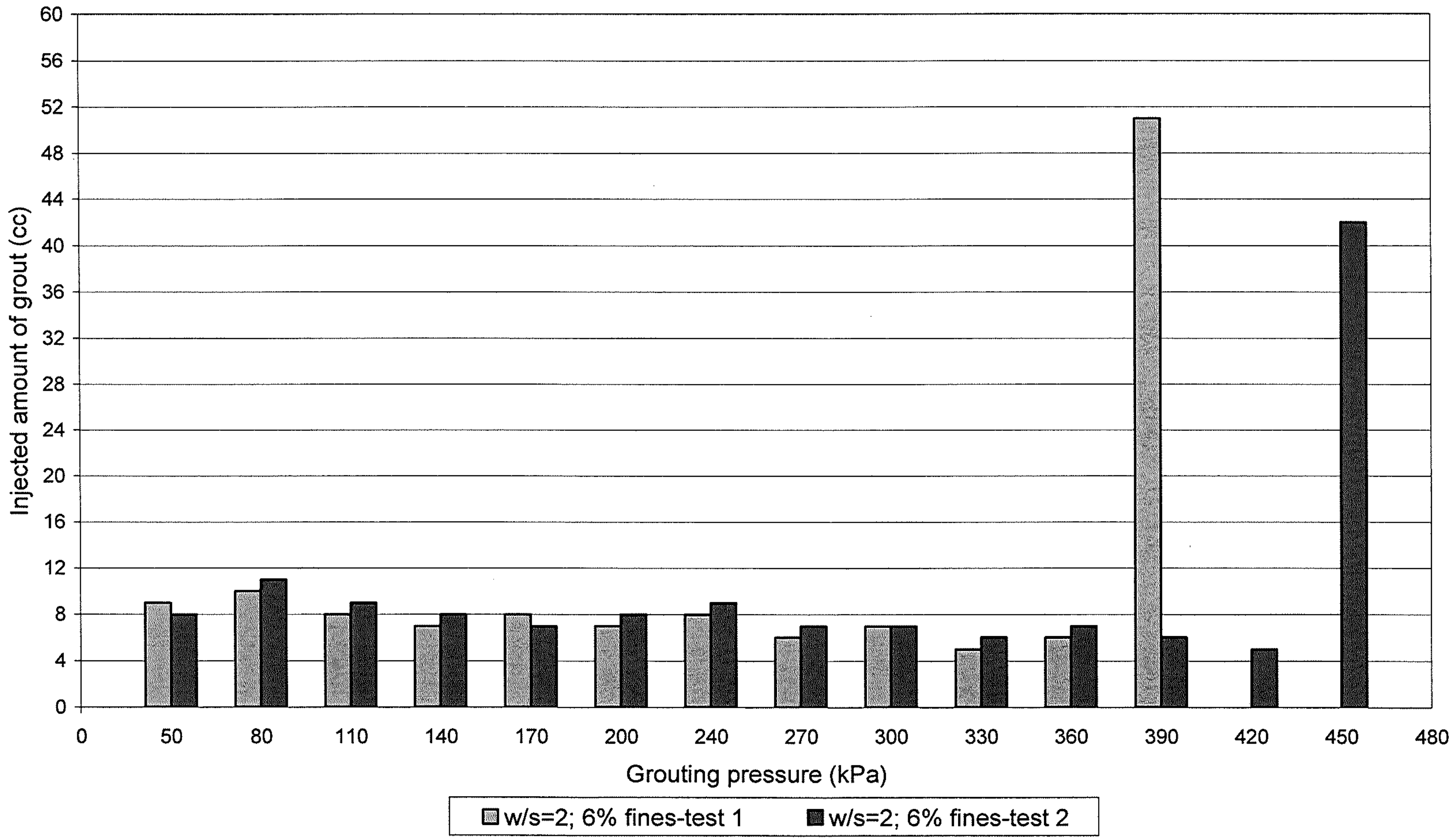


Figure 4.3 Grouting pressure versus injected grout volume; 6% fine, w/s=2.0

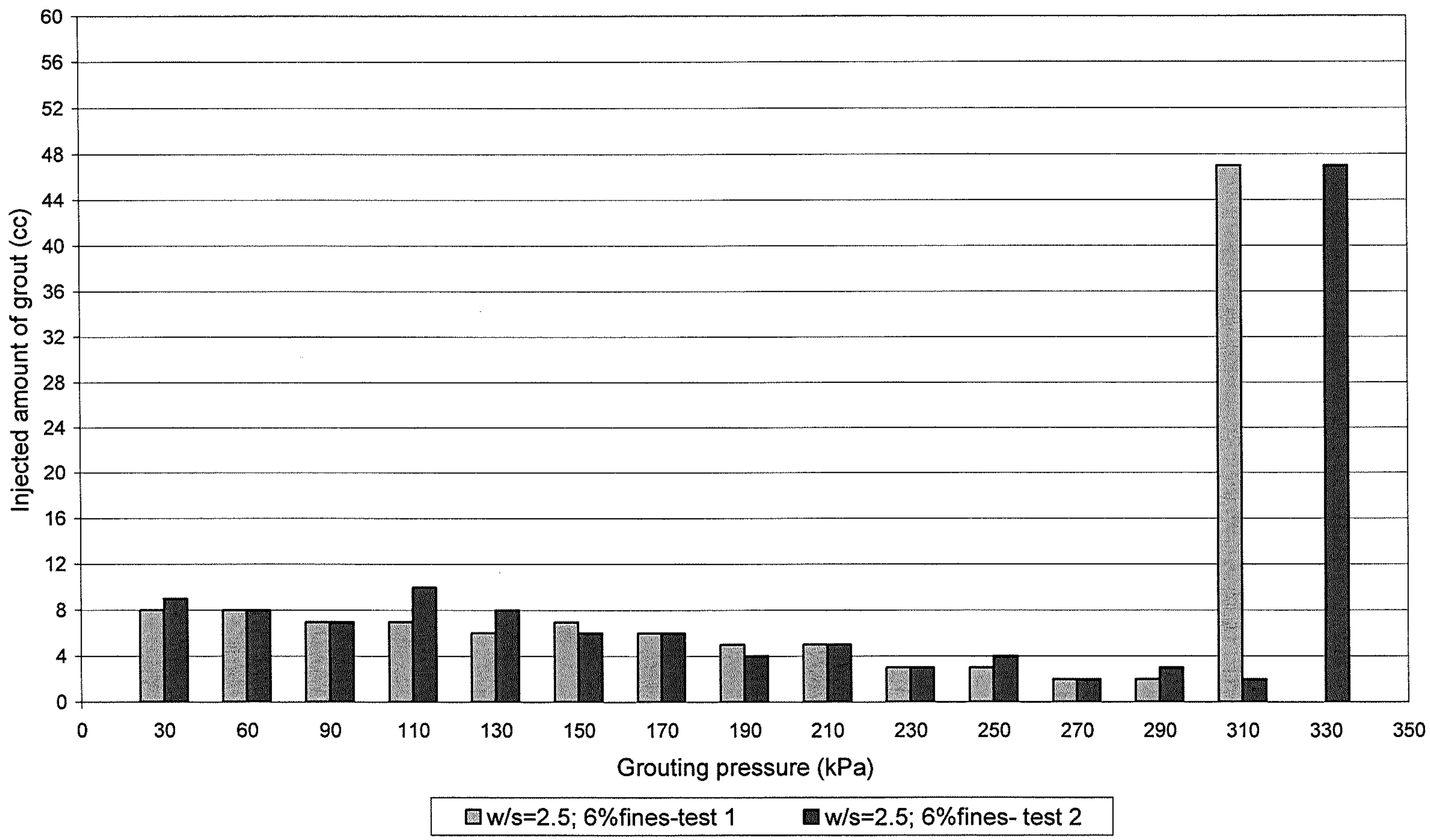


Figure 4.4 Grouting pressure versus injected grout volume; 6% fine, w/s=2.5

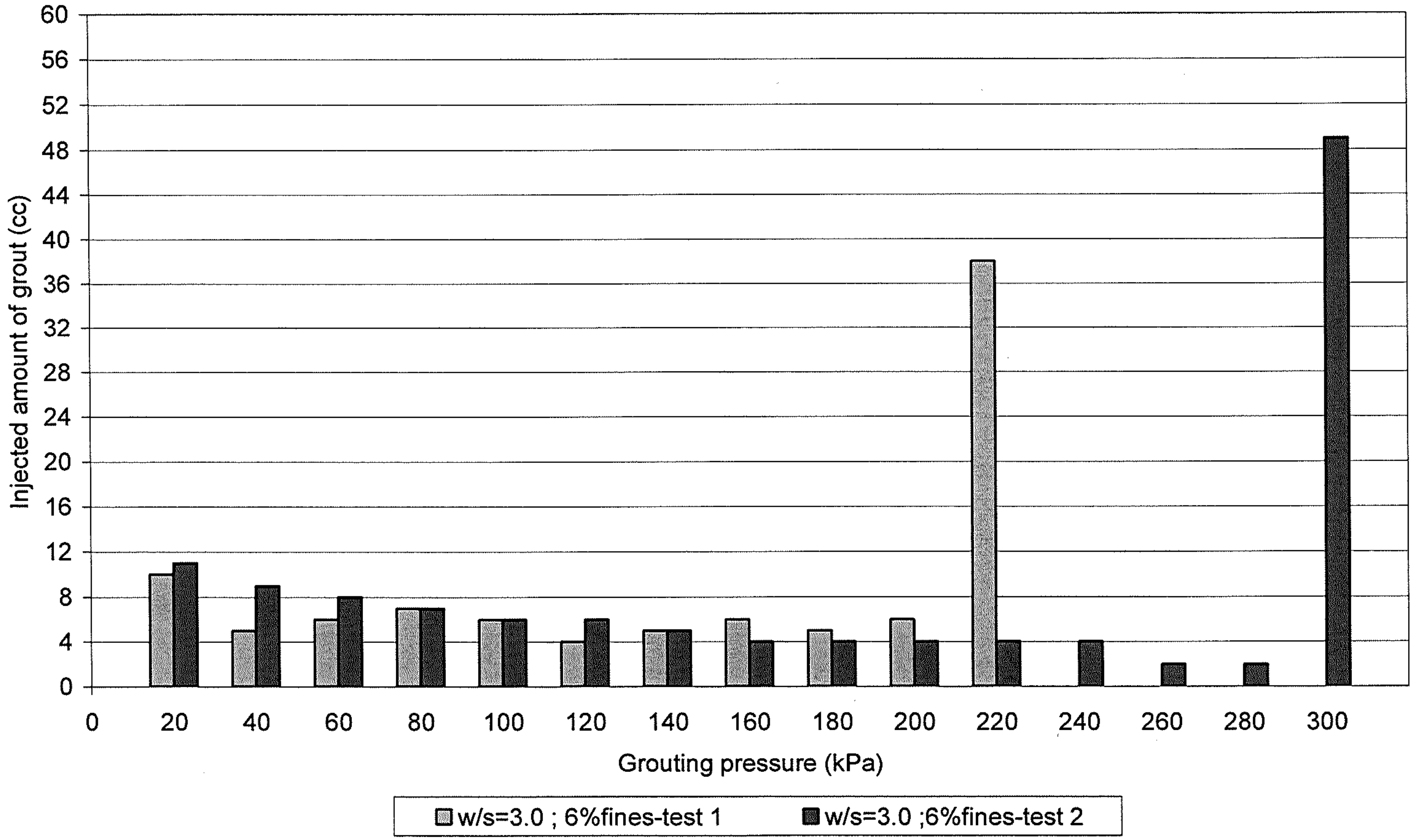


Figure 4.5 Grouting pressure versus injected grout volume; 6% fine, w/s=3.0

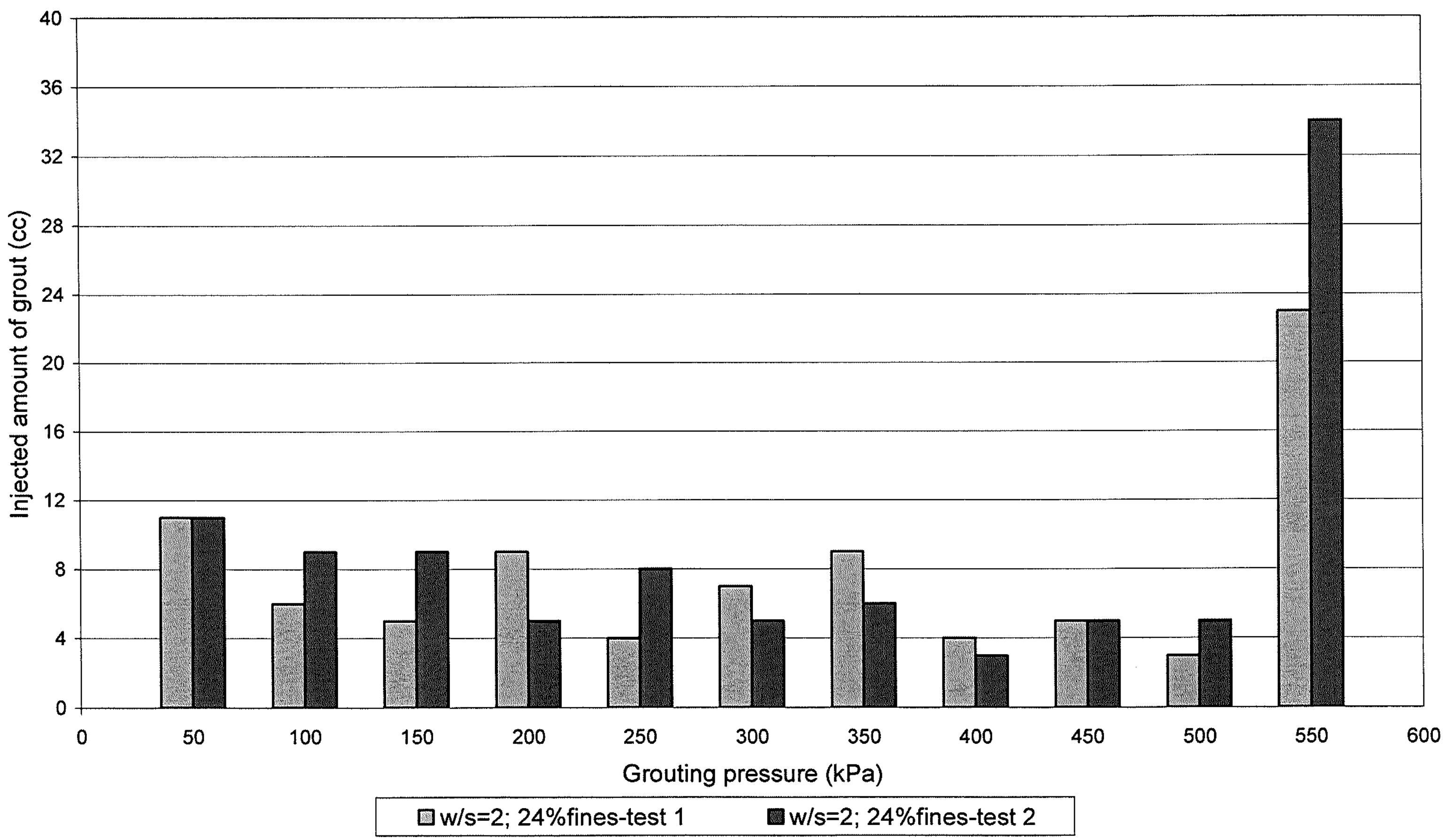


Figure 4.6 Grouting pressure versus injected grout volume; 24% fine, w/s=2.0

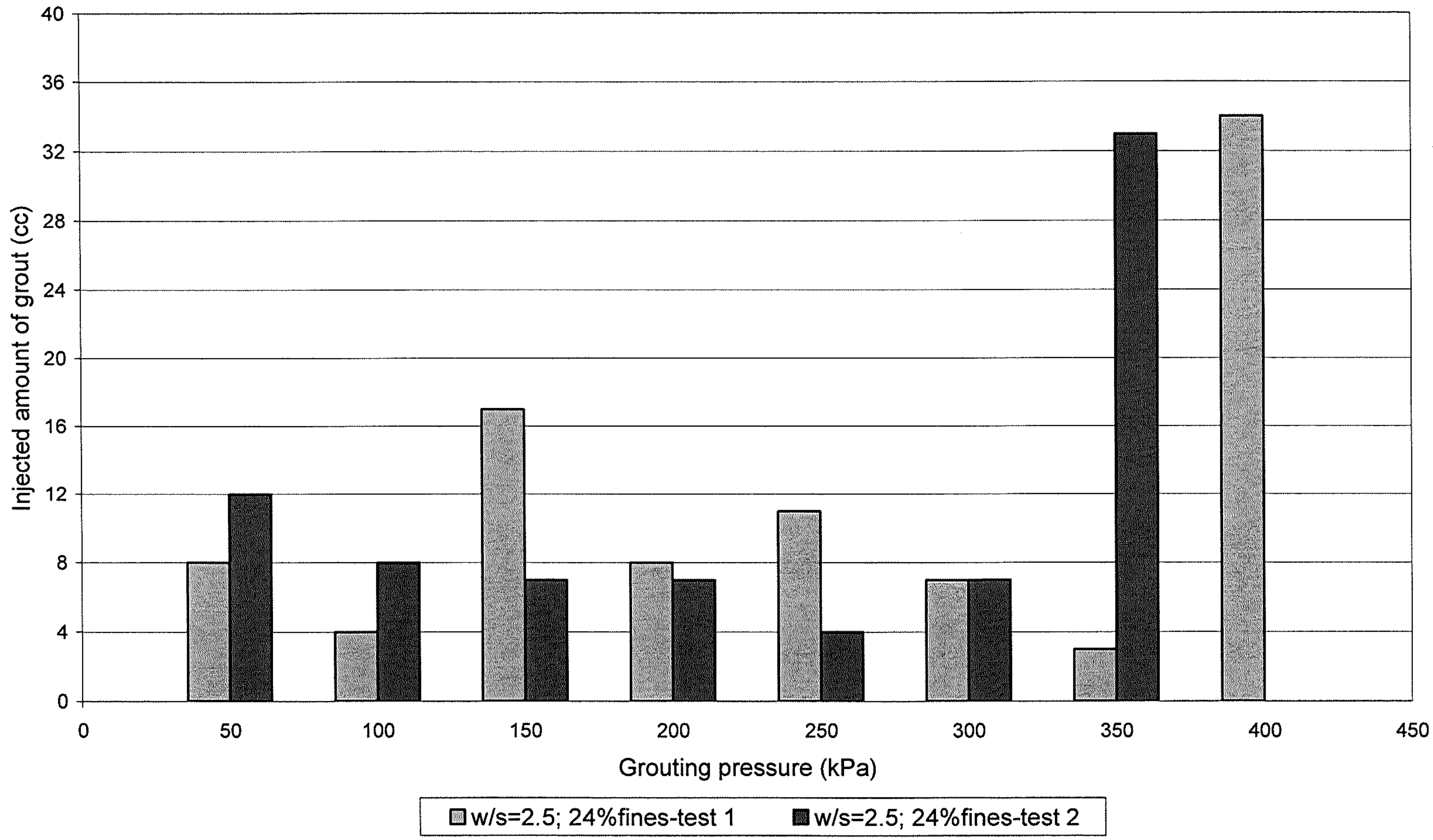


Figure 4.7 Grouting pressure versus injected grout volume; 24% fine, w/s=2.5

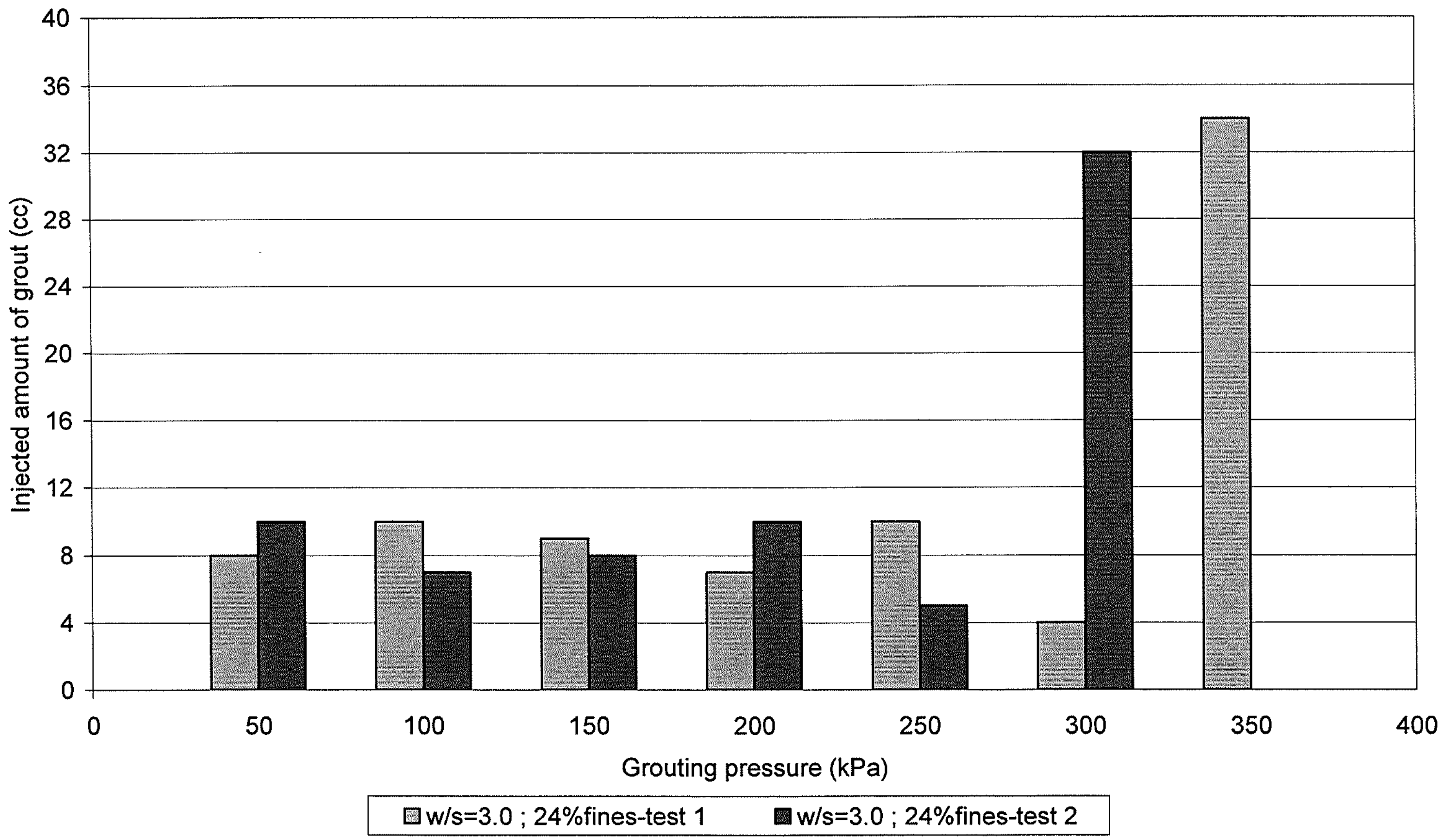


Figure 4.8 Grouting pressure versus injected grout volume; 24% fine, w/s=3.0

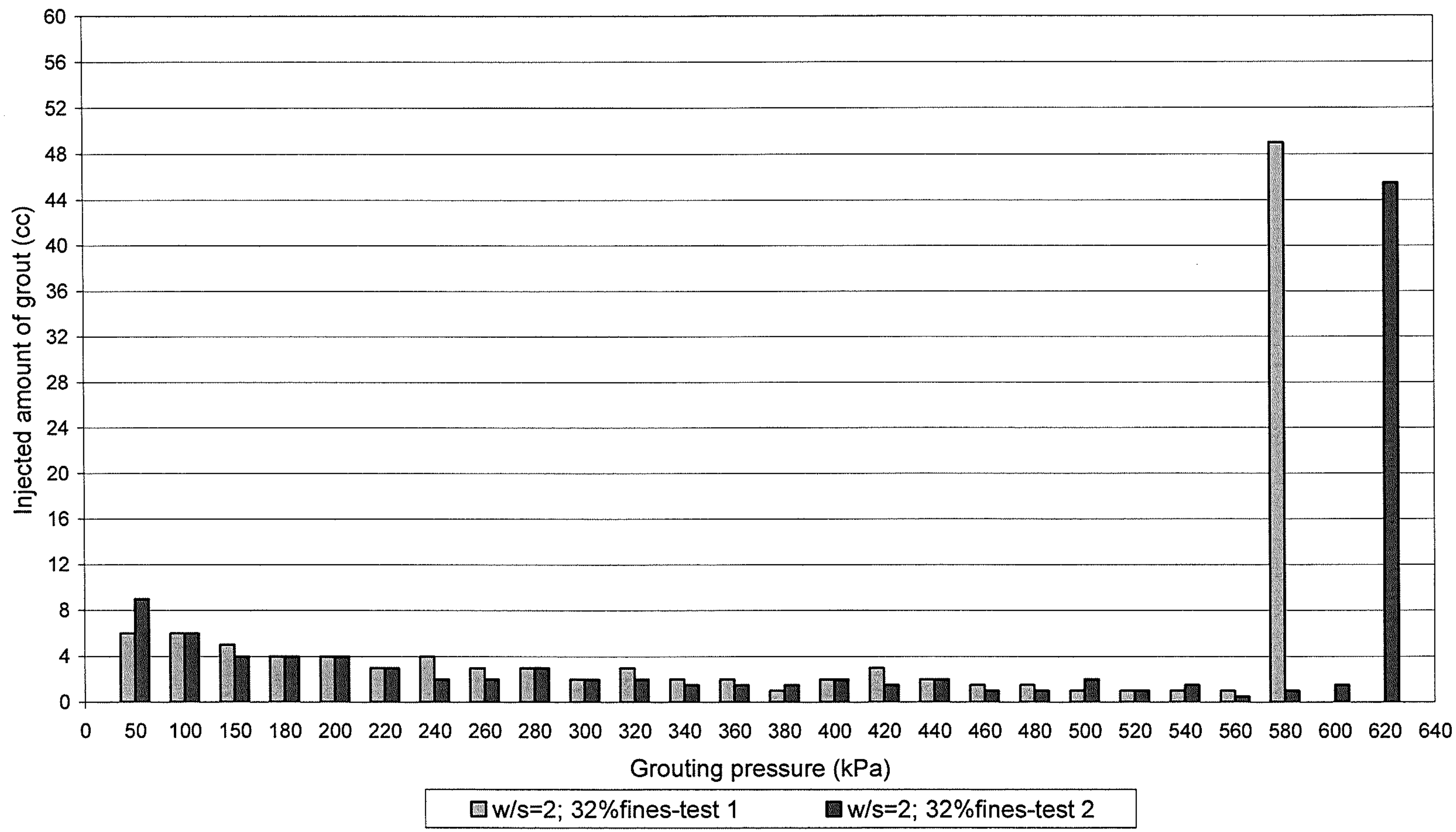


Figure 4.9 Grouting pressure versus injected grout volume; 32% fine, w/s=2.0

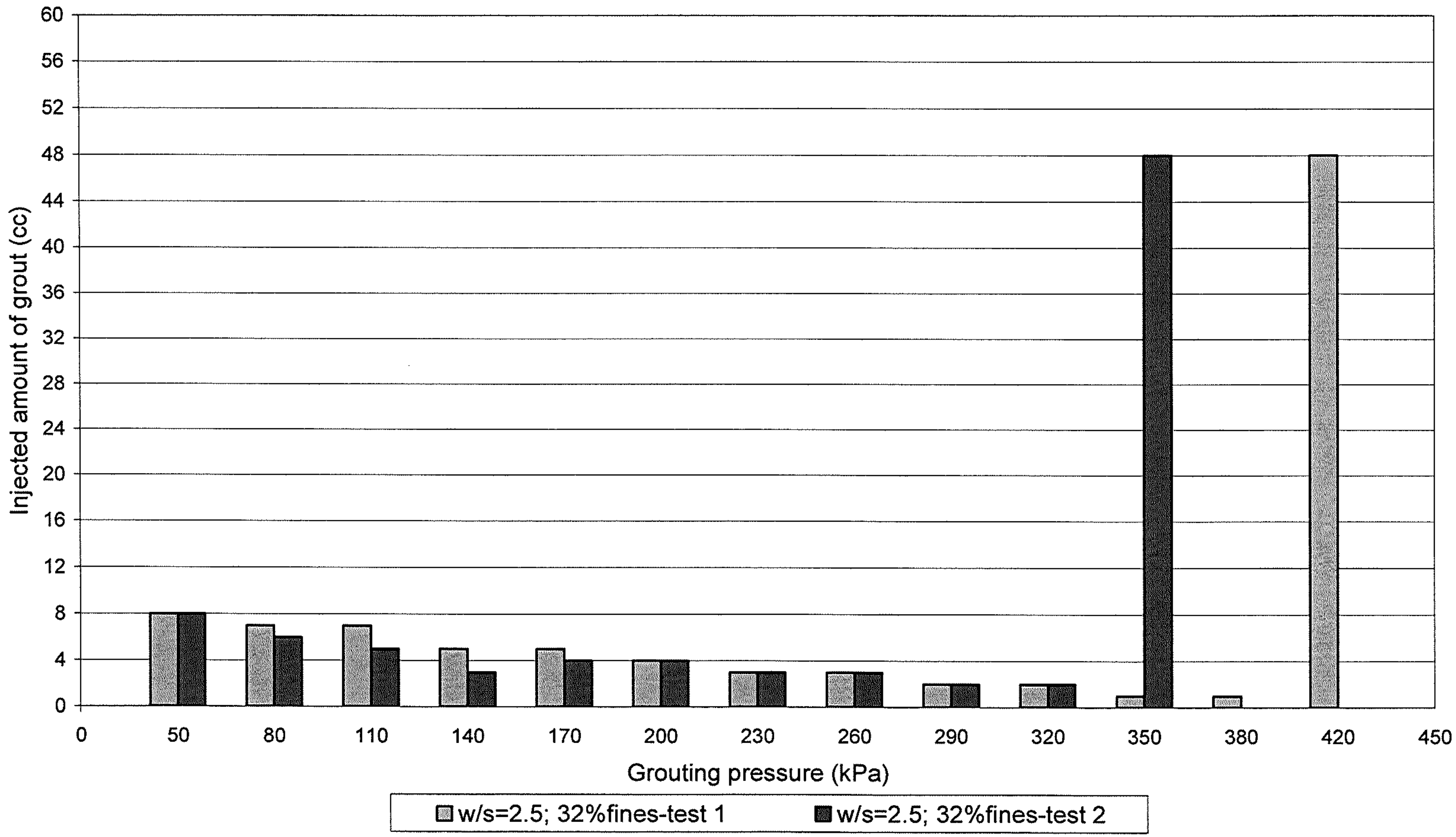


Figure 4.10 Grouting pressure versus injected grout volume; 32% fine, w/s=2.5

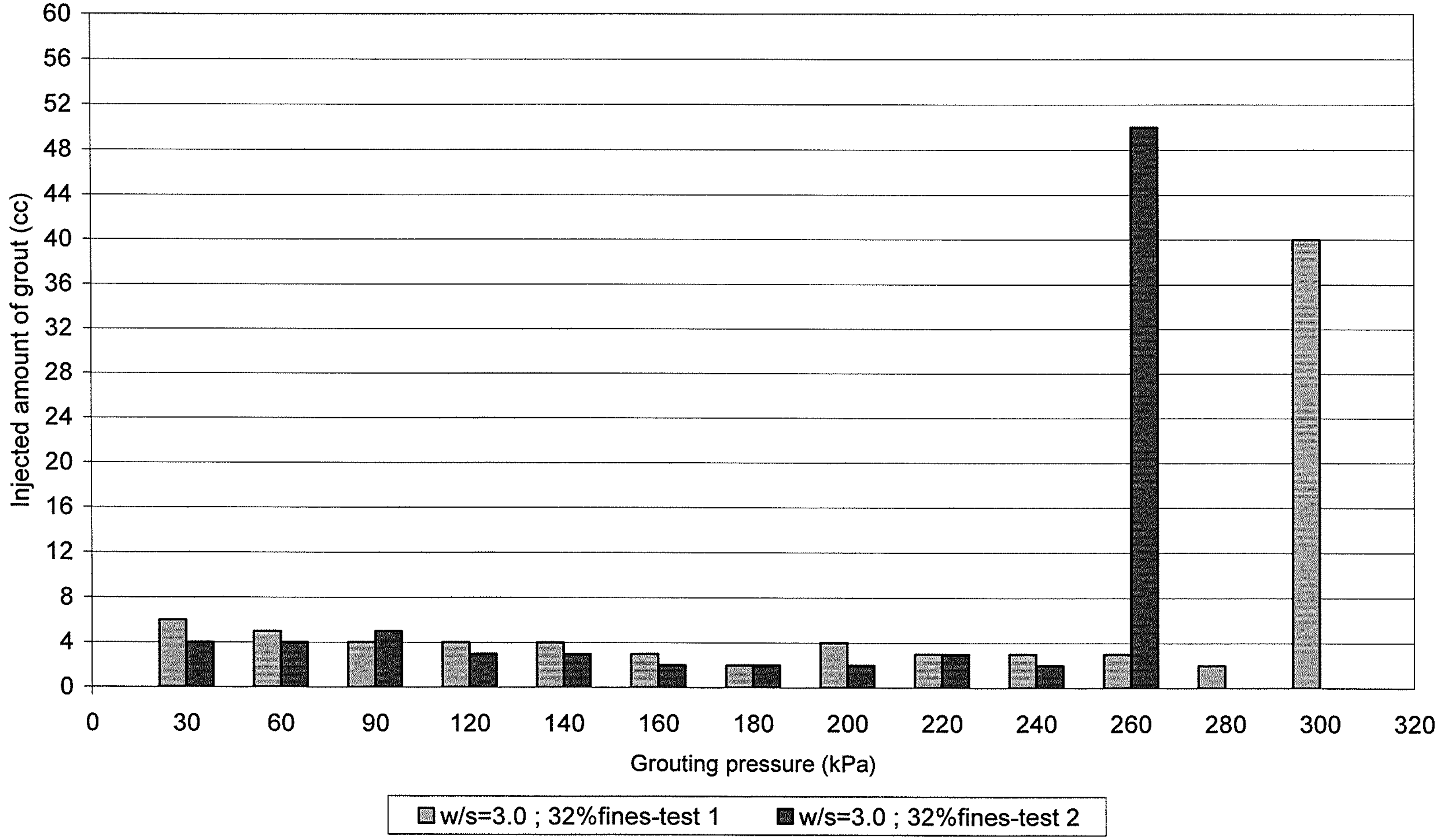


Figure 4.11 Grouting pressure versus injected grout volume; 32% fine, w/s=3.0

Graphical representation of grouting pressure versus cummulative volume of grout material injected into soil specimens of different fine content with a relative density of 40 % according to various water/solids ratios of grout can be seen in Figures 4.12 to 4.14.

Volumes of grout material injected into all of the soil specimens with a relative density of 40% at each pressure increment with respect to different water/solids ratios can be seen in Figures 4.15 to 4.17.

Graphical representation of grouting pressure versus cummulative volume of grout material injected into all of the soil specimens with a relative density of 40% according to various water/solids ratios of grout can be seen in Figures 4.18 to 4.20.

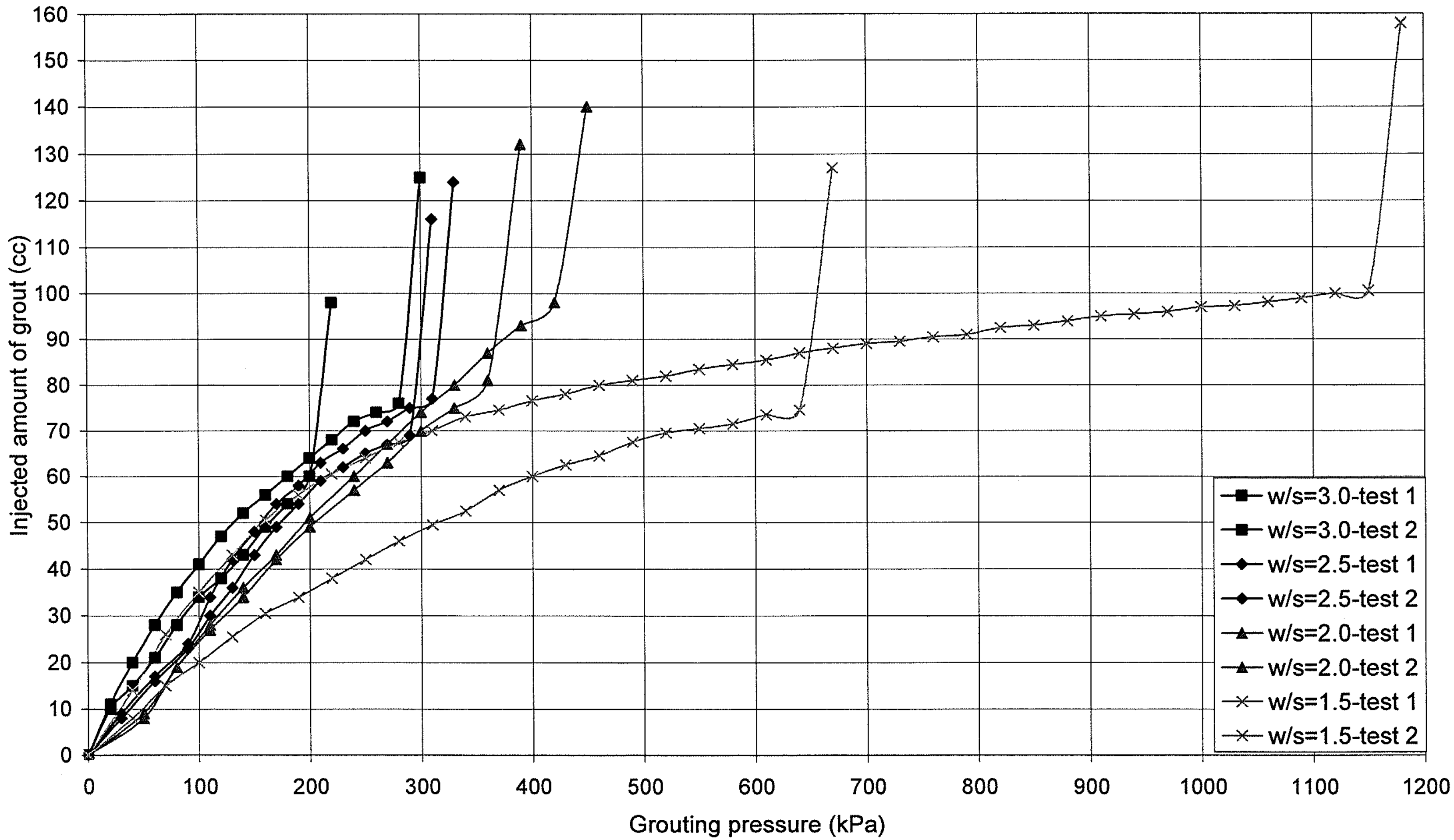


Figure 4.12 Grout pressure - cummulative grout volume relationship; 6% fine; 40 % relative density

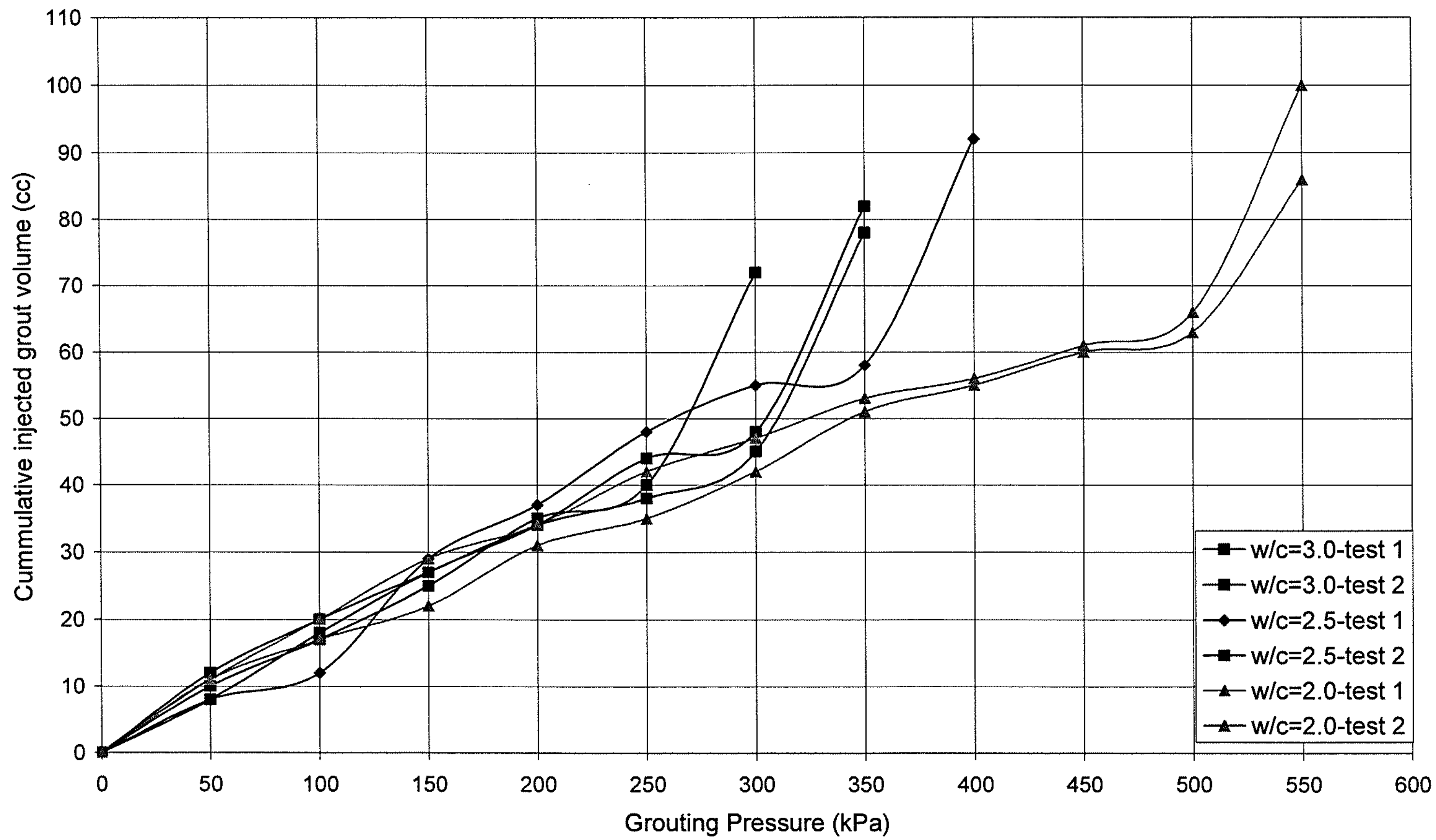


Figure 4.13 Grout pressure - cummulative grout volume relationship; 24% fine; 40 % relative density

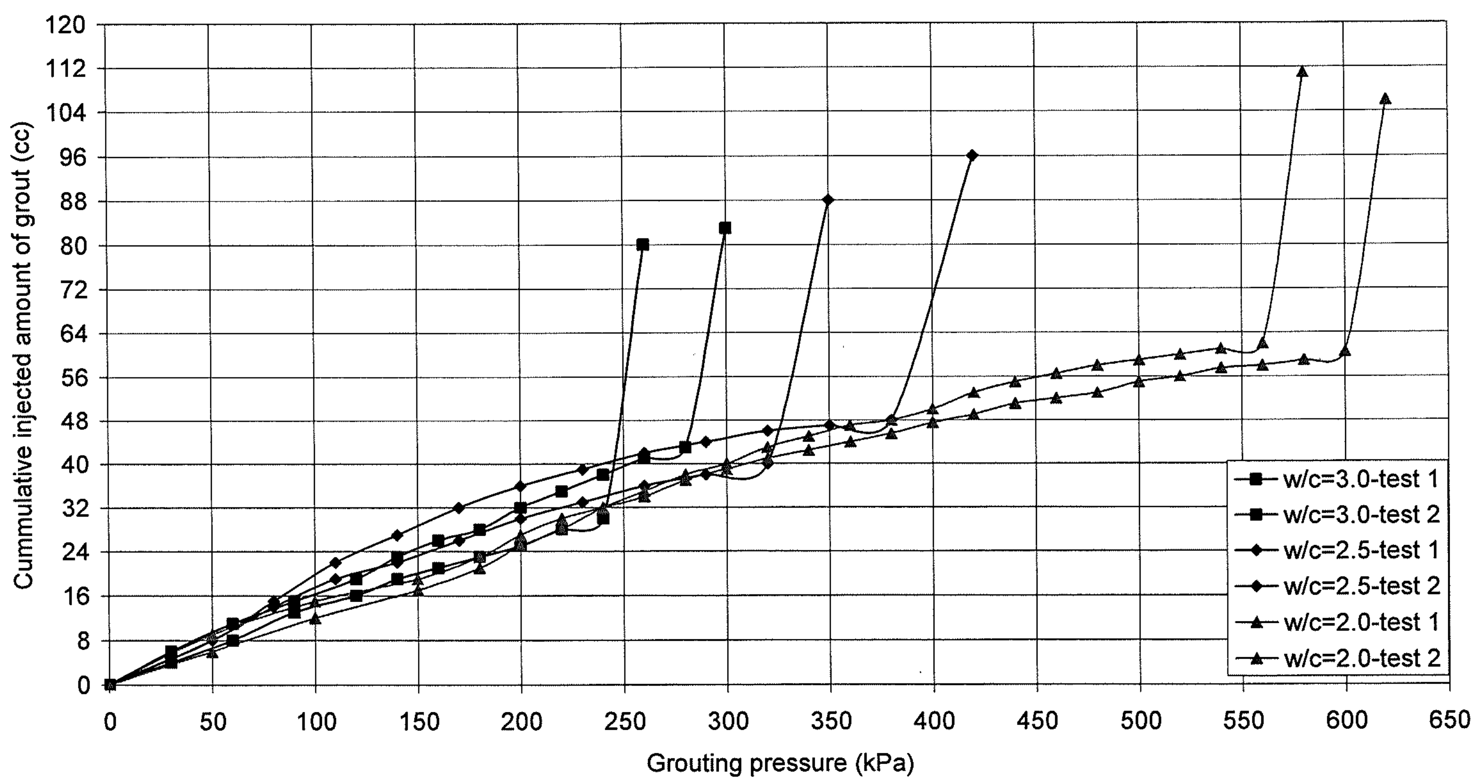


Figure 4.14 Grout pressure - cummulative grout volume relationship; 32% fine; 40 % relative density

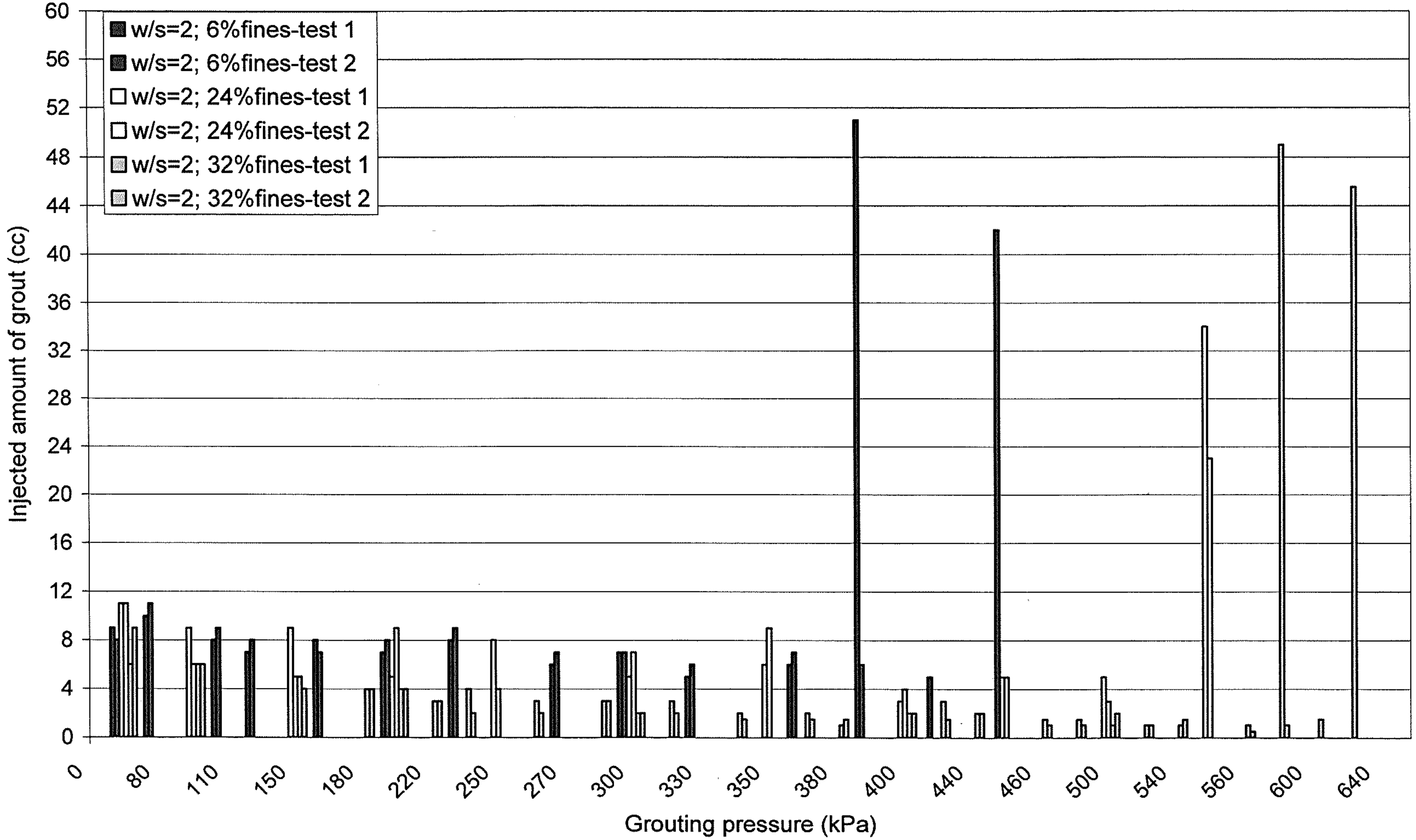


Figure 4.15 Grouting pressure versus injected grout volume in all soil specimens; w/s=2.0; 40 % relative density

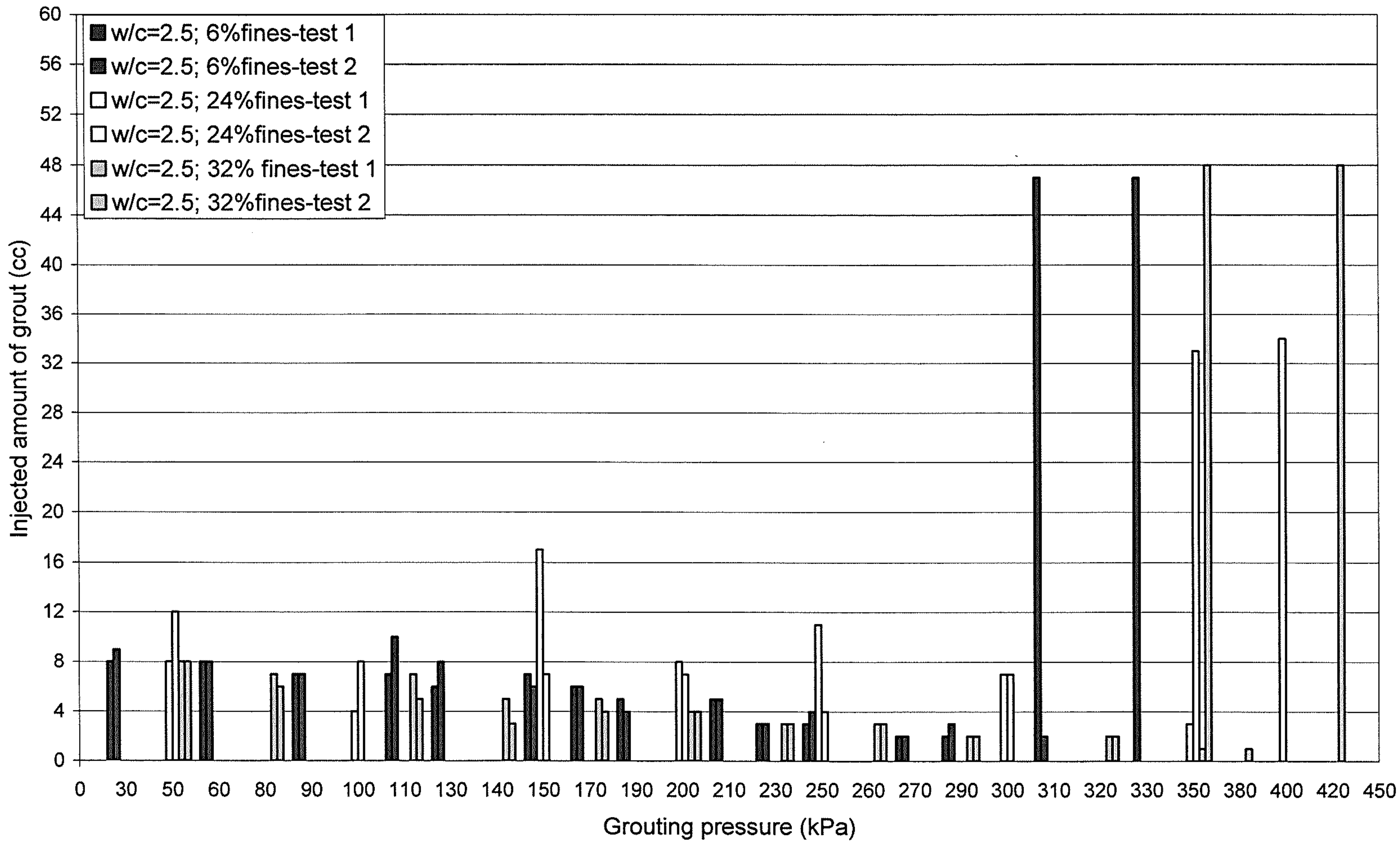


Figure 4.16 Grouting pressure versus injected grout volume in all soil specimens; w/s=2.5; 40 % relative density

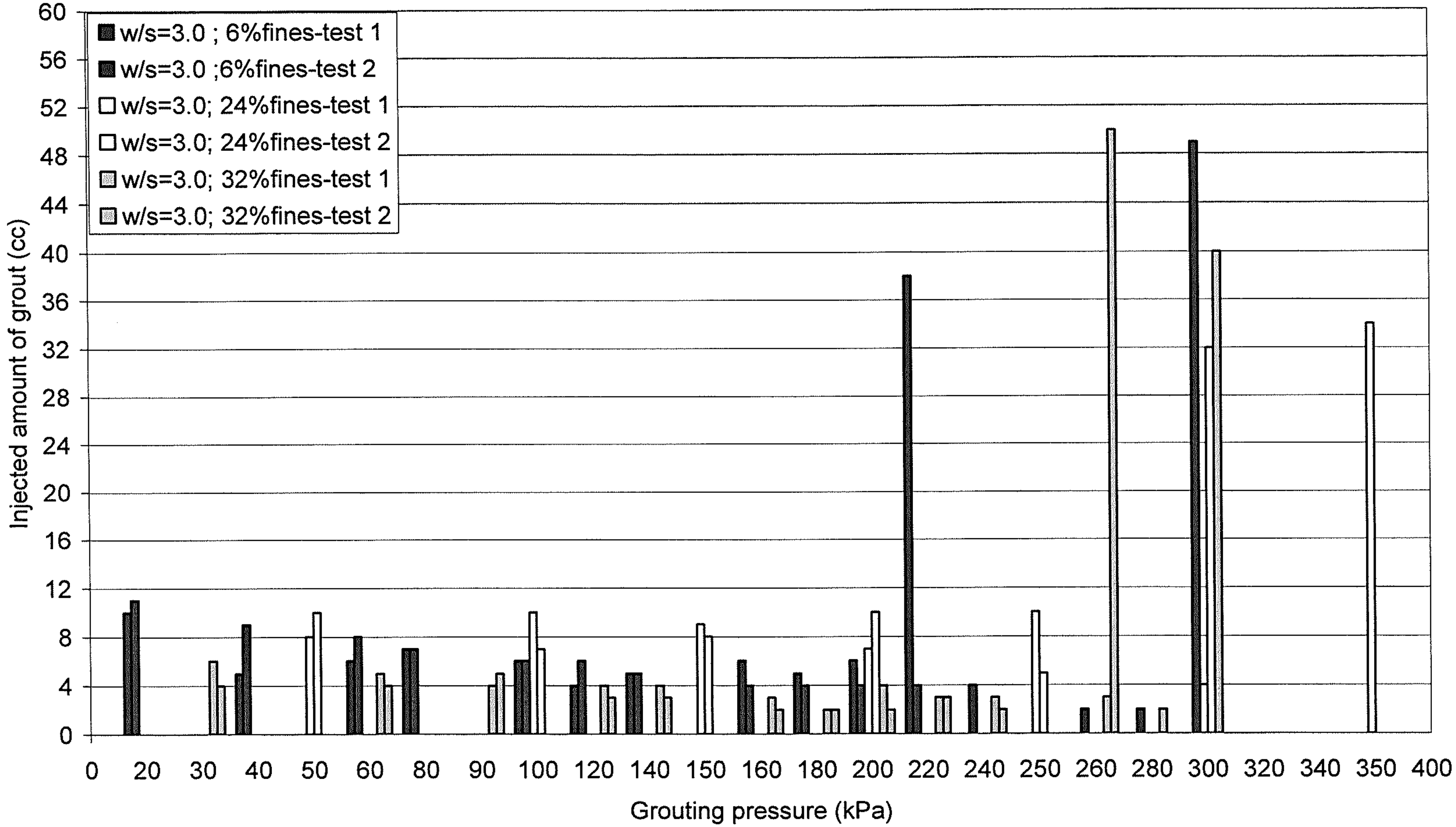


Figure 4.17 Grouting pressure versus injected grout volume in all soil specimens; w/s=3.0; 40 % relative density

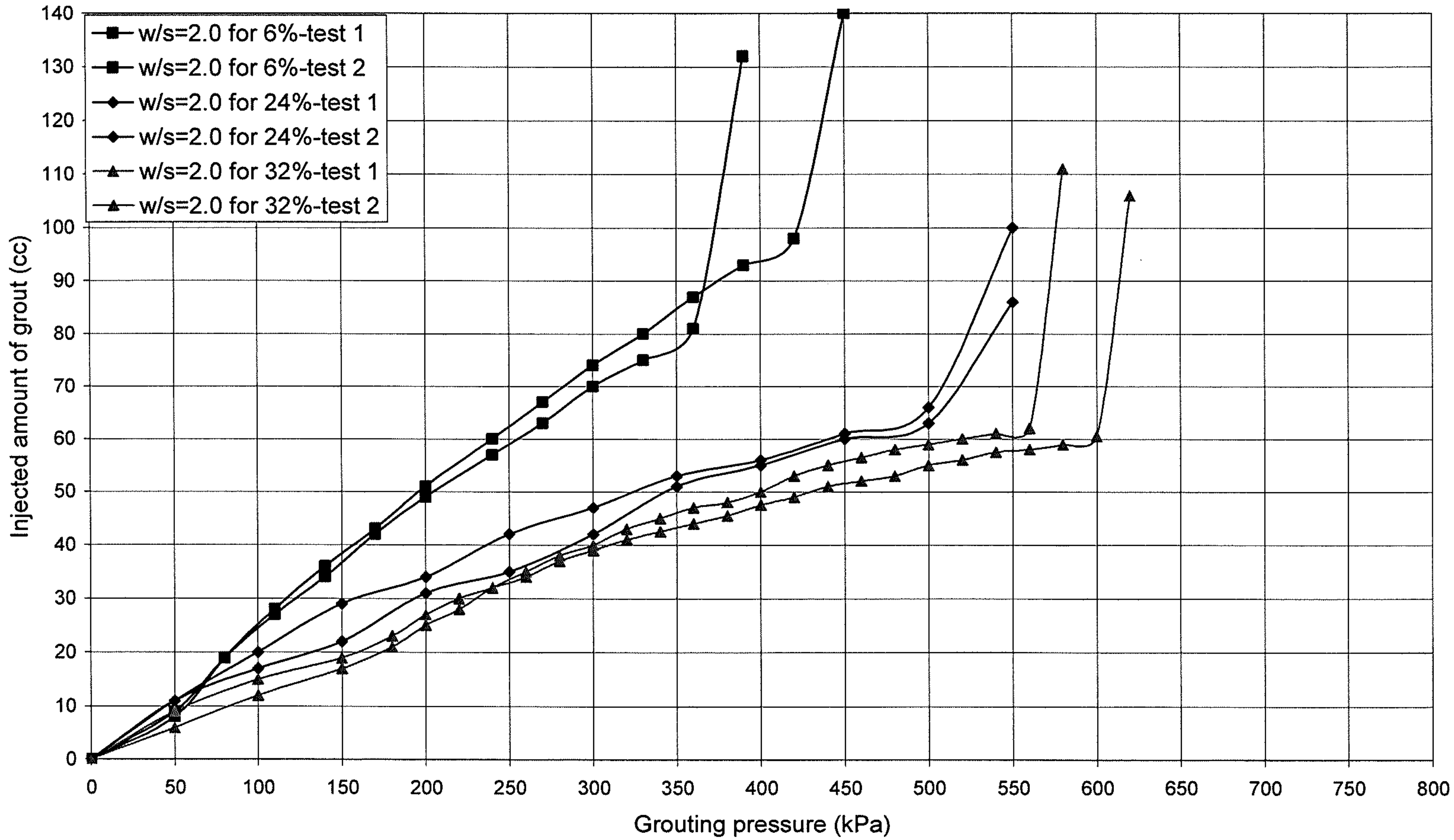


Figure 4.18 Grouting pressure - cummulative grout volume relationship for all soils; w/s=2.0; 40 % relative density

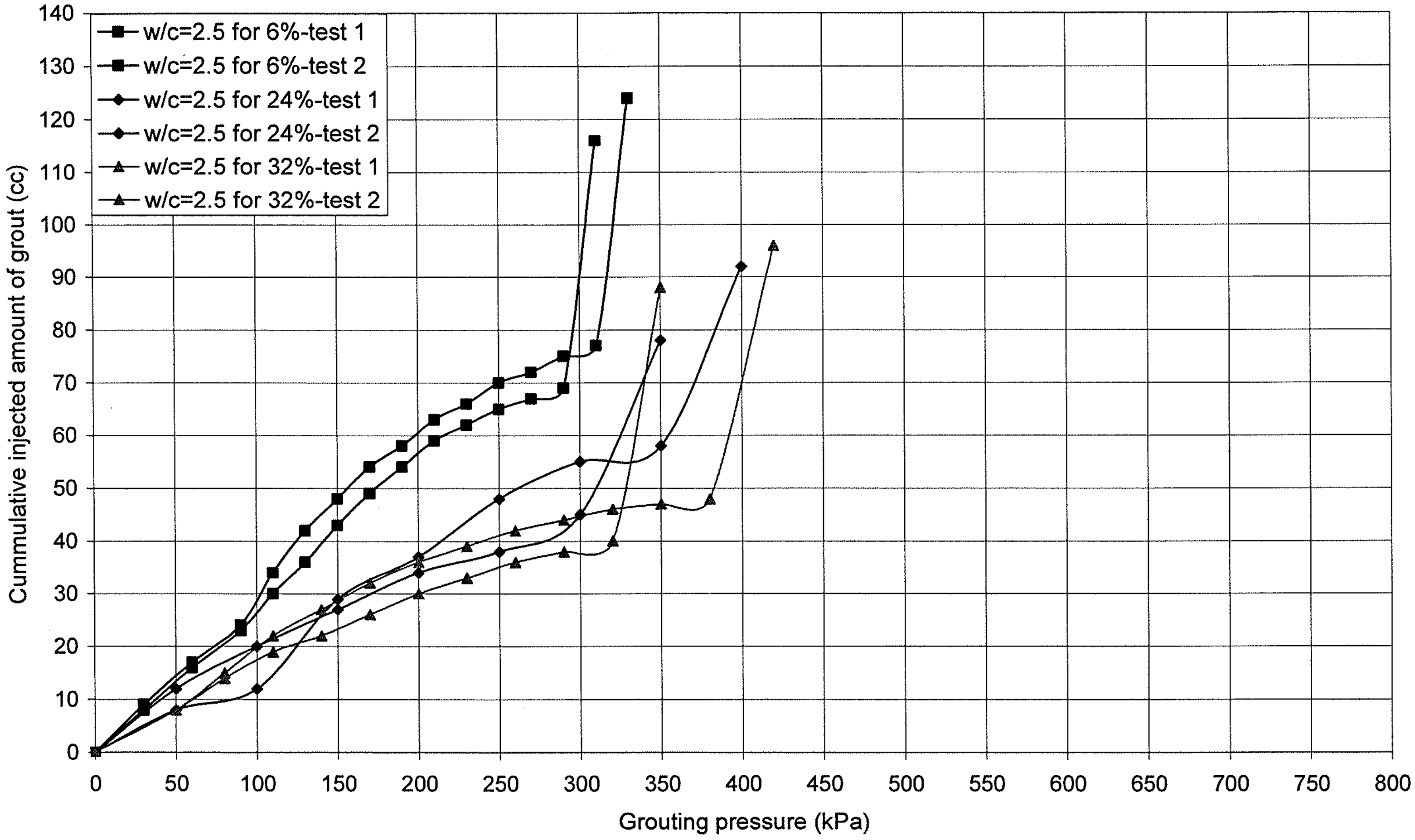


Figure 4.19 Grouting pressure - cummulative grout volume relationship for all soils; w/s=2.5; 40 % relative density

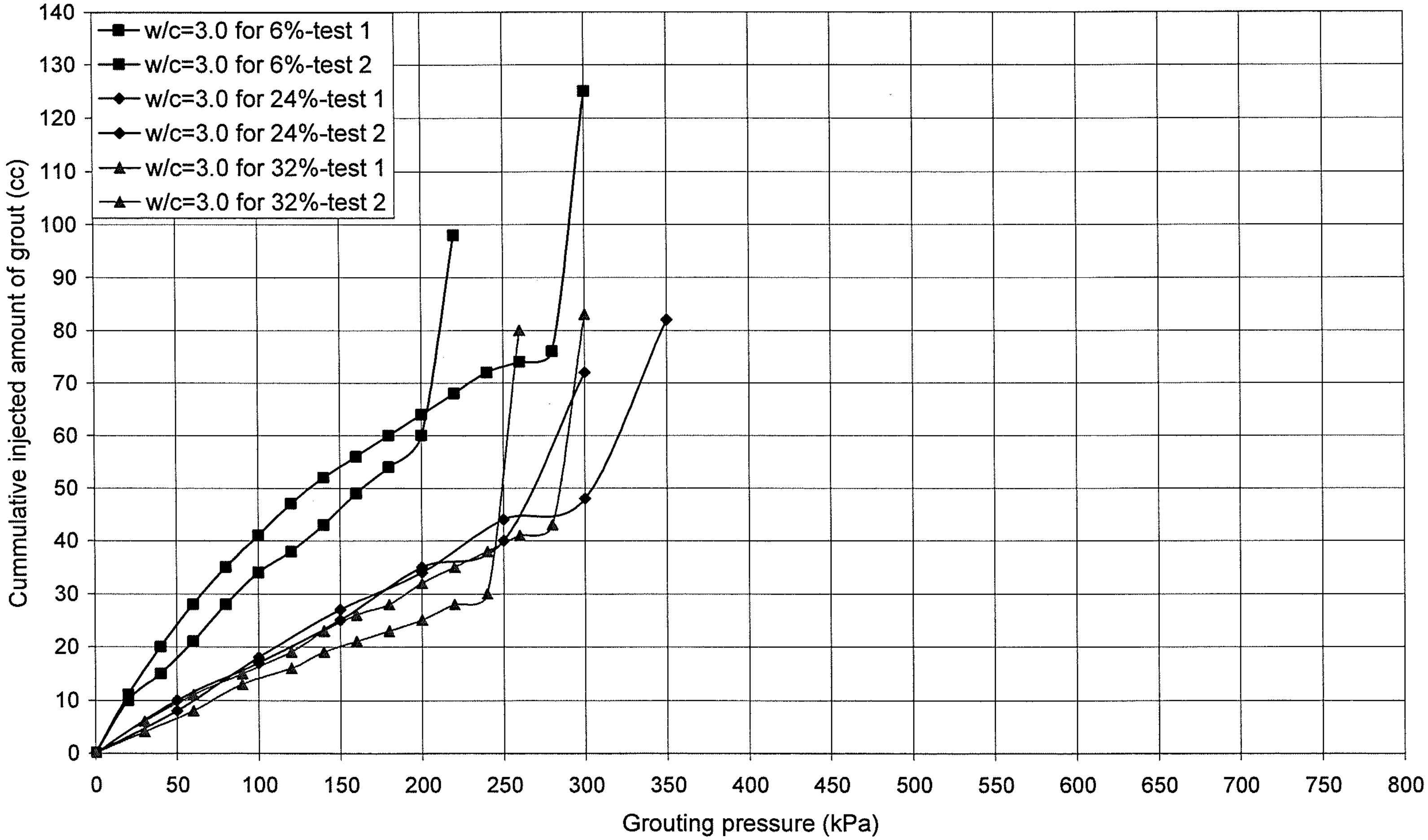


Figure 4.20 Grouting pressure - cummulative grout volume relationship for all soils; w/s=3.0; 40 % relative density

Relationship between the fracturing pressure and the injected grout volumes until fracturing can be seen in Figure 4.21. Effect of the relative density of the soil specimen is also presented in this graphical representation.

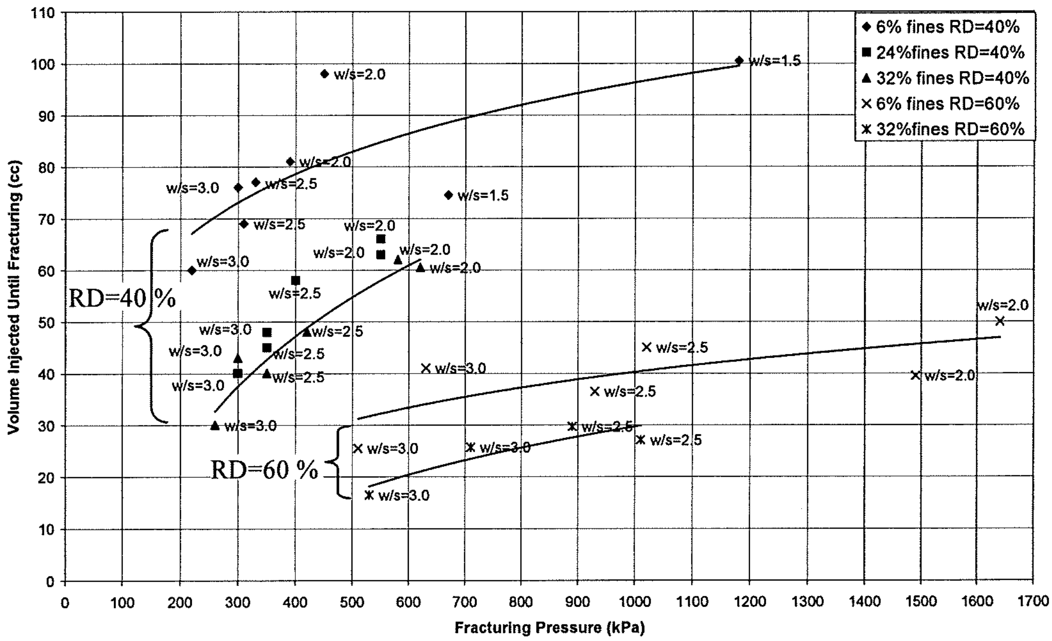


Figure 4.21 Effects of w/s ratio and relative density on the volume injected until fracturing

Observed amounts of vertical heave on top of the soil specimens with a relative density of 40 % during fracturing can be seen in Figure 4.22 with respect to various water/solids ratios of grout material.

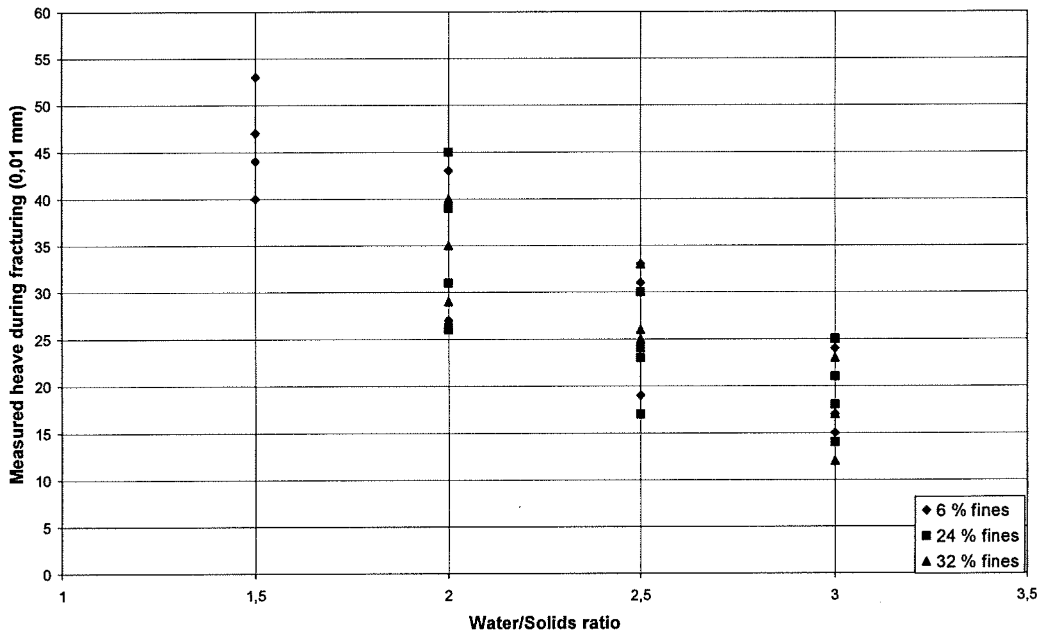


Figure 4.22 Water/solids ratio and observed heave relationship for all soil specimens

Volumes of grout material injected into soil specimens of fine content of 6 % and 32 % with a relative density of 60 % can be seen at each pressure increment in Figures 4.23 to 4.27.

Graphical representation of grouting pressure versus cummulative volume of grout material injected into soil specimens of 6% and 32% fine content with a relative density of 60% according to various water/solids ratios of grout can be seen in Figures 4.28 and 4.29 respectively.

Volumes of grout material injected into soil specimens of fine content of 6 % and 32 % with a relative density of 40 % can be seen at each pressure increment in Figures 4.30 to 4.33. Applied overburden stress in these tests are 100 kPa.

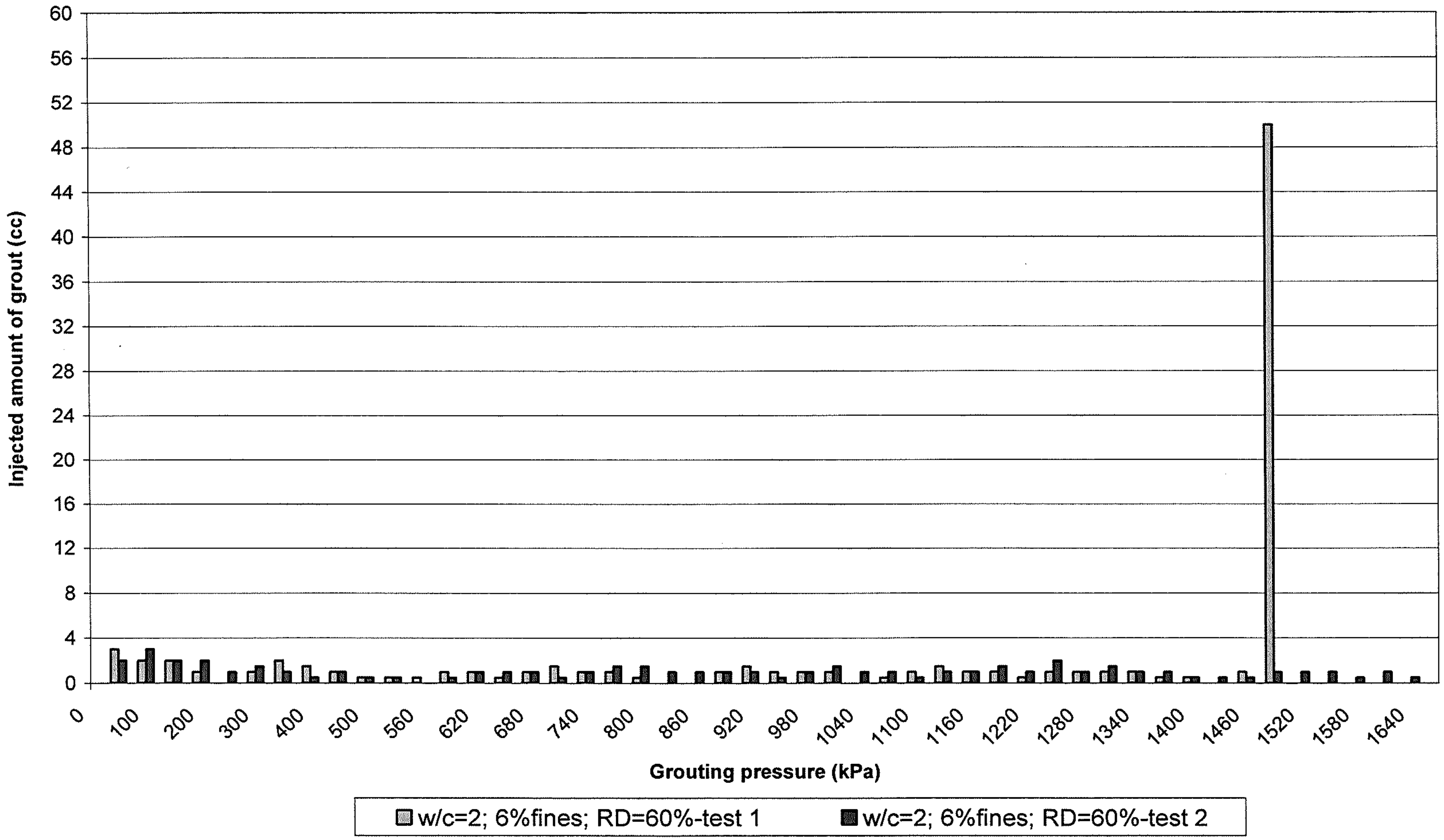


Figure 4.23. Grouting pressure versus injected grout volume; 6% fine, w/s=2.0, 60% relative density

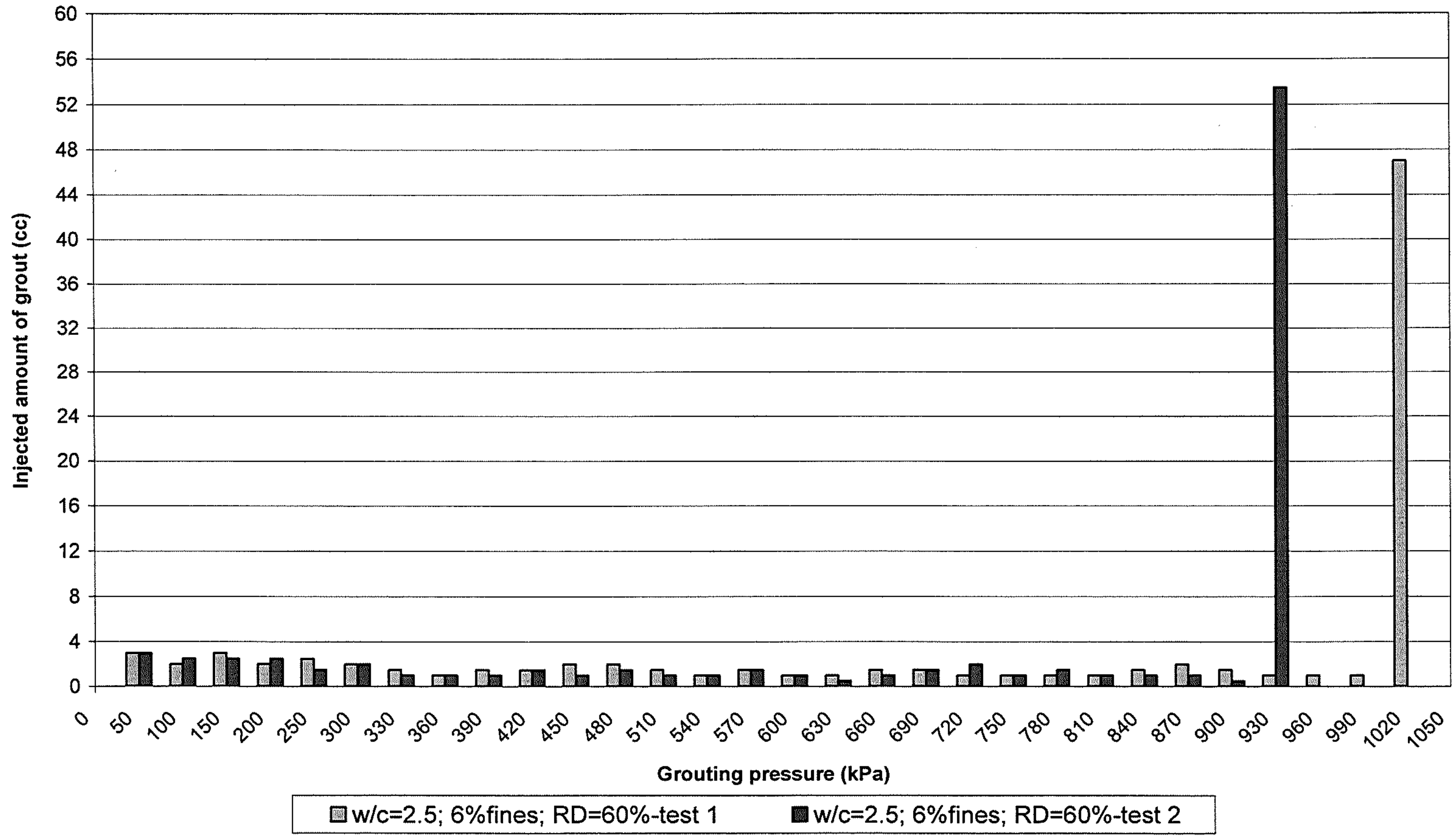


Figure 4.24 Grouting pressure versus injected grout volume; 6% fine, w/s=2.5, 60% relative density

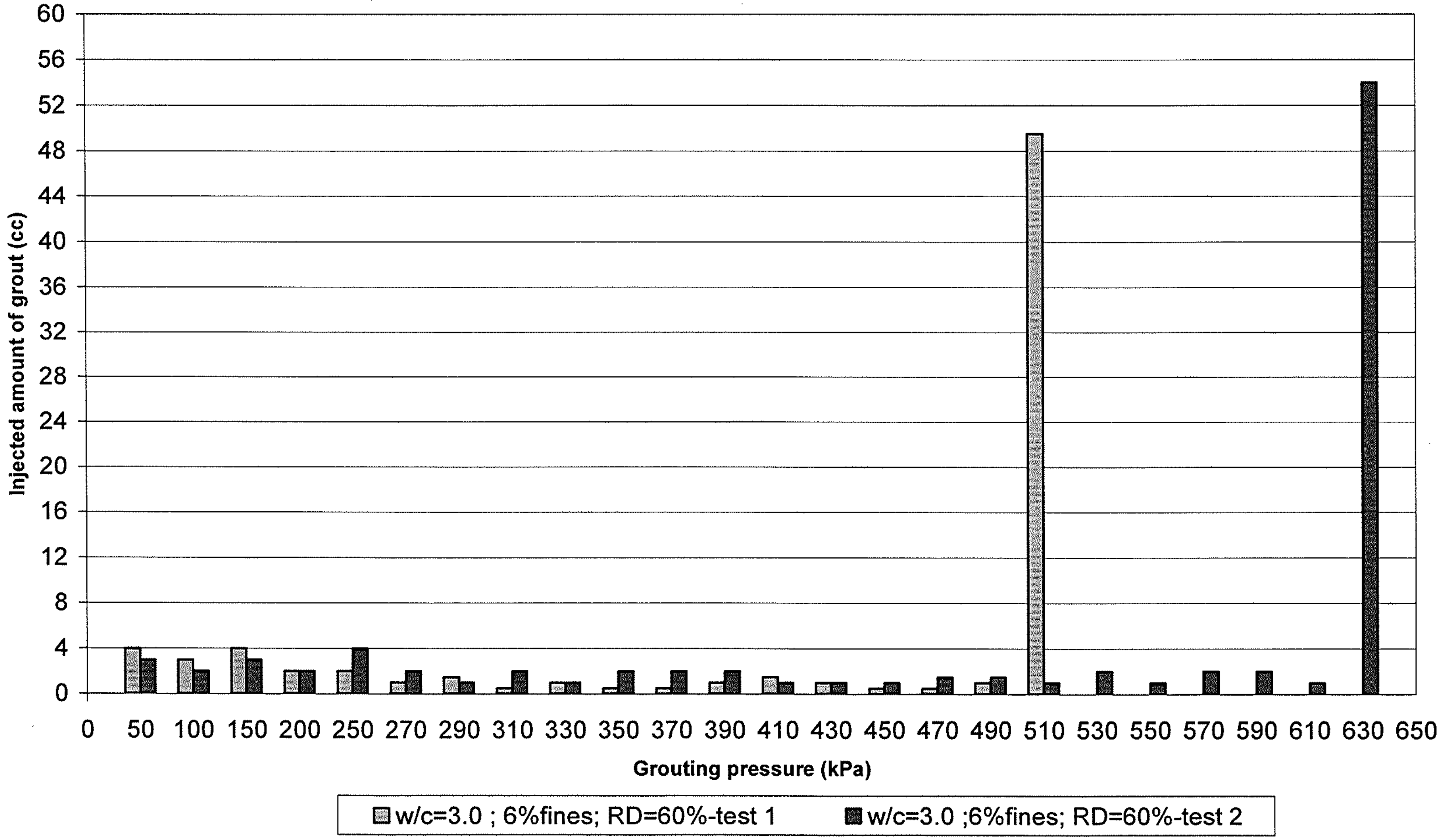


Figure 4.25 Grouting pressure versus injected grout volume; 6% fine, w/s=3.0, 60% relative density

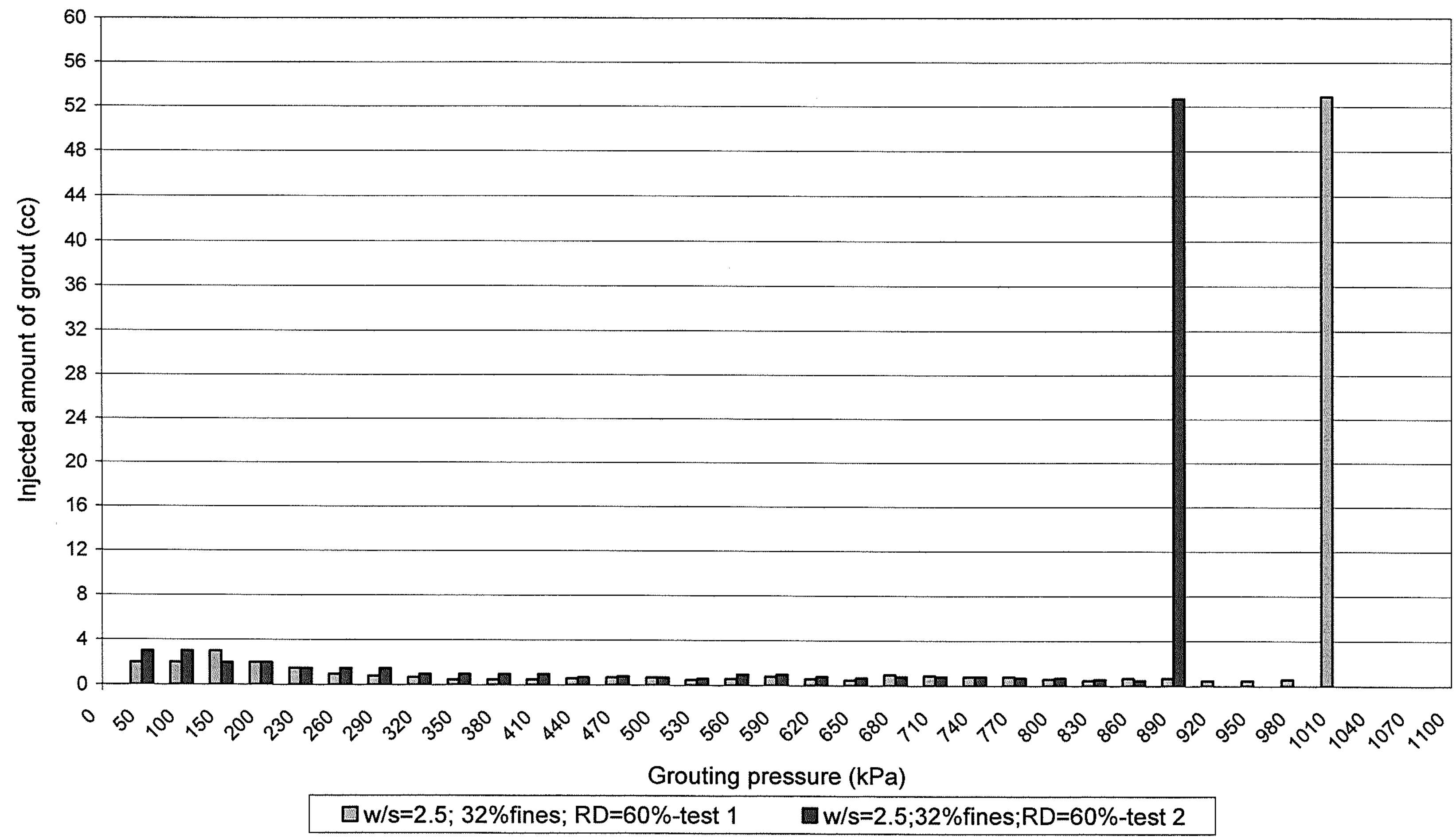


Figure 4.26 Grouting pressure versus injected grout volume; 32% fine, w/s=2.5, 60% relative density

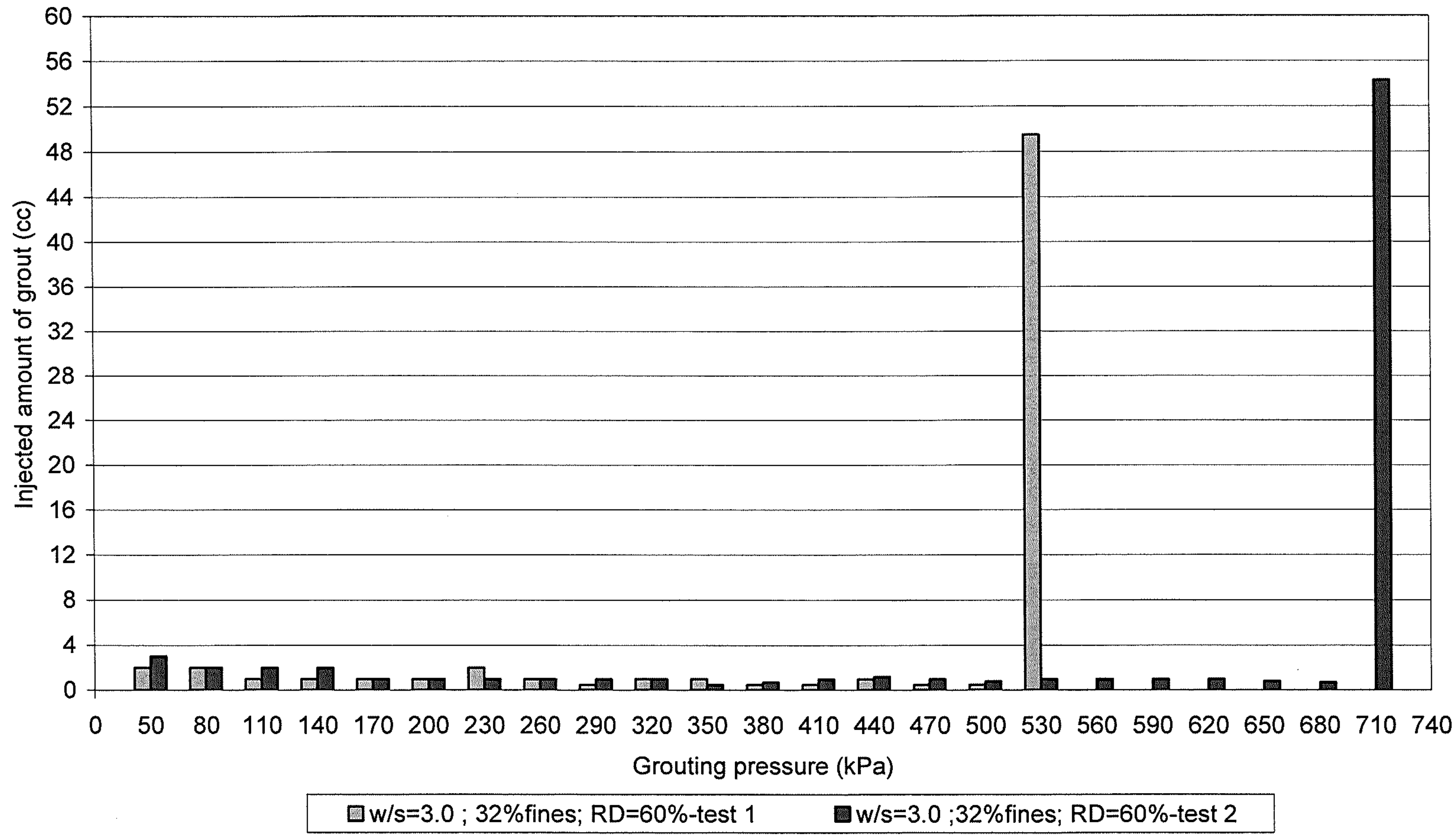


Figure 4.27 Grouting pressure versus injected grout volume; 32% fine, w/s=3.0, 60% relative density

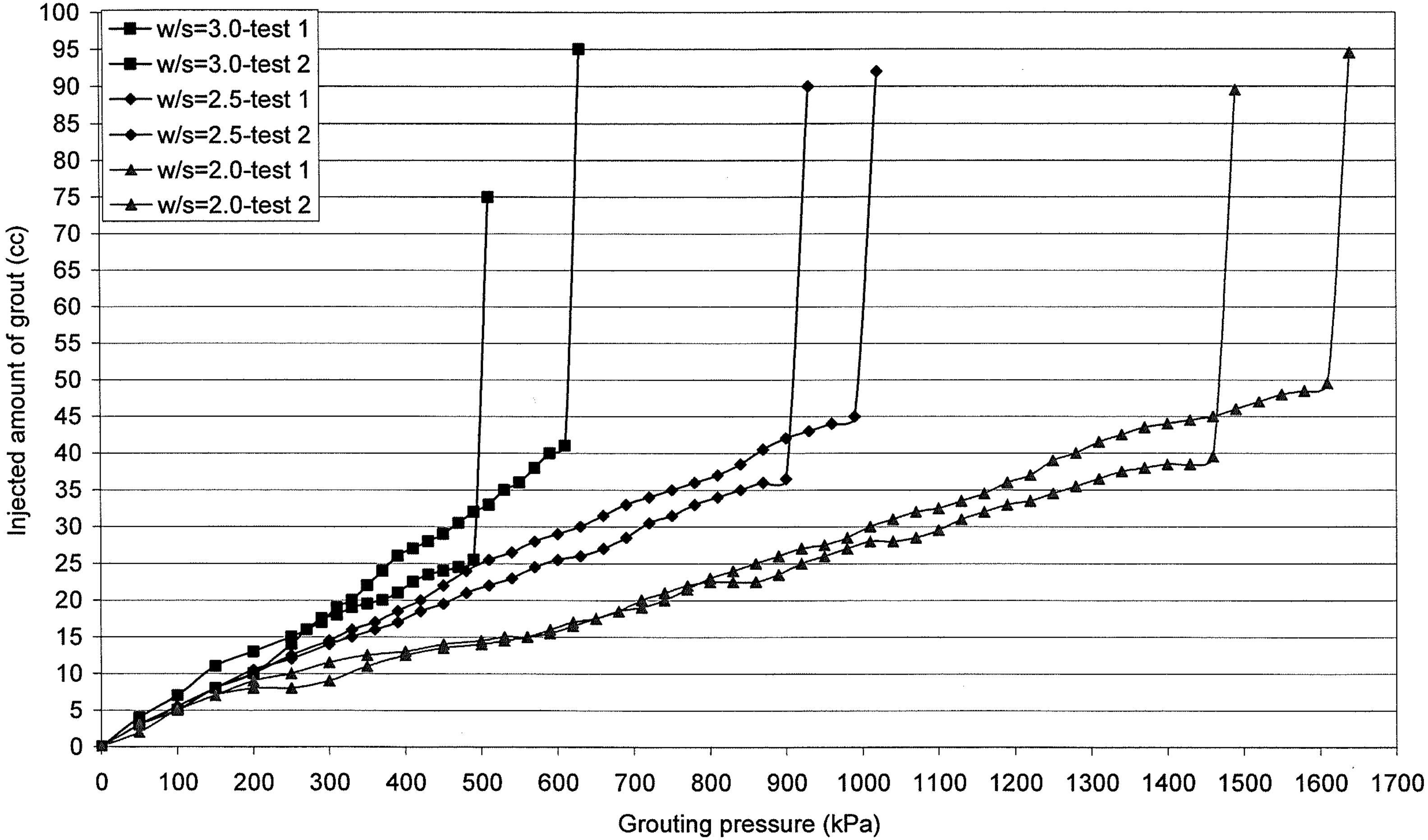


Figure 4.28 Grouting pressure - cummulative grout volume relationship; 6% fine; 60 % relative density

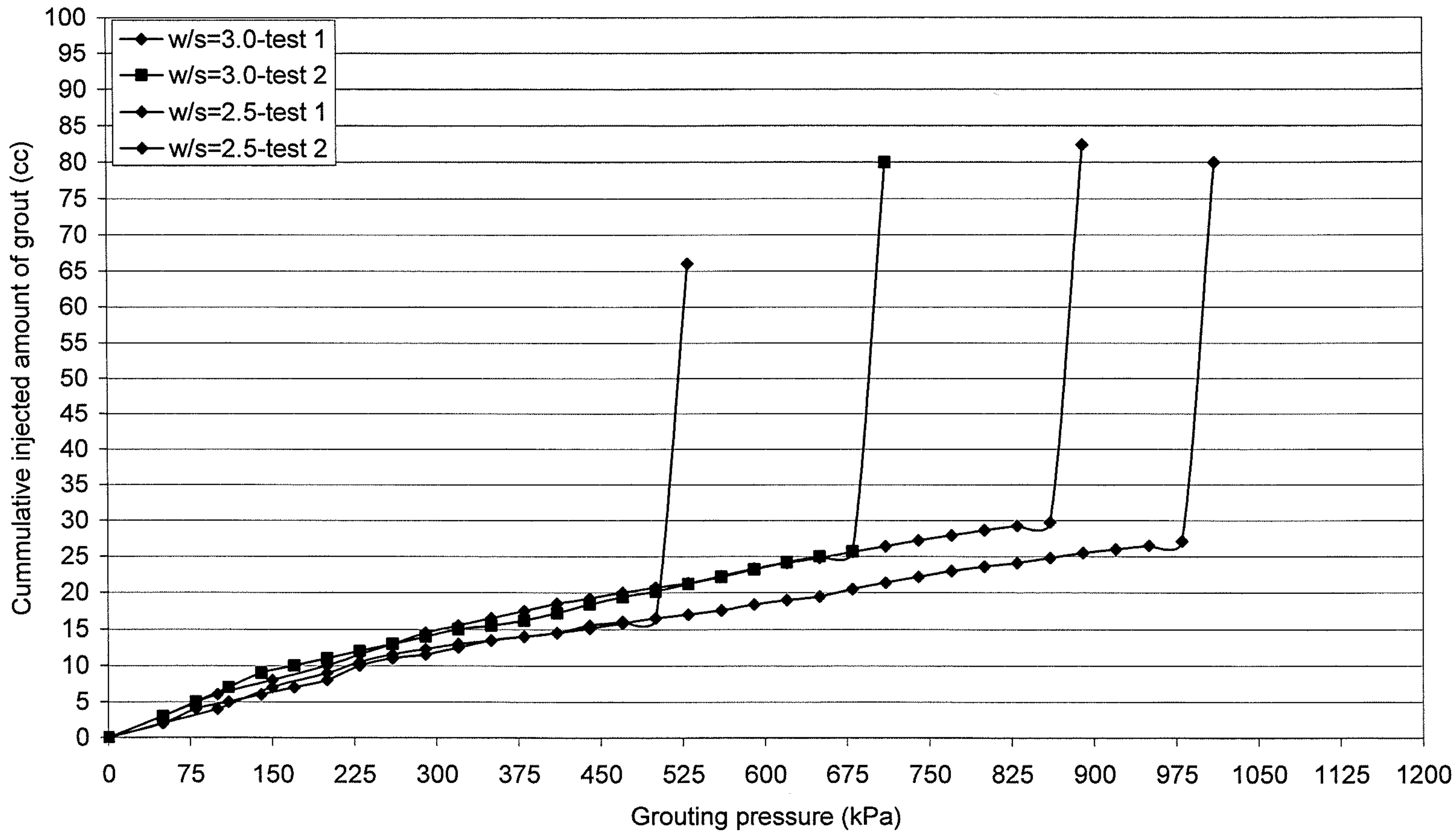


Figure 4.29 Grouting pressure - cummulative grout volume relationship; 32% fine; 60 % relative density

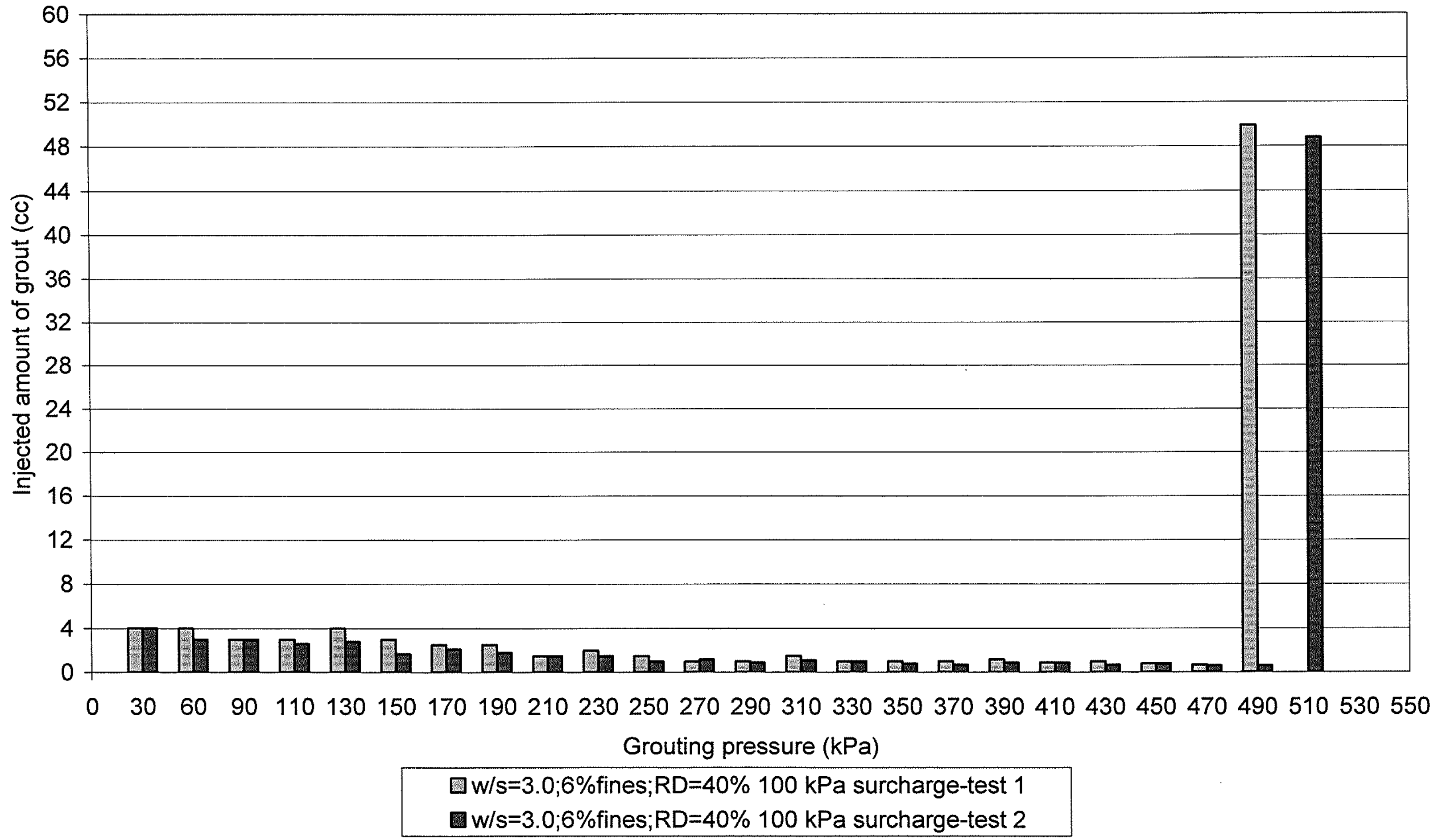


Figure 4.30 Grouting pressure versus injected grout volume; 6% fine, w/s=3.0; RD=40 %; 100 kPa surcharge

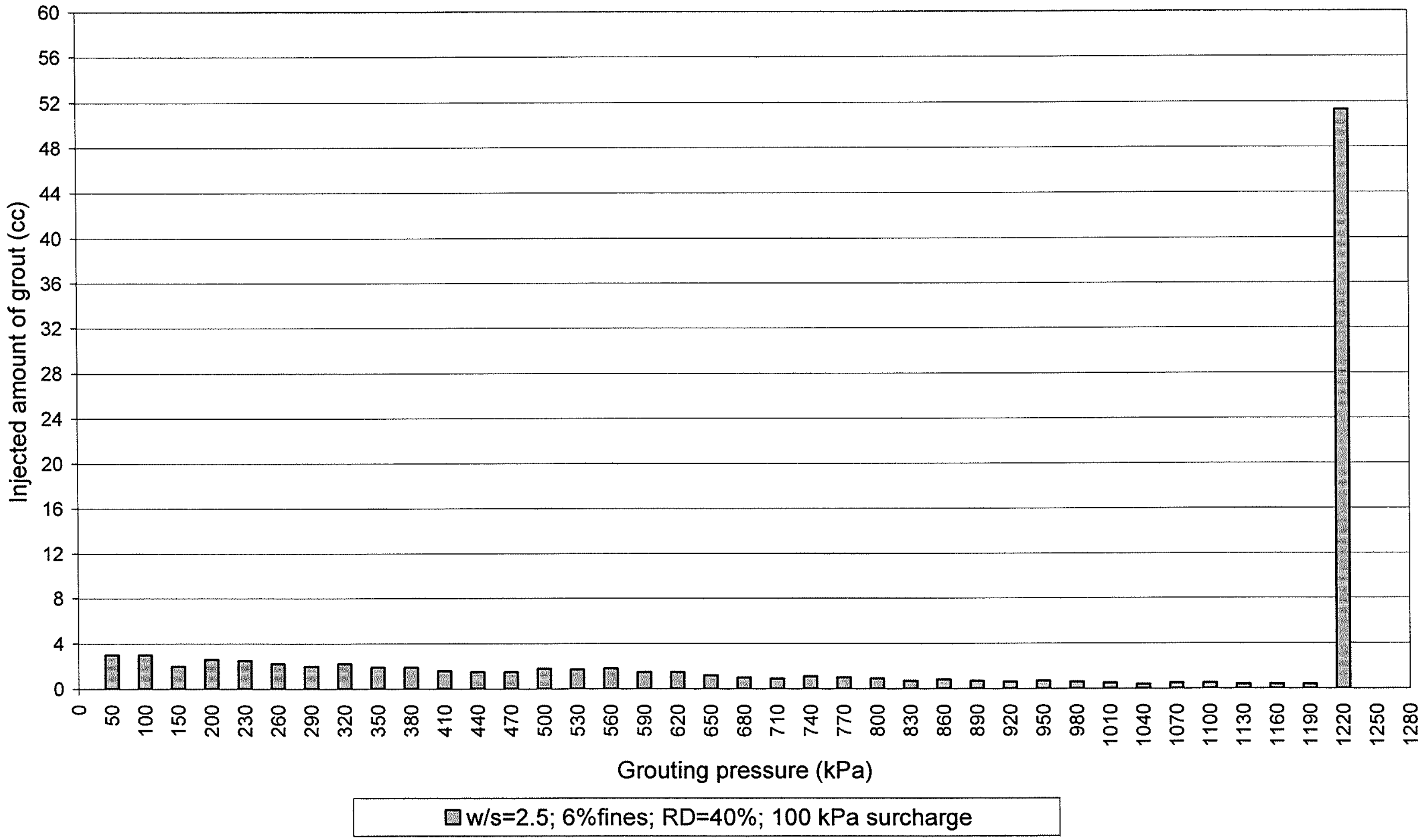


Figure 4.31 Grouting pressure versus injected grout volume; 6% fine, w/s=2.5; RD=40 %; 100 kPa surcharge

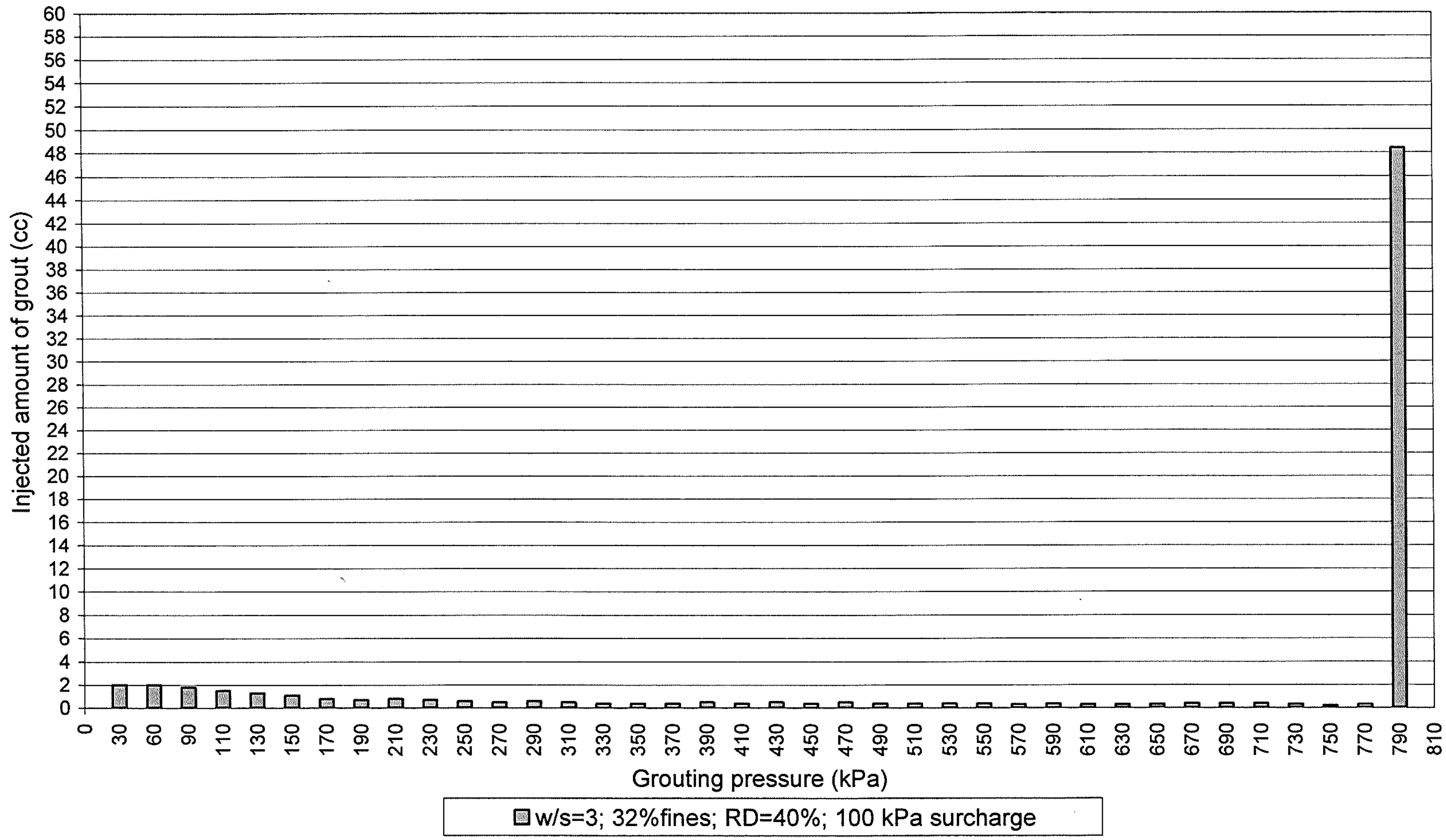


Figure 4.32 Grouting pressure versus injected grout volume; 32% fine, w/s=3.0; RD=40 %; 100 kPa surcharge

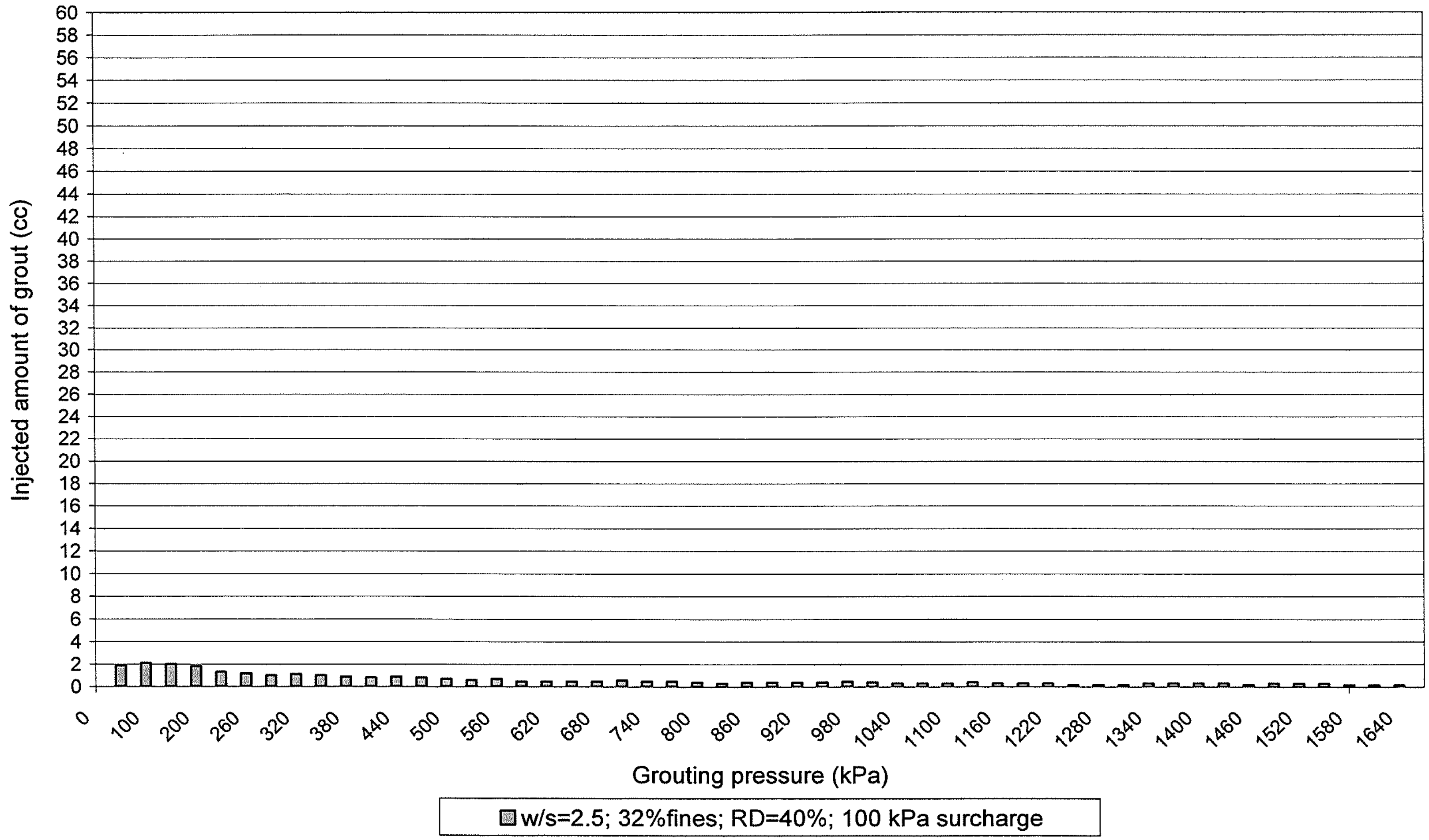


Figure 4.33 Grouting pressure versus injected grout volume; 32% fine, w/s=2.5; RD=40 %; 100 kPa surcharge

Graphical representation of grouting pressure versus cummulative volume of grout material injected into soil specimens of 6% and 32% fine content with a relative density of 40 % according to various water/solids ratios of grout can be seen in Figures 4.34 and 4.35 respectively. The applied vertical stress on top of the specimens in these tests was 100 kPa.

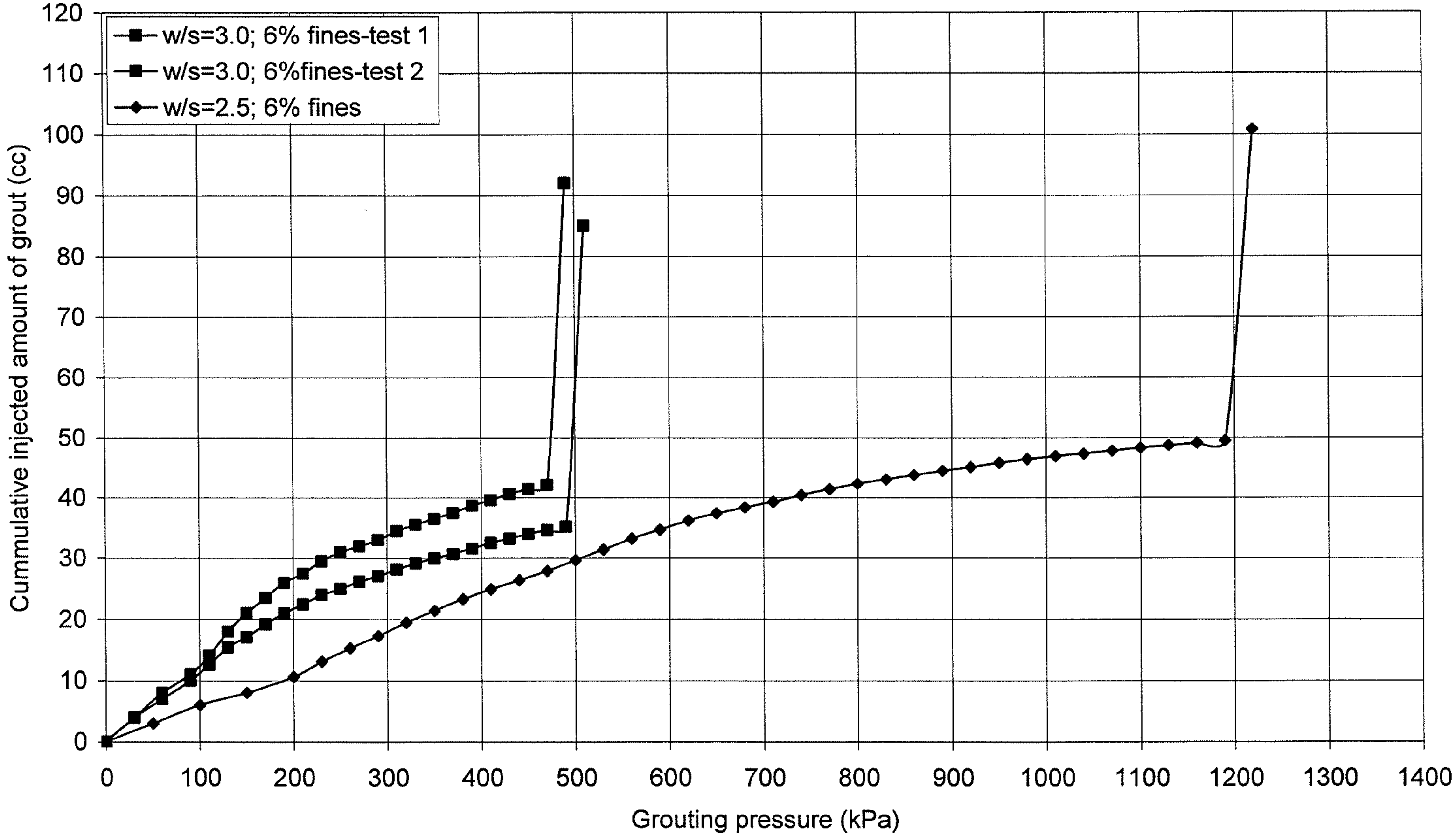


Figure 4.34 Grouting pressure - cummulative grout volume relationship; 6% fine; RD=40 %; 100 kPa surcharge

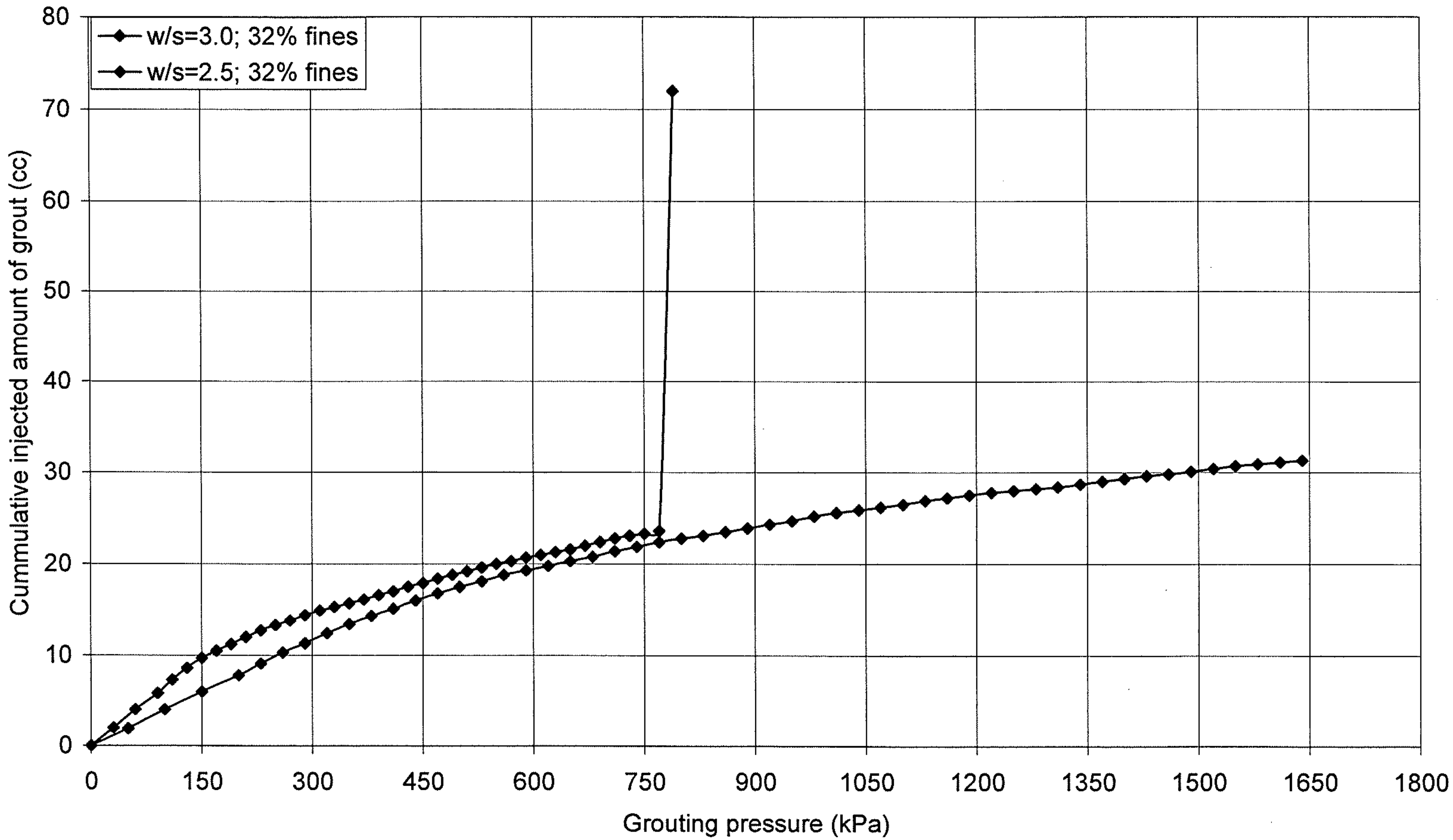


Figure 4.35 Grouting pressure - cummulative grout volume relationship; 32% fine; RD=40 %; 100 kPa surcharge

4.2 Discussions regarding the tests performed by micro fine cement grouts

Based on the tests performed by micro fine cement grouts the followings are discussed:

In all of the tests, grout material has been forced into the soil specimen by increasing the pressure gradually. Injected volumes of grout have been recorded for each pressure increment. When there was no change in the volume change measurement device, in other words, when the the grouting pressure became less than the soil resistance, the pressure was then increased. Main behavior in all of the tests is such that while the soil specimens are forced to be fractured by increasing the pressure of grout, some volume of grout is accumulated at the end of the injection pipe. This volume changed with respect to water/solids ratio of the grout and the fine content and relative density of the soil specimens. After the accumulation of this amount of grout and the tensile limit of the soil is overcome, a brittle fracturing and decrease in grouting pressure are observed.

In all of the tests injected volumes of grout at the initial stages of pressure increase are relatively higher. As the tensile limit of soil specimens are approached this injected volume becomes less. This behavior somehow resembles the preconditioning phase in compensation grouting applications. In practice the soil is conditioned at the initial stages of grouting by filling the local voids, cracks etc. and then further grouting creates fractures in the soil and the settled structures are restored. In the tests, grout volumes injected until fracturing compacts the loose soil. During this period soil sample is conditioned with almost no formation of heave at the top of the specimen, however, once fracturing occurs a certain amount of vertical heave is observed based on the volume of grout injected during fracturing.

The results of the tests performed with micro fine cement grouts show that fracturing pressure generally decreases with an increase in the water content of the grout (Figure 4.1 and 4.21). As the water/solids ratio increases the fracturing pressure decreases. Because as the water/solids ratio increases viscosity, in other words, resistance to flow, decreases and it becomes more easy to fracture the soil specimen. This behavior is observed in all of the soil specimens regardless of fine content.

It is also observed that fracturing pressure generally increases as the fine content of the soil increases. For example for water/solids ratio of 2.0, fracturing pressures of 450 kPa and 390 kPa are required in soil specimens with 6% fines, wheras fracturing pressures of 620 kPa and 580 kPa are required in soil specimens with 32% fines. However, this increase in fracturing pressure is not so significant for higher water/solids ratios. Especially for water/solids ratio of 3.0, almost the same fracturing pressures are required in all of the soil specimens. In other words, effect of fine content on the fracturing pressure becomes more clear as the viscosity of the grout incerases (water/solids ratio decreases).

It is seen that relative density of the soil affects the fracturing pressure tremendously. For example for water/solids ratio of 2.0, fracturing pressures of 1490 kPa and greater than 1640 kPa are required in soil specimens of 6% fine with a 60 % of relative density. However, the corresponding pressures for the soil specimen with a 40% of relative density are only 450 kPa and 390 kPa. This suggests that the effect of relative density on the fracturing pressure is much more influencial than that of fine content.

It was observed that an increase in overburden stress applied on top of the specimens increase the fracturing pressures. This behavior indicates requirement of higher grouting pressures in deeper soil layers. This effect becomes more influencial as the viscosity of the grout mass increases.

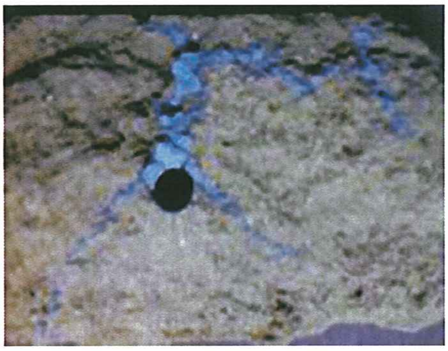
When the volumes of grout injected into soil specimens until fracturing are examined, an increasing tendency is observed as the water/solids ratio increase. For example, in grouting soil specimens of 6% fine with a relative denisty of 40%, 100.5 cc and 74.5 cc volumes of grout were injected when the water/solids ratio is 1.5. However, when the ratio is 3.0, injected volumes of grout until fracturing are 76 cc and 60 cc. This behavior is also observed in other soil specimens. For example, in grouting soil specimens with 32% fines with a relative denisty of 40%, 60.5 cc and 66 cc volumes of grout were injected when the water/solids ratio is 2.0, whereas injected volumes of grout until fracturing are 43 cc and 30 cc in tests performed with water/solids ratio of 3.0. This result shows the difference between the compaction grouting and fracture grouting mechanisms. In compaction grouting, improvement is due to densification and the success is dependent on the accumulation of the grout around the injection point without fracturing the soil. However, in fracture grouting improvement is due to reinforcement and the soil is intentionally fractured by high mobility fluid like grouts. Therefore, when the grout gets stiffer, the mechanism turns into the compaction grouting and tends to accumulate at the tip of the injection pipe. When the grout becomes more mobile (grout of higher water/solids ratio) then it is more easy and possible to fracture the soil. Then the accumulated volume becomes less than that performed with a stiffer grout.

When the volumes of grout injected into soil specimens until fracturing are examined in view of fine content of the soil, it is observed that the volumes are higher for soil specimens of 6% fine content. Indeed, when the injected volumes until fracturing for soil specimens of 6% and 24% are compared, volumes become less as the fine content increases to 24%. However, as the fine content is further increased to 32%, this decrease in injected volume until fracturing is not so significant (See Figures 4.15 to 4.17). However, they are generally less than those injected in soil specimens of 24 % fine content. This behavior shows the effect of fine content on the penetration of grouts into the soil specimens. In soil specimens of 6% fine content, maximum volumes are injected until fracturing. It is probable that some amount of permeation grouting occurs at the initial stages of pressure increase.

When the volumes of grout injected into soil specimens until fracturing are examined, in view of relative density of the soil, injected volumes until fracturing becomes signicantly less as the relative density increases to 60%. Actually, effect of relative density on the volume of grout injected until fracturing is more predominant than that of fine content.

In the tests performed on soil specimens with a relative density of 40% no vertical heave is observed until fracturing. However, when the relative density of the soil specimens are increased to 60% some amount of heave is observed as the volume of grout until fracturing is injected with increasing grouting pressure. This may be due to the fact that soil specimens with a relative density of 60% are already dense and reflect the volume of grout injected until fracturing as a vertical heave at the top. In case of soils with a relative density of 40 %, initial injections provide the preconditioning phase.

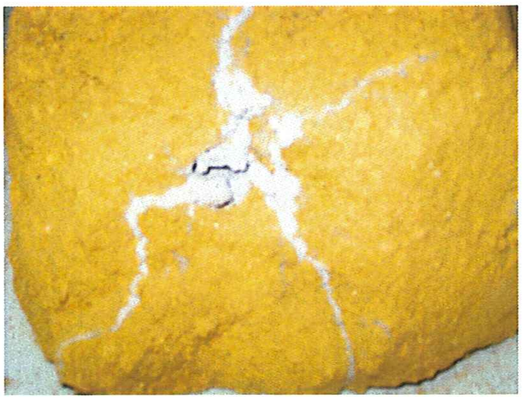
Grouted soil specimens were exposed after the tests and the fracture patterns were observed. It was seen that grout accumulation around the injection pipe was higher with stiffer grouts and the fractures were thicker. On the other hand, injection with less viscous grouts produced a brittle failure in the specimen with the accumulated volumes being smaller and the fractures being thinner (See Figure 4.36).



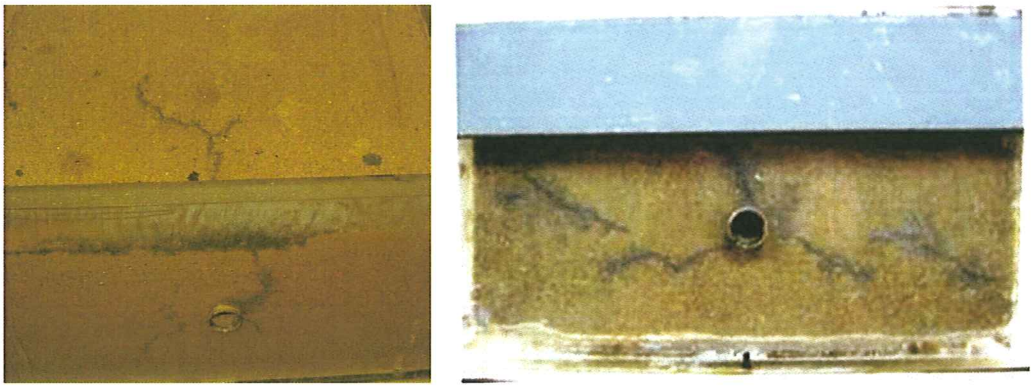
a) Water/solids = 2.0; 6% fine

b) Water/solids = 1.5; 6 % fine

c) Water/solids = 2.0; 32% fine



d) Water/solids = 2.5; 32 % fine



e) Water/solids = 3.0; 32 % fine f) Water/solids = 3.0; 6 % fine

Figure 4.36 Thicker and relatively thinner fractures were observed based on the viscosity of the grout for soil specimens at 40 % relative density

4.3 Results of the tests performed by ordinary Portland cement grouts

Ordinary Portland cement grouts of different water/solid ratios (0.7, 1.0, 1.5 and 2.0) were tried in the tests. Effects of grout composition and fine content and relative density of the soil on the fracturing pressure, exposed grout mass, pressure–volume intake relationship were investigated.

Grouting tests performed in pure Yumurtalık sand has shown that it is diffucult to fracture the fine silty sandy soils with an ordinary Portland cement. Because high percentages of water filtration occured and the grout lost its water under pressure during grouting operation. This in turn let the solid cement particles accumulate inside the injection pipe and the grouting pressures started to increase significantly. At some point further injection became impossible. At the end of these tests (tests performed in Yumurtalık sand) the grouted soil has been exposed and it has been observed that a very small amount of grout has been accumulated at the tip of the injection pipe without fracturing the soil (Figure 4.37). Same behavior has been observed in all of the grouting tests performed in pure Yumurtalık sand with an ordinary Portland cement grout (water/solids ratios varying from 0.70 to 2.0).

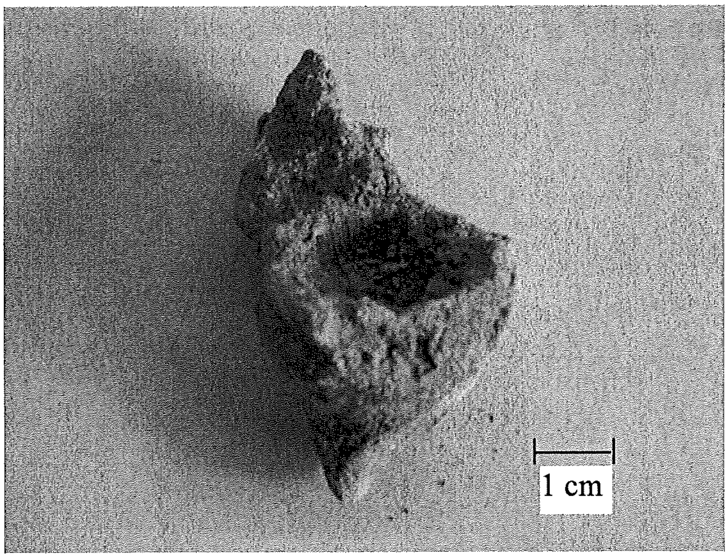


Figure 4.37 A very small volume of grout mass exposed at the end of the grouting tests in pure Yumurtalık sand specimens

Based on the results of these preliminary tests performed with Yumurtalık sand and in order to eliminate water filtration problem, it has been determined to add kaolinite into the test soil at a certain percentage as a replacement for Yumurtalık sand. Therefore, mixtures of Yumurtalık sand and kaolinite have been prepared by adding 8 %, 15 % and 25 % of kaolinite by weight as a replacement for the sand. Grouting tests performed in these mixtures yielded less filtration percentages as compared to those performed in Yumurtalık sand depending on their fine content.

Table 4.5 shows the results of the tests performed in Yumurtalık sand – kaolinite mixtures with ordinary Portland cement grouts. In these tests soil specimens were placed at lower densities.

Table 4.5 Results of grouting tests performed in specimens of Yumurtalık sand – kaolinite mixtures (placed at lower densities)

25% kaolinite	Water/solid	0.7	0.7	1.0	1.0	1.5	1.5	2.0	2.0
	Maximum observed grout pressure (bars)	40	-	20	25	14	13	12	12
	Volume injected (cc)	543		597	489	615	724	706	724
	Volume measured (cc)	458		480	410	485	590	510	535
	Filtration percentage (%)	15.6		19.6	16.1	21.2	18.5	27.7	26
	Dry density of the exposed mass (g/cm3)	2.02		1.86	1.82	2.03	1.94	2.09	2.04
15% kaolinite	Water/solid	0.7	1.0	1.0	1.5	1.5	1.5	2.0	2.0
	Maximum observed grout pressure (bars)	45	30	26	18	19	20	14	13
	Volume injected (cc)	543	651	651	670	688	651	724	724
	Volume measured (cc)	445	515	495	510	470	465	480	445
	Filtration percentage (%)	18	20.9	24	23.8	31.6	28.6	33.7	38.5
	Dry density of the exposed mass (g/cm3)	1.99	1.91	1.89	1.95	1.97	2.01	2.02	2.08
8% kaolinite	Water/solid	1.0	1.0	1.5	1.5	2.0			
	Maximum observed grout pressure (bars)	39	24	37	27	28	No other tests performed		
	Volume injected (cc)	651	434	670	651	597			
	Volume measured (cc)	345	225	375	395	325			
	Filtration percentage (%)	47	48.1	44	39.4	45.6			
	Dry density of the exposed mass (g/cm3)	2.04	2.11	2.07	2.05	2.09			

Figures 4.38 to 4.40 show the relationships between the injected volume and the corresponding pressure build up for the addition of 8 %, 15 % and 25 % of kaolinite to the Yumurtalık sand, respectively. The soil specimens were placed at lower densities for these tests. It should be noted that any drop in grout pressure means fracturing the soil specimen..

Figure 4.41 shows the relationship between the water/solids ratio of the grout material and the fracturing pressure (maximum observed pressure) for all of the specimens placed at lower densities.

Figure 4.42 presents the relationship between the filtration percentage and the water/solids ratio of the ordinary Portland cement grout when the soil specimens are placed at lower densities. Filtration percentage is the volume of water separated from the grout during injection divided by the injected volume of grout. The amount of filtrated water was obtained by measuring the volume of the exposed grout mass and by taking the difference with the injected volume.

Figure 4.43 indicates the relationship between the dry densities of the exposed grout masses and the corresponding filtration percentages for all of the Yumurtalık sand and kaolinite mixtures which are placed at lower densities.

Similar to the relationship between the dry densities of the exposed grout masses and filtration percentages, Figure 4.44 shows the relationship between the dry densities of the exposed grout masses and the water/solids ratio of the grout material. In these tests soil specimens are placed at lower densities.

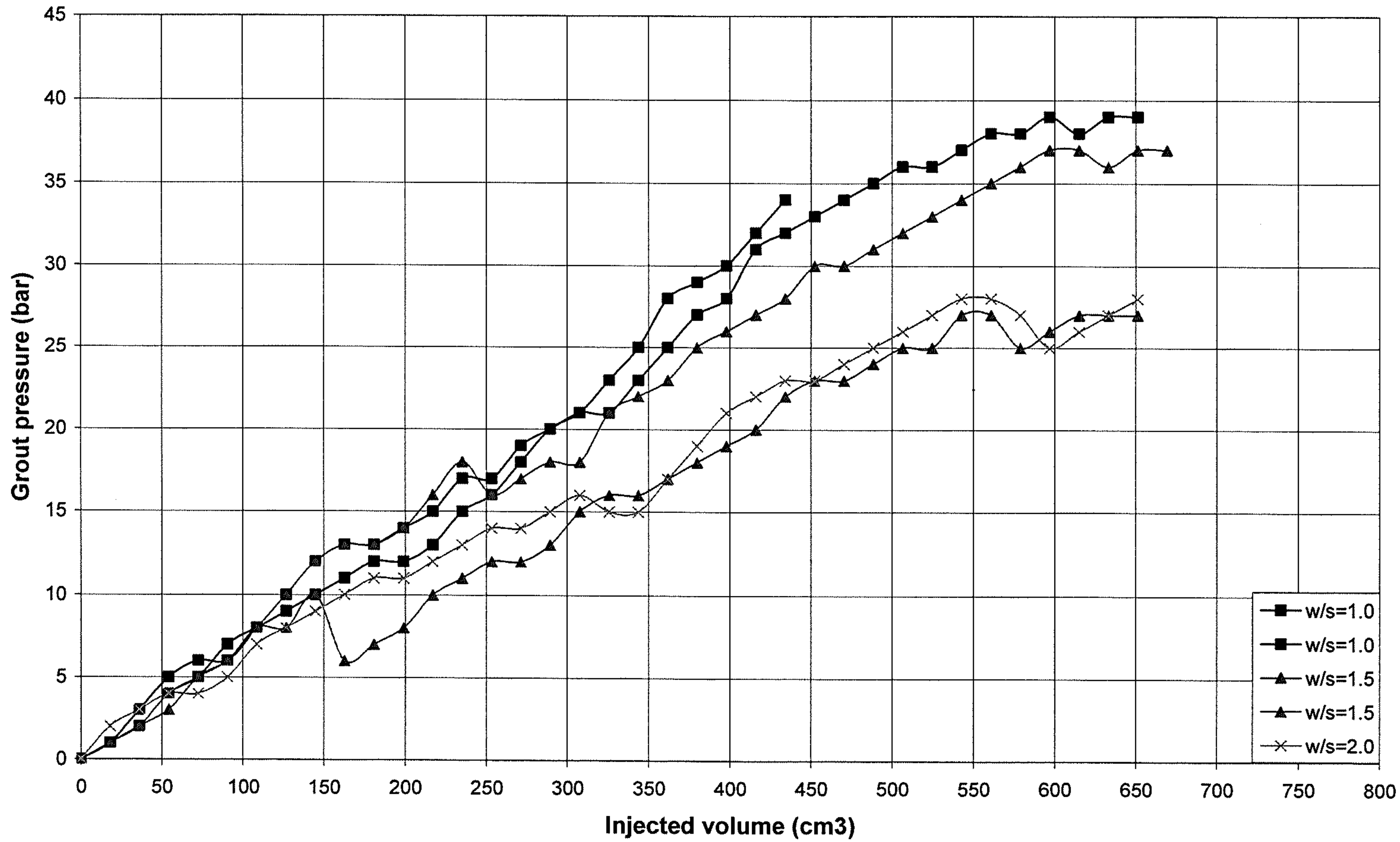


Figure 4.38 Injected volume – grout pressure build up relationship for addition of 8 % of kaolinite (placement at lower density)

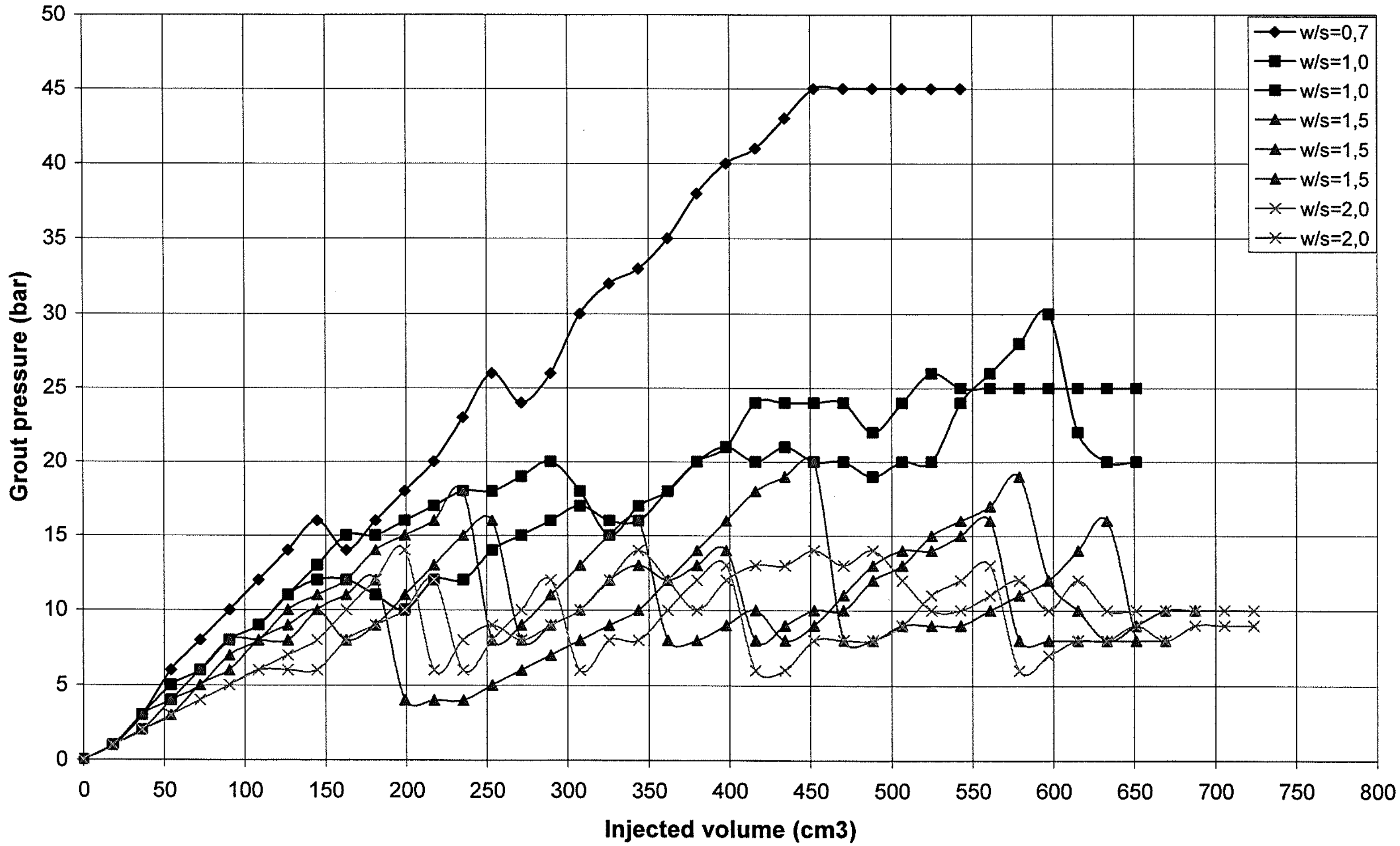


Figure 4.39 Injected volume – grout pressure build up relationship for addition of 15 % of kaolinite (placement at lower density)

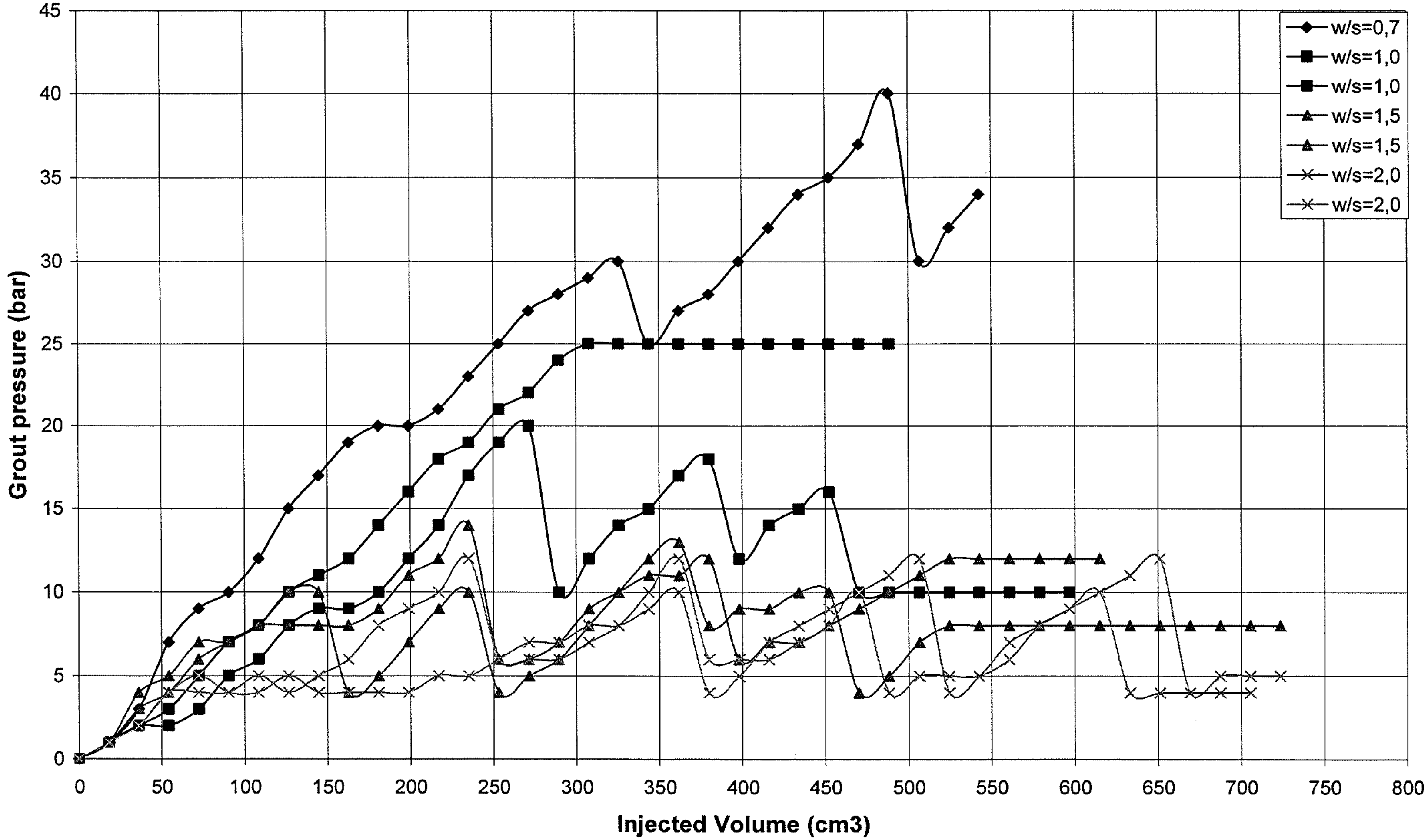


Figure 4.40 Injected volume – grout pressure build up relationship for addition of 25 % of kaolinite (placement at lower density)

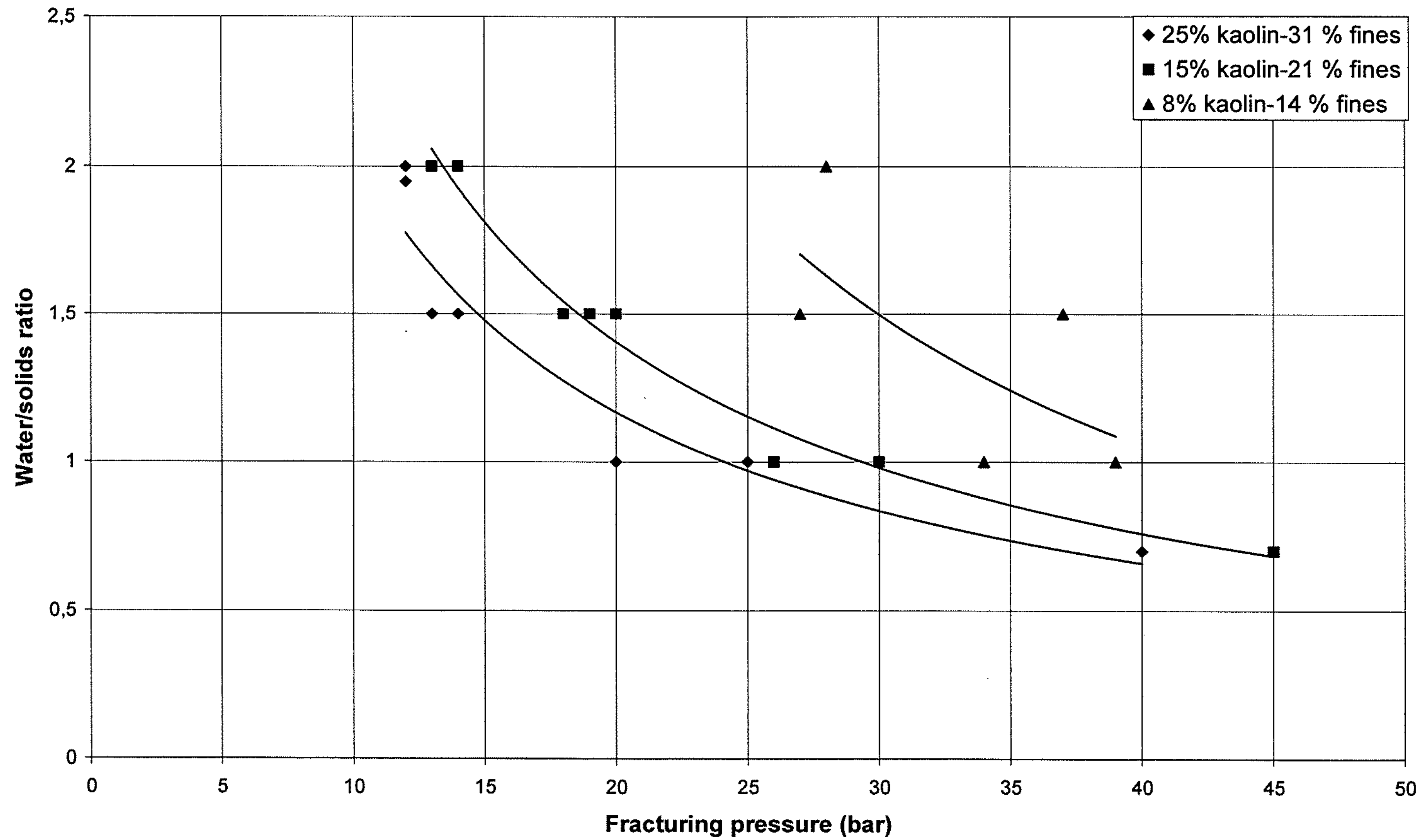


Figure 4.41 Relationship between the water/solids ratio of the grout material and the fracturing pressure (maximum observed pressure) for soil specimens placed at lower densities

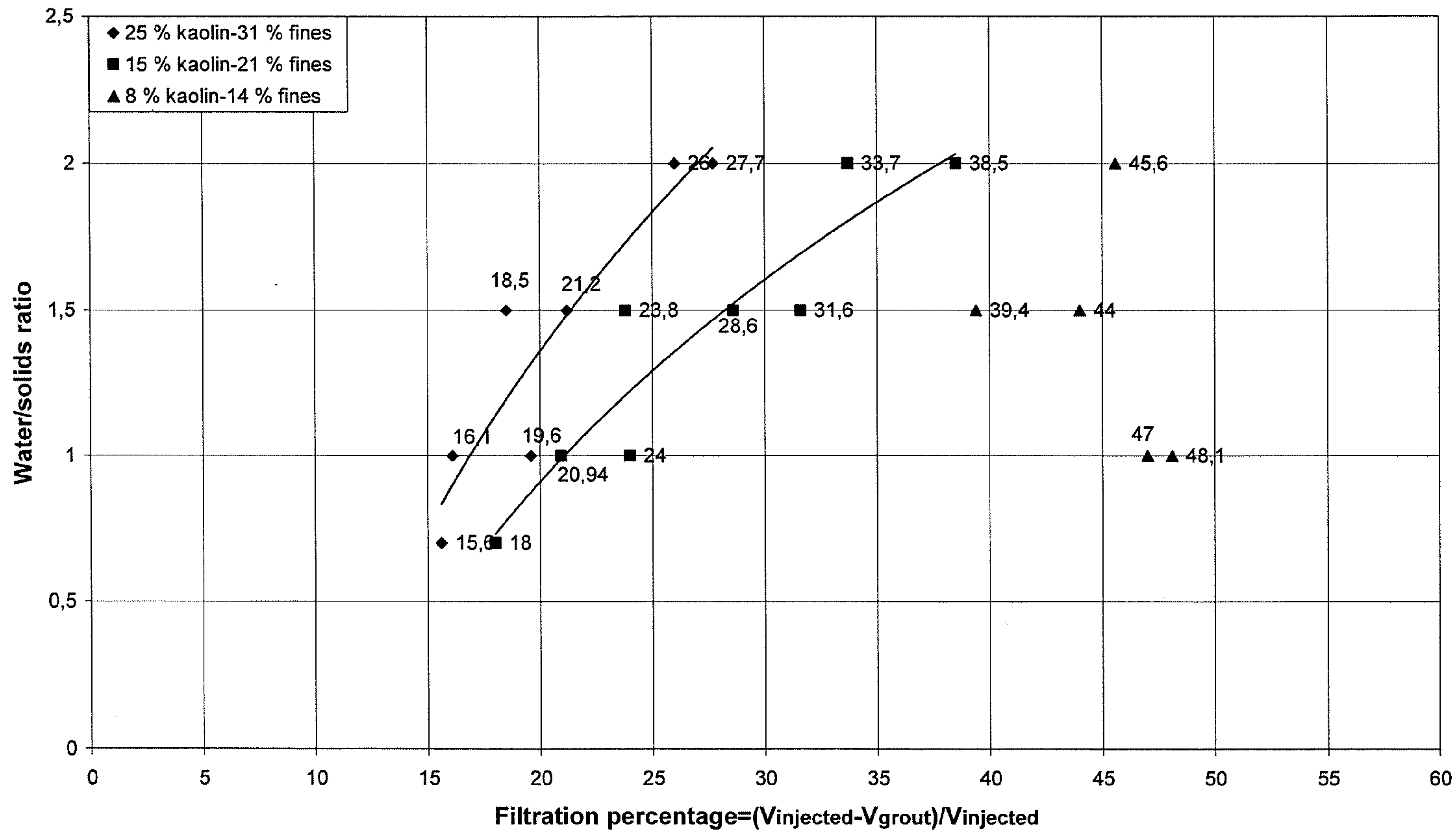


Figure 4.42 Relationship between the filtration percentage and the water/solids ratio of the grout with soil specimens being placed at lower densities

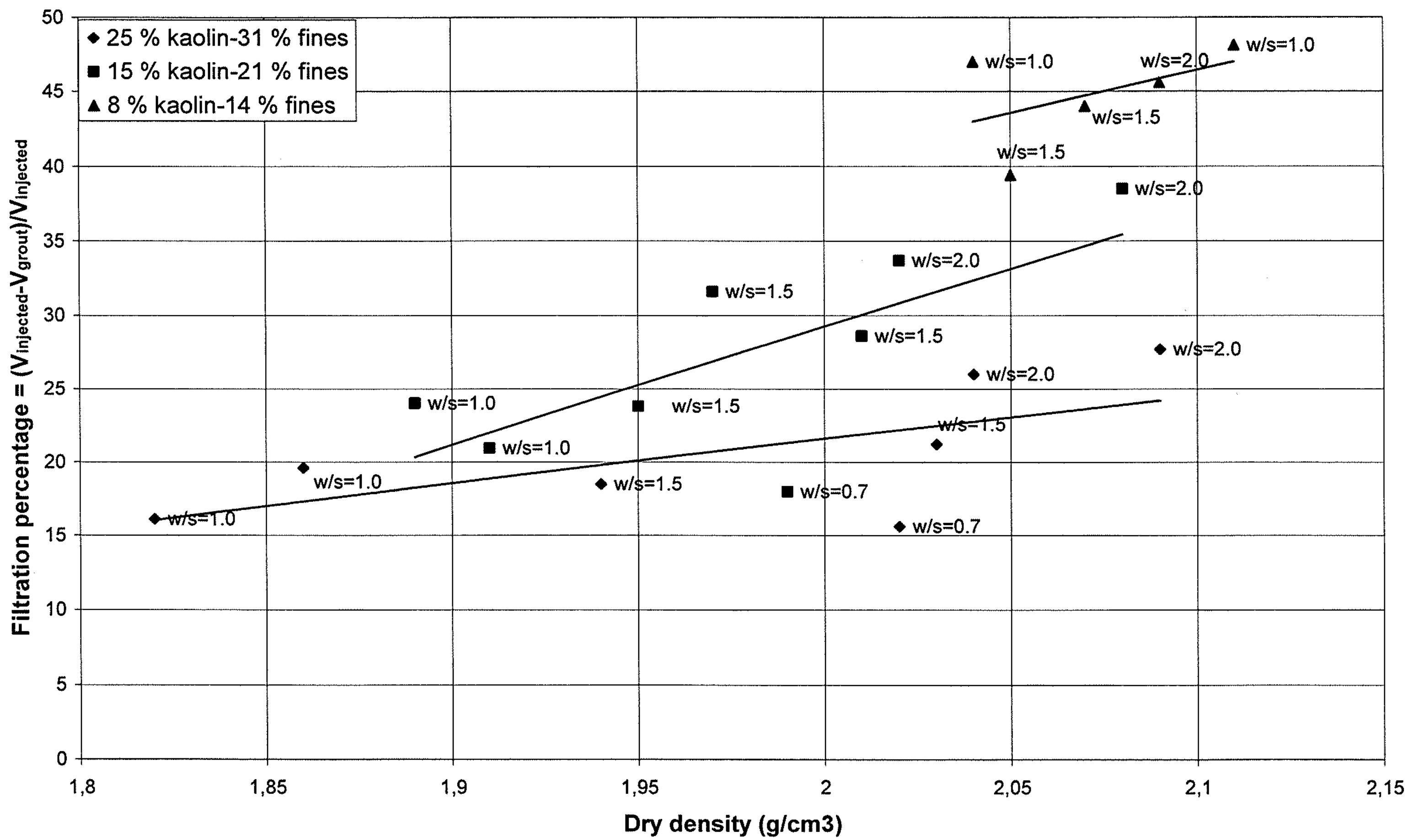


Figure 4.43 Relationship between the dry densities of the exposed grout masses and the corresponding filtration percentages for the soil specimens placed at lower densities

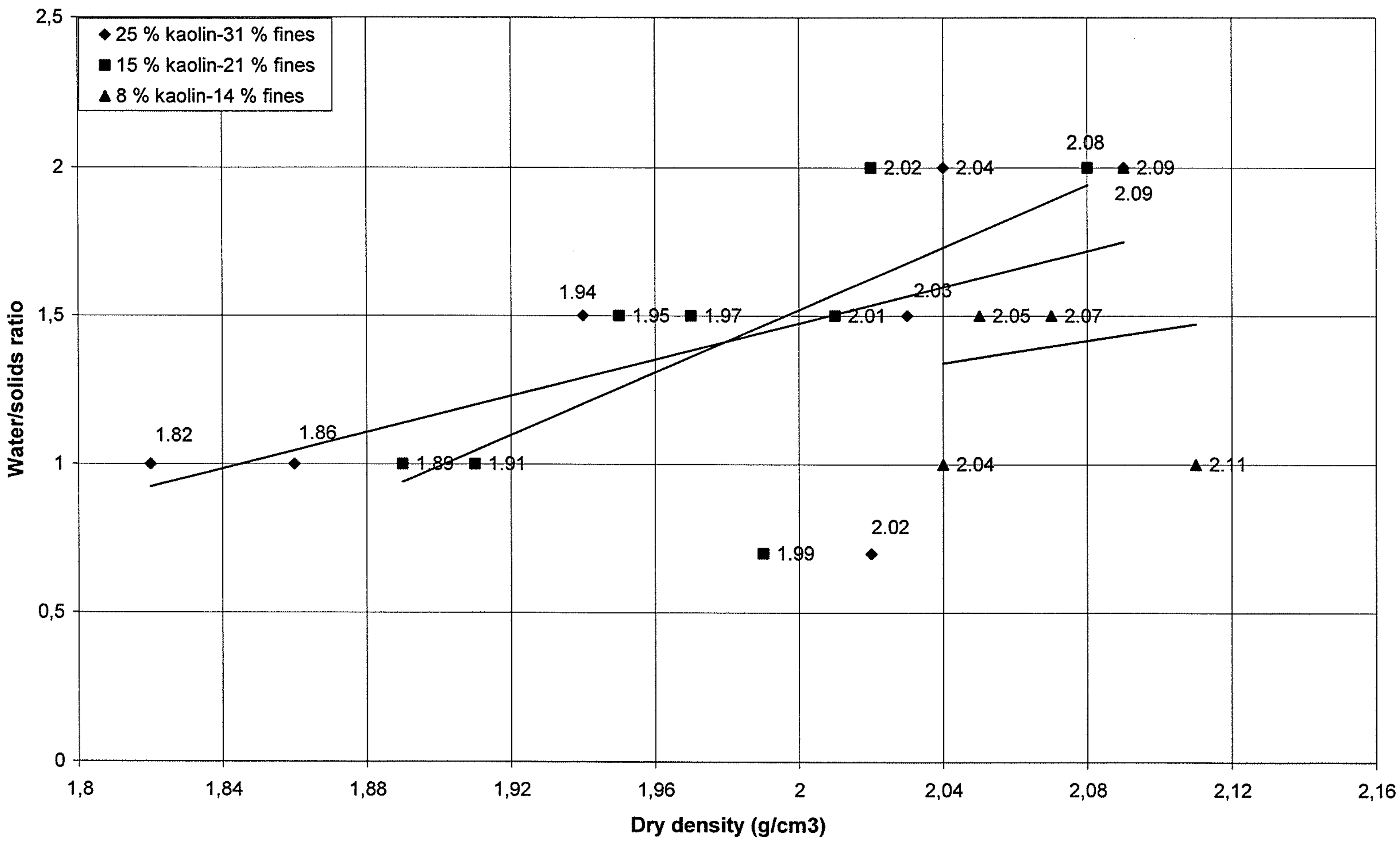


Figure 4.44 Relationship between the dry densities of the exposed grout masses and the water/solids ratio of the grout material for the soil specimens placed at lower densities

In order to see the effect of relative density on the grouting test results some of the tests were repeated by placing the soil specimens at higher densities. A higher compaction effort has been utilized for the specimens to have the required higher densities (See Figure 3.9). Grout material used in these tests has a water/solids ratio of 1.5. The tests were performed for all of the Yumurtalık sand and kaolinite mixtures. Table 4.6 presents the results of these tests.

Table 4.6 Results of grouting tests performed in specimens of Yumurtalık sand – kaolinite mixtures (placed at higher densities)

Soil specimen	25 % kaolinite Water/solids=1.5		15 % kaolinite Water/solids=1.5		8 % kaolinite Water/solids=1.5	
Maximum observed grout pressure (bars)	16	21	24	28	39	44
Volume injected (cc)	670	670	670	670	670	670
Volume measured (cc)	515	495	445	425	340	325
Filtration percentage (%)	23.1	26.1	33.6	36.6	49.3	51.5
Dry density of the exposed mass (g/cm3)	2.00	1.98	2.04	2.02	2.11	2.14

Figures 4.45 shows the relationships between the injected volume and the pressure build up when the specimens were placed at higher densities.

Figure 4.46 compares the fracturing pressures (maximum observed pressure) for soil specimens placed at lower and higher densities.

Figure 4.47 presents the effect of relative density of the soil specimens on the filtration percentages.

Figure 4.48 shows the dry densities of the exposed grout masses for all of the soil specimens which are placed both at lower and higher densities.

Figure 4.49 shows the relationship between the dry densities of the exposed grout masses and the water/solids ratio of the grout material when the soil specimens are placed at lower and higher densities.

Figure 4.50 presents the recorded amounts of vertical heave at the top of the specimen during the injection process in soil specimens of higher density. In soils of lower relative density almost no appreciable amount of heave was recorded.

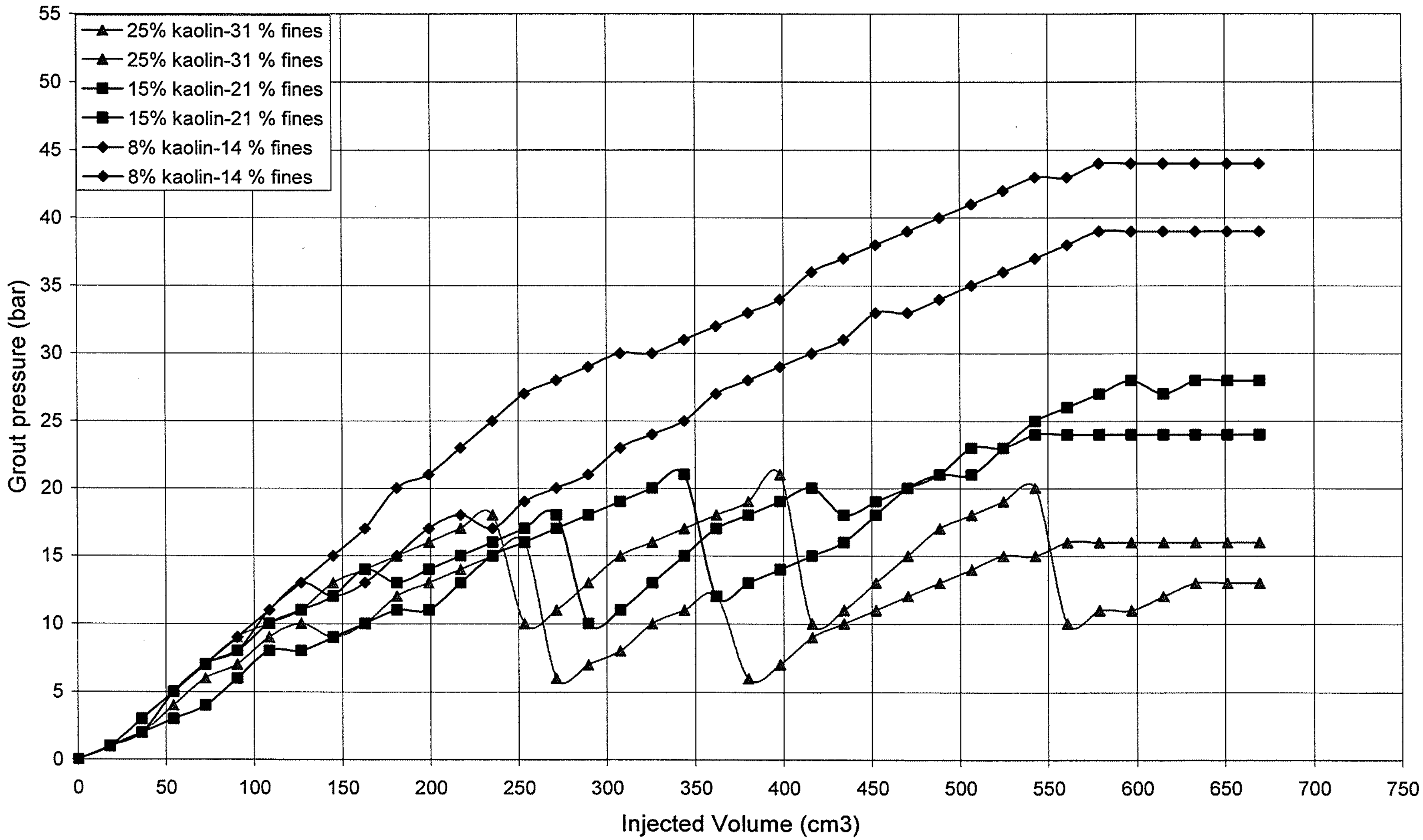


Figure 4.45 Injected volume – grout pressure build up relationship for all Yumurtalık sand – kaolinite mixtures (placement at higher density)

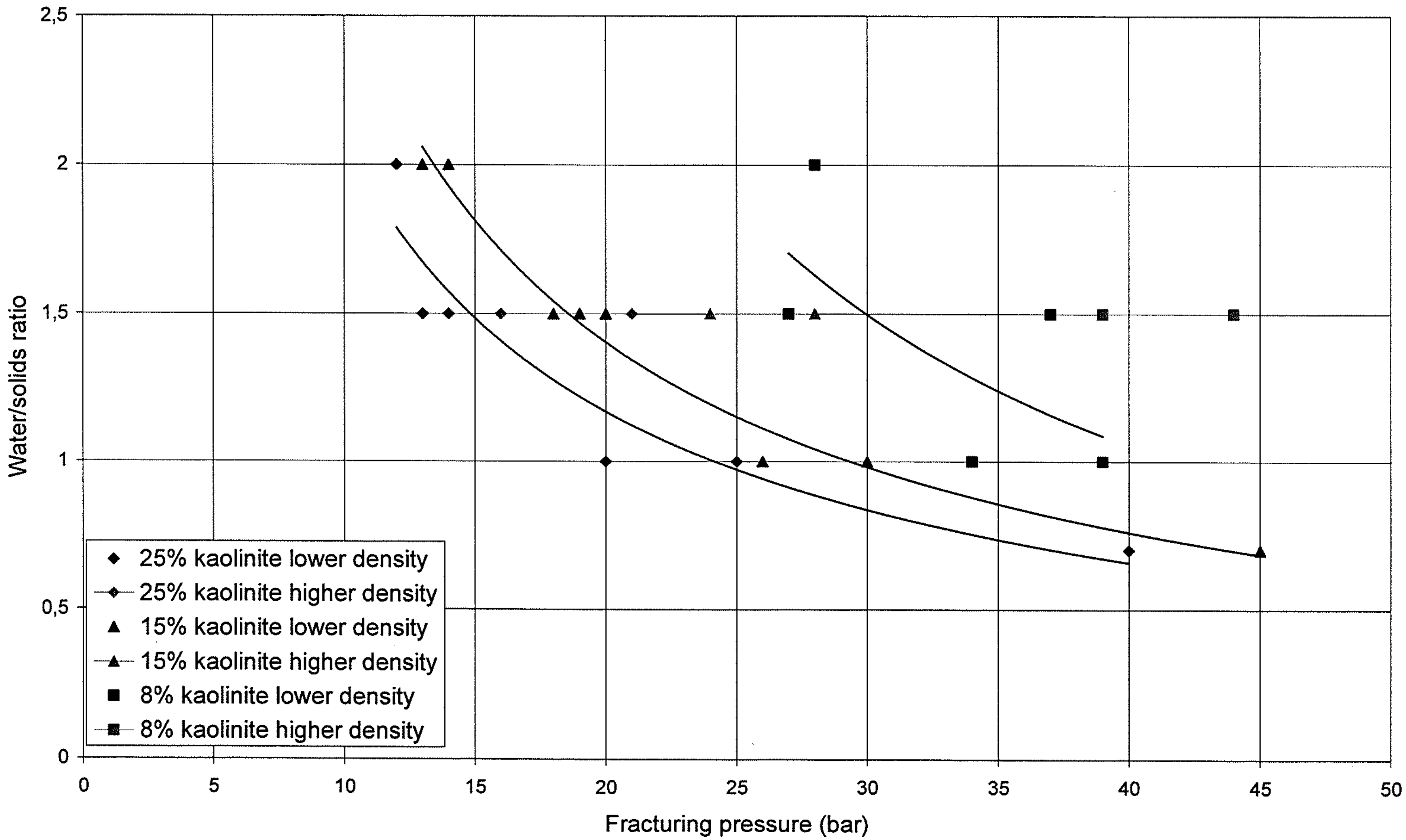


Figure 4.46 Comparison of fracturing pressures (maximum observed pressure) for soil specimens placed at lower and higher densities

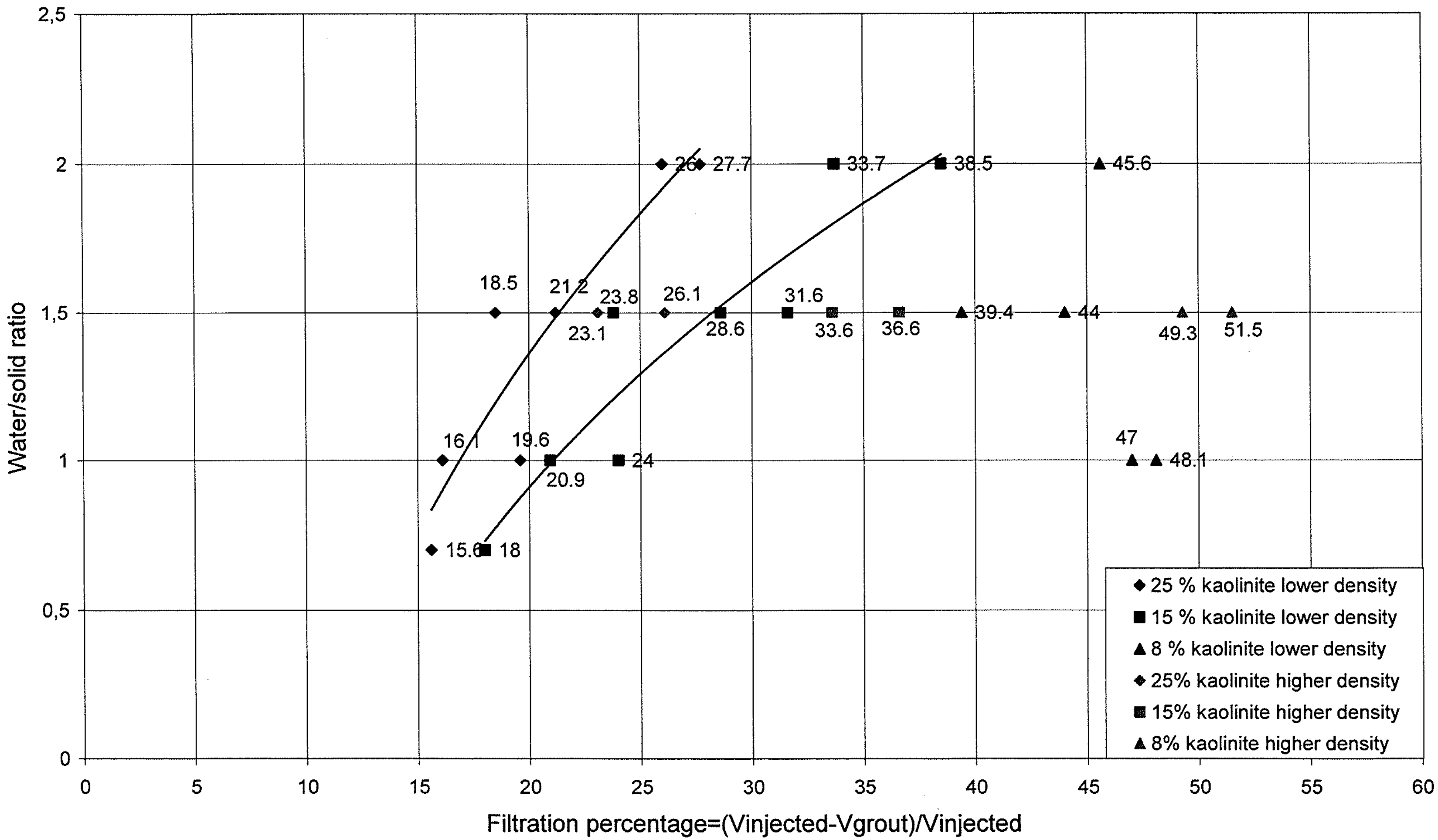


Figure 4.47 Effect of higher relative density of the soil specimens on the filtration percentages

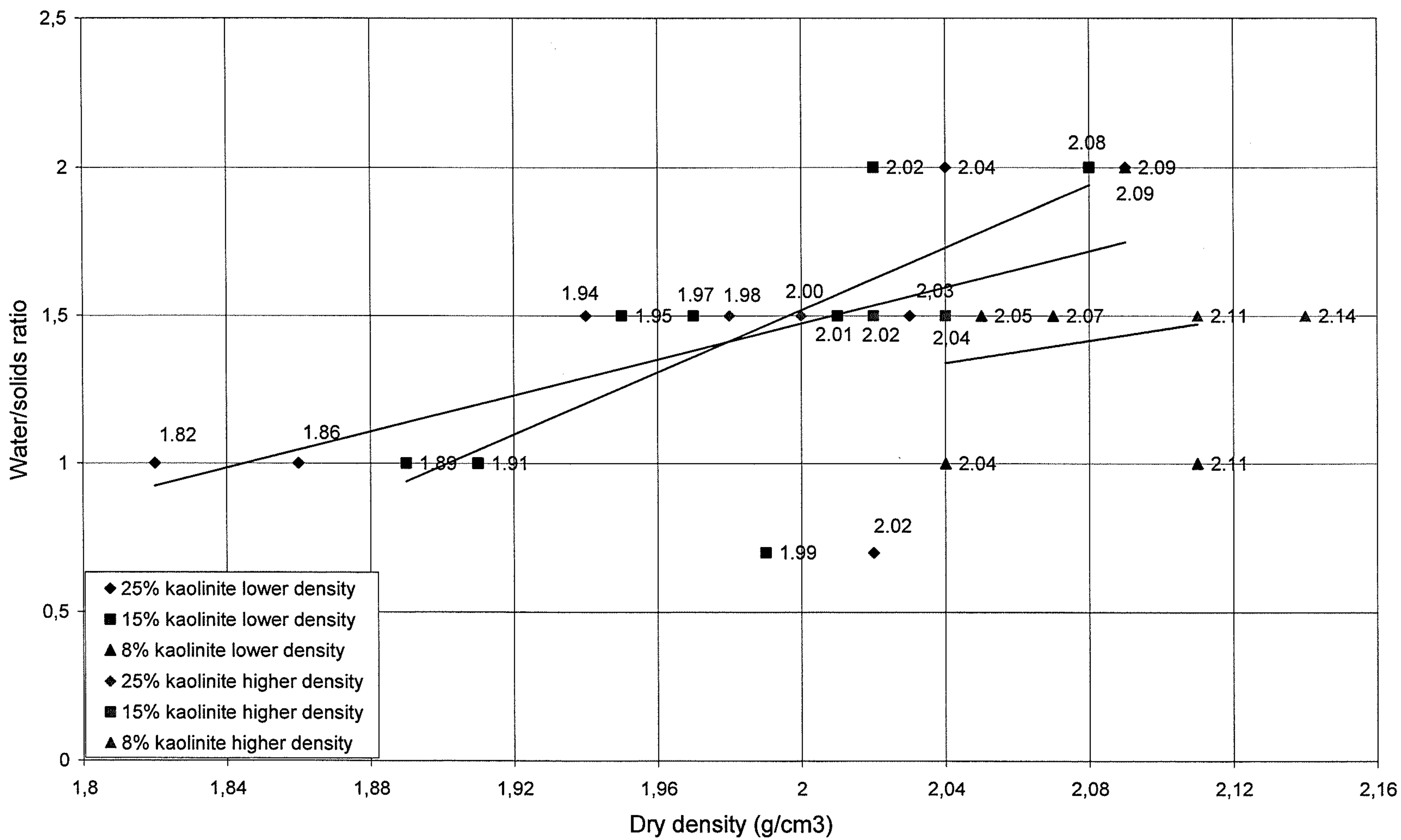


Figure 4.48 Effect of the relative density of the soil specimens on the dry densities of the exposed grout masses

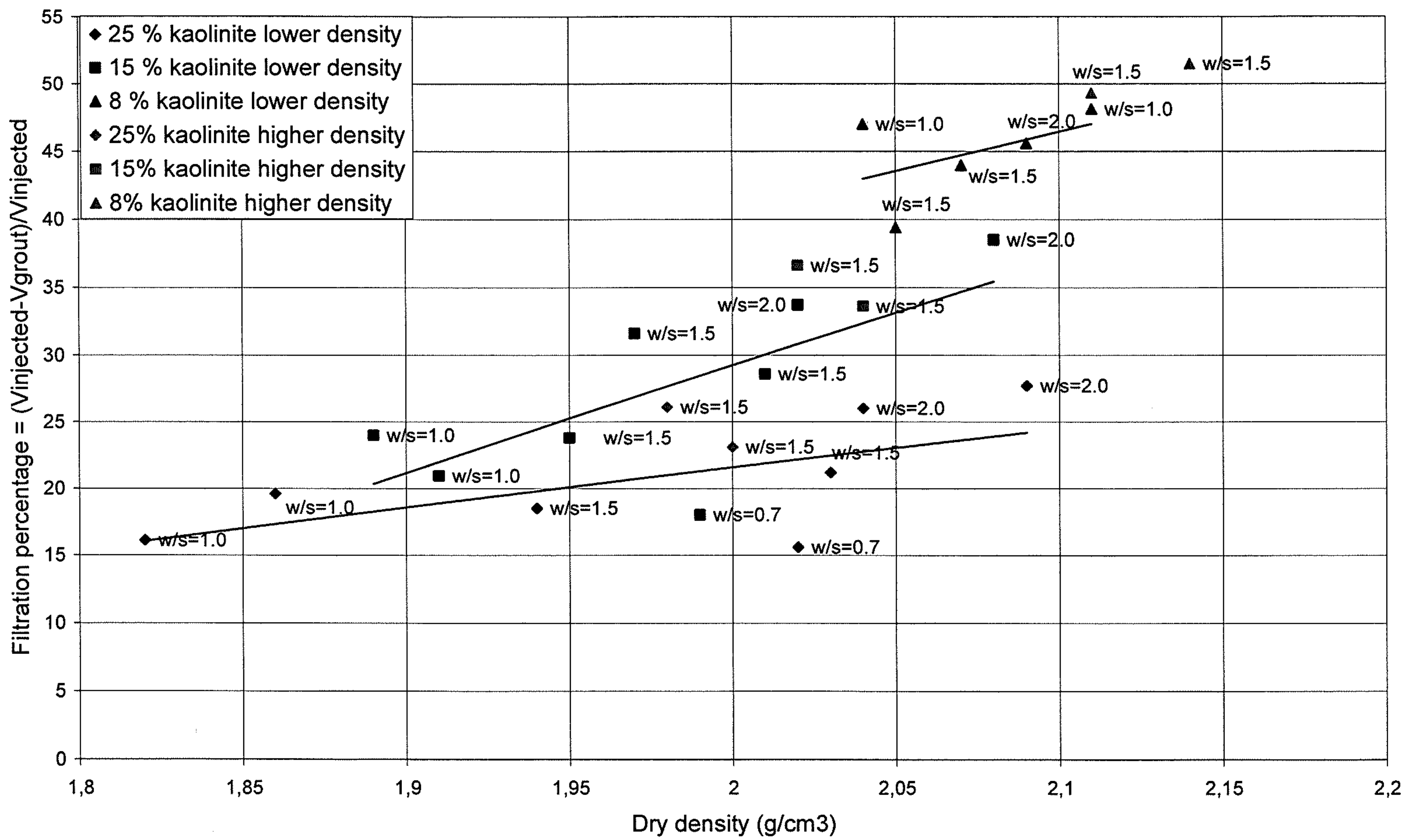


Figure 4.49 Relationship between the dry densities of the exposed grout masses and the water/solids ratio of the grout material when the soil specimens are placed at lower and higher densities

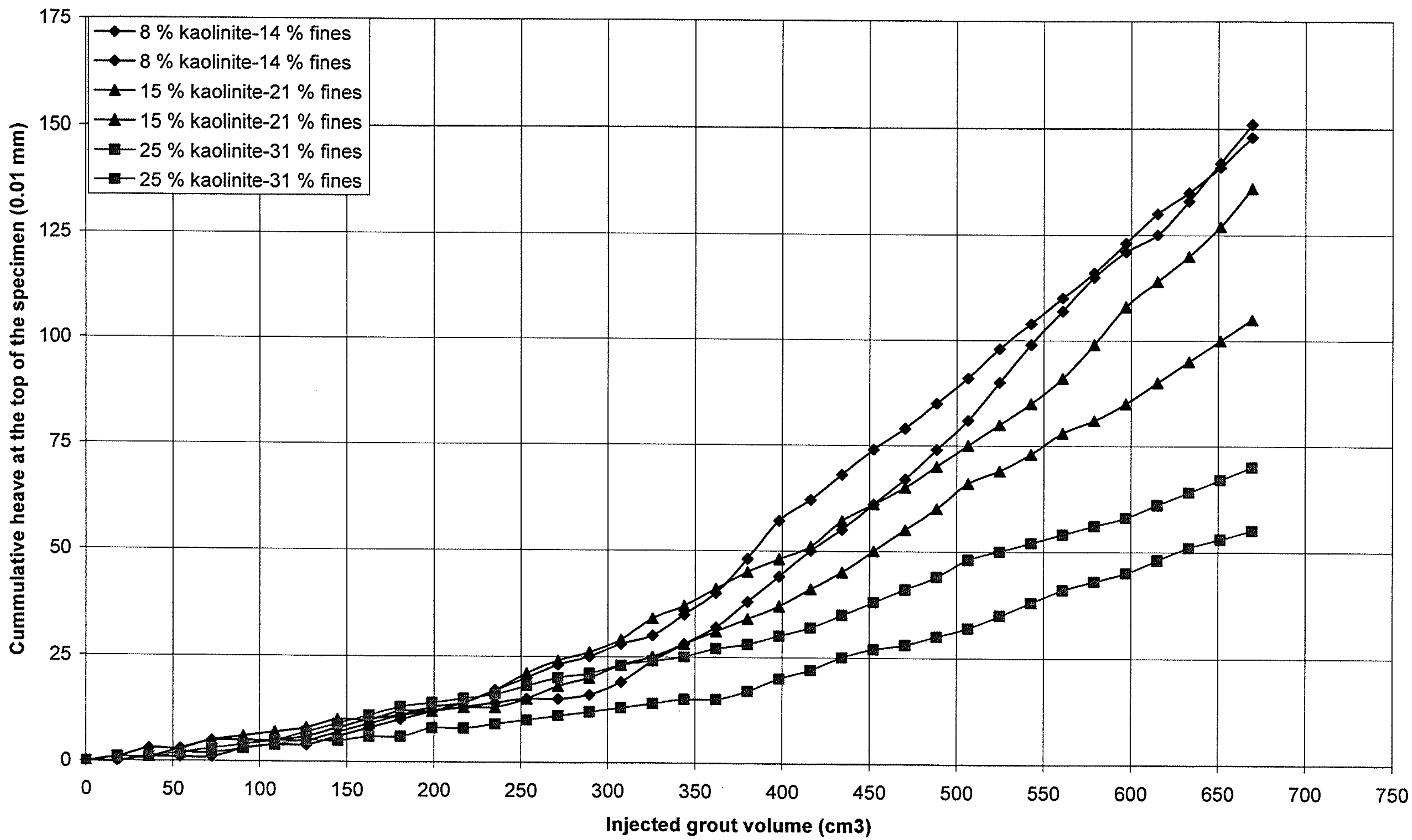


Figure 4.50 Recorded amounts of vertical heave at the top of the specimen during the injection process in soil specimens of higher density

4.4 Discussions regarding the tests performed by ordinary Portland cement grouts

Based on the tests performed by ordinary Portland cement grouts the following are discussed:

Grouting tests performed in pure Yumurtalık sand specimens (6 % fine content) showed that it is not possible and thus practicable to fracture fine sandy soils by ordinary Portland cement grouts. Because grouting could not be successful in most of the tests performed in this sand specimen. The reason for this may be very high percentages of pressure filtration of water in the grout. In these tests water filtration causes the water to be squeezed out of the grout increasing the grout viscosity and thus preventing the grout to penetrate into the ground. Indeed when the soil specimen is mined through to expose the injected grout mass, almost no or a very small amount of grout was observed at the tip of the grouting pipe (Figure 4.37). This behavior, to some extent, resembles to the compaction grouting mechanism where the grout material expands around the injection pipe without fracturing. The grout forces a mass within the soil to expand, compacting and displacing the adjacent soil. However, this is not a fracturing mechanism, but on the contrary resembles to compaction grouting which is sometimes referred as intrusion grouting. In all of the tests with ordinary Portland cement (water/solids ratios varying from 1.5 to 3.0) same behavior is observed. This conclusion is also reached through the observation of the pressure build up during injection. In these tests, a rapid pressure increase was observed followed by a refusal. This means that the water is separated from the grout when it is forced into the soil specimen under pressure and thus the remaining solid cement particles accumulates around the tip of the grouting pipe and eliminates further injection. At this point the difference between a micro fine cement grout and an ordinary Portland cement grout is understood clearly. A micro fine cement having a much higher Blaine fineness (almost 10 times higher) and less specific gravity behaves as colloidal rather than a suspension as in the case of an ordinary Portland cement.

In order to decrease amount of filtration and to see if grouting would be possible, 8%, 15 % and 25% of kaolinite was mixed with the Yumurtalık sand as a replacement by weight. These correspond to 14 %, 21 % and 31 % fine contents, respectively. Grouting tests were performed in these soil specimens by taking the water/solids ratio of the grout 0.7, 1.0, 1.5 and 2.0. Grouting was successful in these tests, however, significant differences were observed based on the added kaolinite content.

All of the tests indicate that the fracturing pressure (or the maximum observed pressure measured during grouting) decreases as the water/solids ratio of the grout increases. This is simply because of the smaller viscosity values of the grouts with higher water/solids ratio. As the grouts get thicker it becomes more difficult to fracture the soil specimen and thus higher pressures are required. Figure 4.41 and Figure 4.46 show the decrease of fracturing pressures as the water/solids ratio increases. All the soil specimens, whether mixed with an 8 % or 25 % of kaolinite; showed the same tendency.

Injected volumes – grout pressure build up relationships for the soil specimens placed at lower densities (except soil specimens mixed with 8% of kaolinite) generally show that the soil specimen could be fractured more than once with grouts having water/solids ratio of 1.5 and 2.0. As the grout gets stiffer, i.e. as it becomes more viscous, then it tends to compact rather than to fracture the soil. A continuous build up of grouting pressure without any falls shows this effect. Indeed, grouting pressure records of the tests performed with grouts of water/solids ratio of 0.7 or 1.0 do not show much drops in pressure as in the case with grouts of higher water/solids ratio. However, this behavior is not seen in the tests performed in soil specimens mixed with 8 % of kaolinite. Here, there occurs a continuous pressure build up regardless of the water/solids ratio of the grout (Figure 4.38 and Figure 4.45). The reason for this behavior may be directly related to the amount of filtration during grouting. If the filtration percentages are compared, higher values are observed for the specimens mixed with 8% of kaolinite. Since the grout in these specimens loses its water more than it does in other specimens mixed with 15 % or 25 % of kaolinite, it compacts around the injection pipe without fracturing. This is also accompanied by a continuous pressure increase. However, in specimens mixed with 15 % or 25 % of kaolinite, filtration values notebly less and thus the grout has the ability to fracture the soil specimen. This result shows the difference between what is called intrusion grouting in the literature and the fracture grouting.

Tests performed on soil specimens mixed with 25 % kaolinite show that the fracturing pressure difference between grouts of water/solids ratio of 1.5 and 2.0 is not much. However, for soil specimens mixed with 15 % and 8 % of kaolinite, this difference in fracturing pressure becomes larger. This may be due to the observance of higher filtration percentages for the latter. An increase in the amount of water filtrated out of grout shows its effect on the grouting pressure significantly.

As the kaolinite content added to the pure Yumurtalık sand increases from 8 % to 25 %, fracturing pressures tend to decrease. This result may be due to the fact that higher pressures are required as the filtration percentages increase.

As for the relationship between the filtration percentage and the water/solids ratio of the grout, it is seen that filtration amounts increase as the water in the grout increase. This is because the grout becomes more mobile and less stable and loses more of its water under the influence of the grouting pressure. However, when it is stiffer, i.e. more viscous, it tends to compact the soil by enlarging in diameter around the grouting pipe. During this enlargement process, it of course loses some of its water, but not as much as in the case with less viscous grouts. In other words, making the grout more unstable by adding water more than the required will increase the filtration percentages. It should be noted that there seems to be an optimum range of water/solids ratio for the ordinary Portland cement grouts in view of fracture grouting. According to the tests this range is between 1.0 and 1.5. If the stiffer case is considered (i.e. grouts with water/solids ratio less than 1.0) grout compacts the soil around the injection pipe rather to fracture it. Here, grout compacts the soil not because it is exposed to very high pressure filtration but because of its high stiffness. On the other side, when the grout is very mobile, (i.e. grouts with water/solids ratio more than 1.5), this time the grout itself is exposed to high pressure filtration, and losing most of its water, it again accumulates around the injection pipe compacting the soil with the requirement of higher pressures. However, for the middle range (i.e. with grouts of water/solids ratios between 1.0 and 1.5), grout is not much stiff and also is not exposed to high pressure filtration. At this situation it is more probable to fracture the soil with this type of grout. However, it should be noted that the effect of various additives used in the grouting industry is also very important. In our case 5 % bentonite and 1 % naphthalene sulfanate based superplasticizer have been used as a replacement for cement.

The dry densities of the exposed grout masses also show variance and tend to increase as the water/solids ratio of the grout increases. This is especially the case for soil specimens mixed with 15 % and 25 % of kaolinite. The reason for this result may also be correlated with the amounts of filtration percentages. As the water/solids ratio increases, the filtration percentages increase and the injected grout mass start to solidify with more of its water being filtrated. Thus it becomes the accumulation of solid cement particles with a less porous structure. On the other hand, as the water/solids ratio decreases filtration percentages decrease and the exposed grout masses tend to have smaller dry densities since the injected volume starts to solidfy with retaining most of its water.

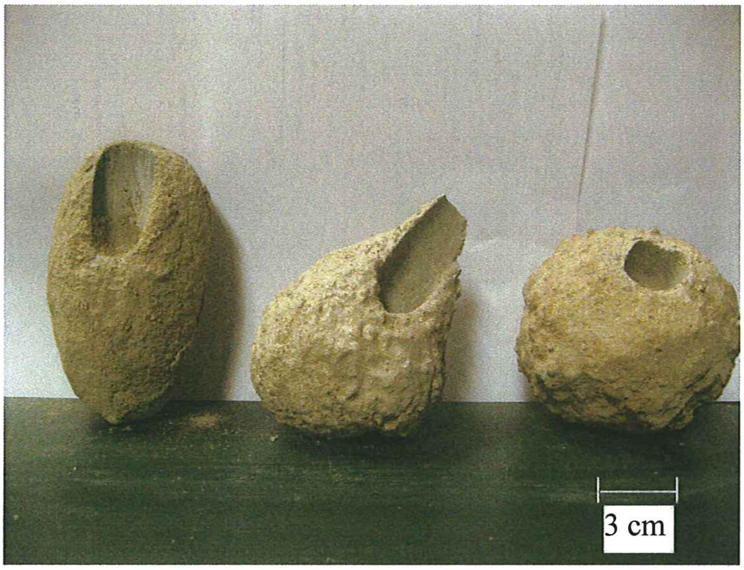
However, for soil specimens mixed with 8 % of kaolinite there is not a general tendency but the dry density values are almost higher than those obtained in the other soil specimens. The reason for this result may be that the filtration percentages are rather high regardless of the water/solids ratio of the grout mixtures (See Table 4.5) and the highest dry densities were obtained. In other words, filtration percentages are more significantly affacted by the fine content rather than the viscosity of the grout for these soil specimens.

Placement of soil specimens at a higher density affected both the fracturing pressures and the filtration percentages. Higher fracturing pressures were obtained as expected. Requirement for a higher fracturing pressure also increased the filtration percentages. This may be due to the fact that the grout material is exposed to a higher grouting pressure during injection. The soil specimen showing a higher resistance to grouting may also produce this result.

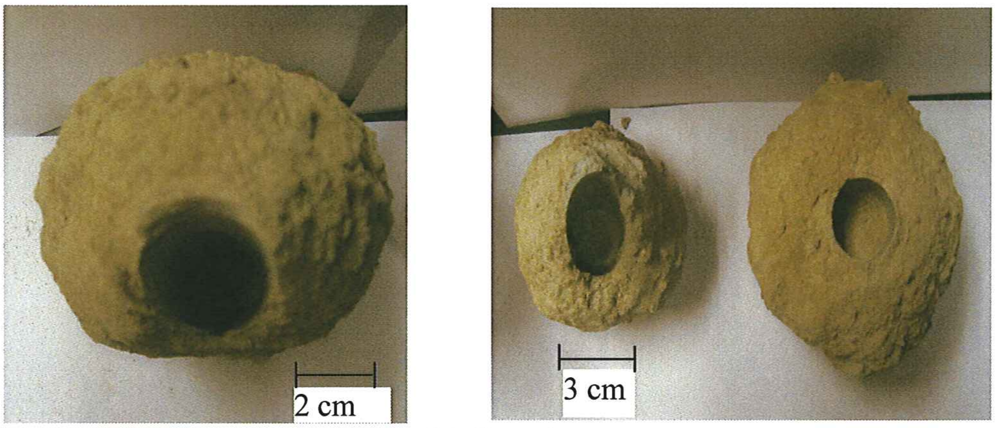
During grouting into the soil specimens which were placed at a higher density some amount of heaving were measured at the top of the specimen after a certain volume of grout were injected. In other words, there was less heave until a certain volume was injected, but after this point the rate of lift started to increase. These two stages correspond to the preconditioning phase and the lifting phase in a typical compensation grouting application. In the first phase in which almost no or less heaving was measured, the soil specimen is conditioned and compacted without reflecting the injected volume as a heave at the top. But in the second phase, further injections provide a certain amount of heave, this time soil specimen being already compacted in the previous phase.

However, tests performed on soil specimens placed at a lower density produced almost no or a negligible amount of heave at the top of the specimen. Because the soil specimen is already loose and almost all of the injected grout volume provides the preconditioning phase in these tests. Heaving would have been obtained if grouting had been continued beyond this point. In other words, the original density of the specimen determines the duration and the volume of grout to be injected until a certain amount of heaving to be obtained. In denser soils, both the volume of grout to be injected and thus the required injection time are less than those in looser soils in order to achieve a certain amount of lift.

Grouting tests performed on soil specimens of higher relative density show that more heaving is measured when the mixed kaolinite content is 8 % and 15%. However, soil specimens mixed with a 25 % kaolinite produced less heaves (Figure 4.46). This may be correlated both with the shape of the exposed grout mass and the filtration percentages. In soil specimens mixed with 8 % or 15 % of kaolinite, filtration amounts are higher and thus the injected grout volumes tend to accumulate at the tip of the injection pipe representing the compaction grouting mechanism. This volume reflects itself as a heave at the top. However, when the soil specimens are mixed a 25 % of kaolinite, filtration percentages decrease and the grout material is more capable to fracture rather than to expand in a globular mass around the injection pipe. Indeed, the exposed grout masses support this result. Figures 4.51, 4.52 and 4.53 show the esposed grout masses obtained from soil specimens of higher density mixed with an 8 %, 15 % and 25 % of kaolinite respectively.

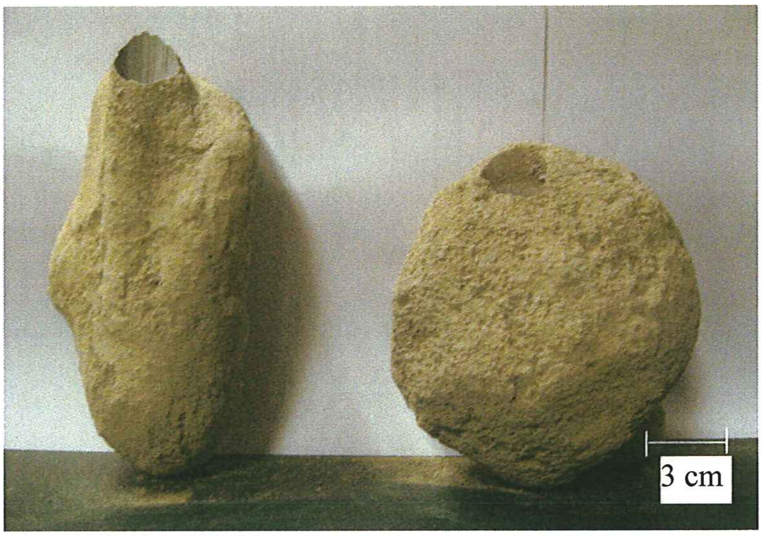


(a) Side view

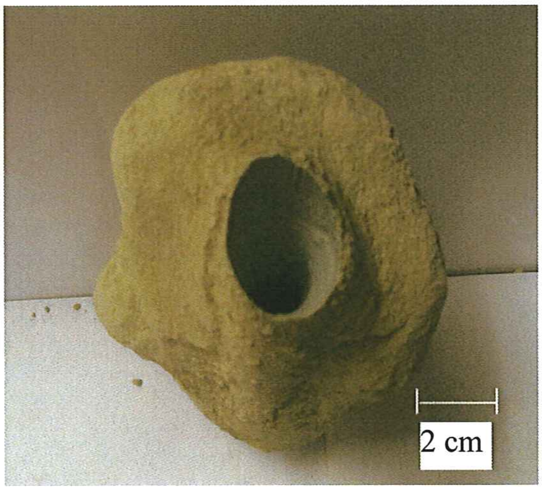


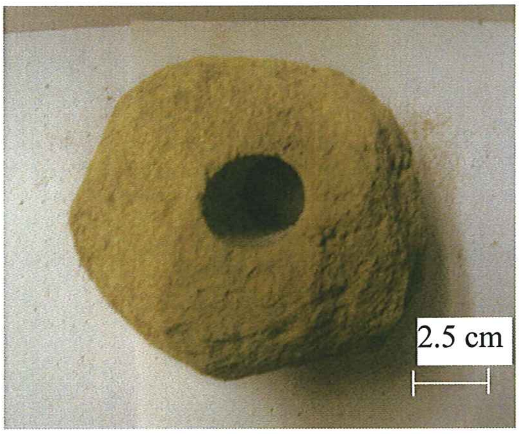
(b) Top view

Figure 4.51 Grout masses exposed from soil specimens of higher density having 8 % of kaolinite (water/solids ratio=1.5)



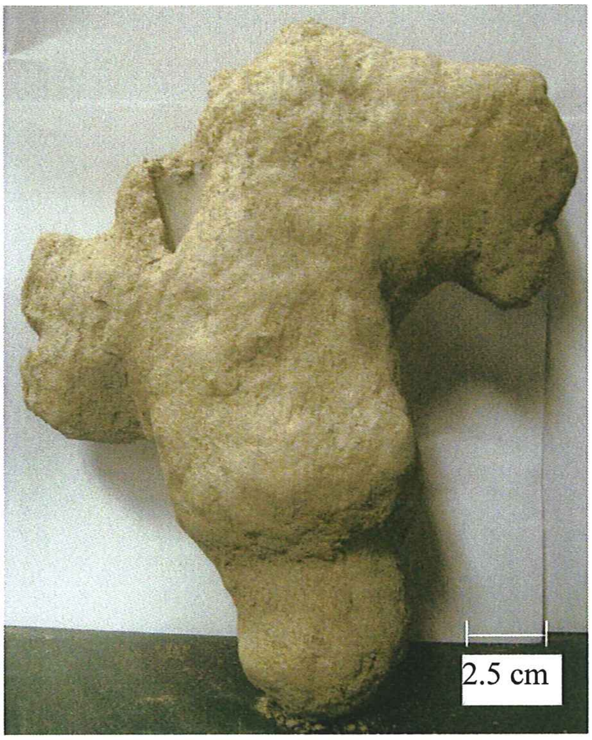
(a) Side view





(b) Top view

Figure 4.52 Grout masses exposed from soil specimens of higher density having 15 % of kaolinite (water/solids ratio=1.5)

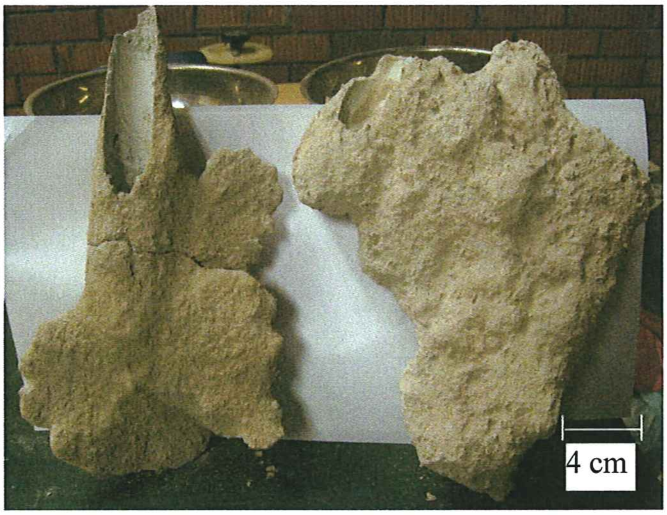


(a) Side view

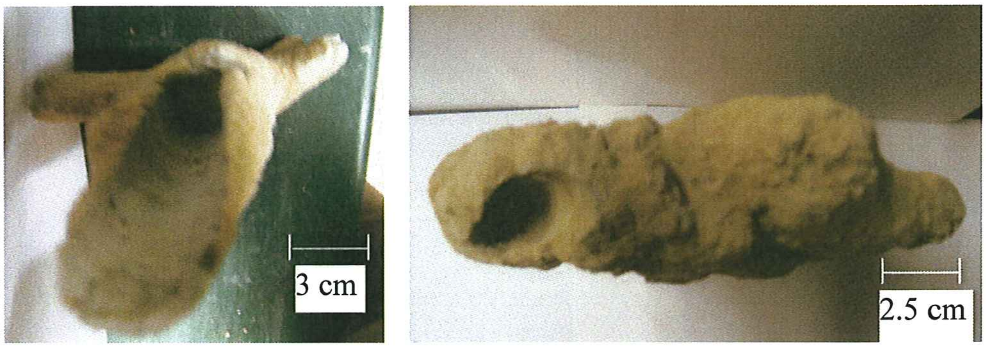
(b) Top view

Figure 4.53 Grout masses exposed from soil specimens of higher density having 25 % of kaolinite (water/solids ratio=1.5)

The shape of the exposed grout mass was influenced both by the kaolinite content and the water/solids ratio of the grout material. In soil specimens mixed with a 25 % of kaolinite the shapes of the exposed grout masses were generally of fractured type. In other words, the extend of the grout from the injection point was greater. On the other hand, in soil specimens having lower kaolinite content, especially in specimens of 8 % kaolinite, exposed grout masses were in the form of a more globular shape tending to expand around the injection point representing the compaction grouting mechanism. These results may also be correlated with the filtration percentages. As the kaolinite content in the soil specimen increases, filtration of water in the grout decreases. Thus the grout, retaining more its water under pressure, has more capacity to fracture into the soil specimen. In the other case, filtration percentages increase in soil specimens of lower kaolinite content and thus the grout material tends to accumulate around the injection point with more of its water being filtrated out. Figure 4.54 and 4.55 show the grout masses exposed through the specimens of lower density mixed with 25 % of kaolinite for the water/solids ratios of 1.5 and 1.0 respectively.

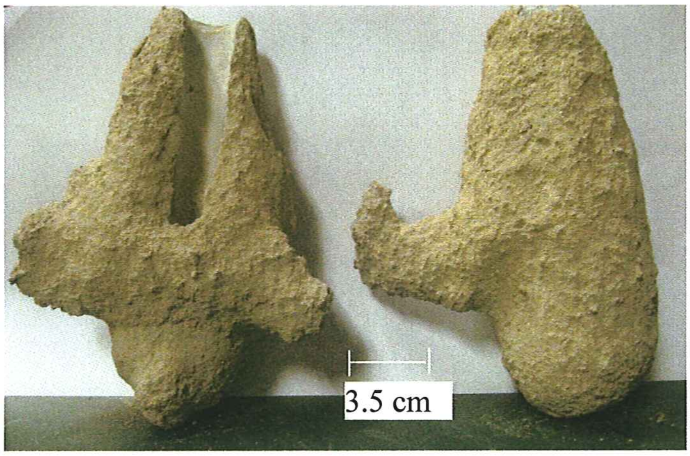


(a) Side view

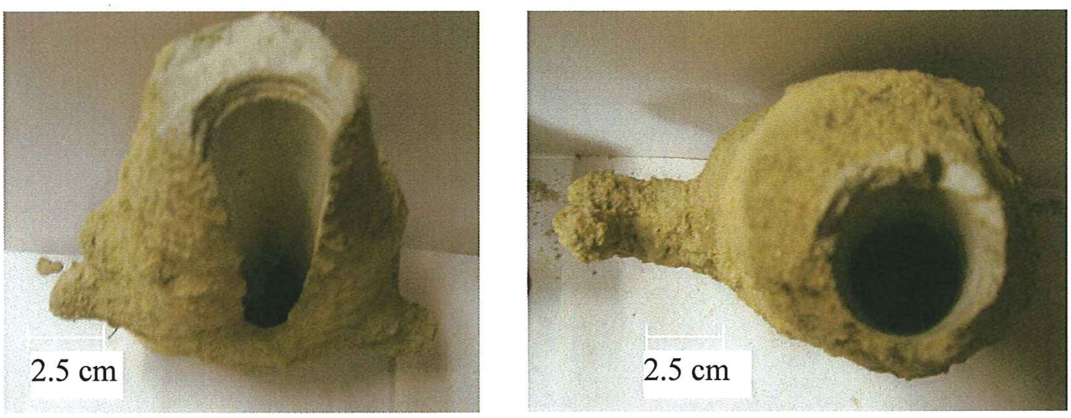


(b) Top view

Figure 4.54 Grout masses exposed from the soil specimens of lower density having 25 % of kaolinite (water/solids ratio = 1.5)



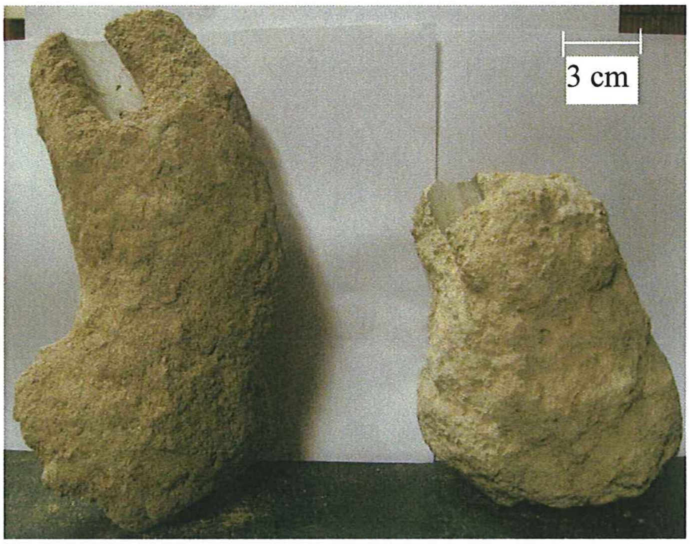
(a) Side view



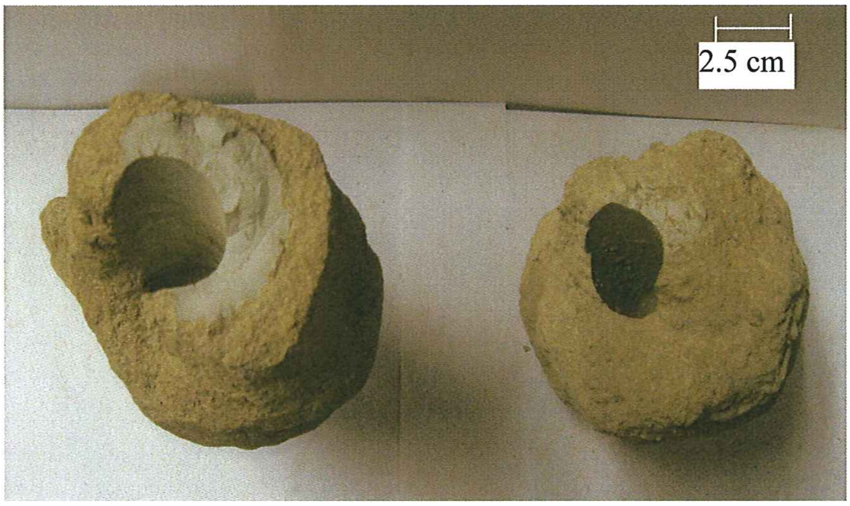
(b) Top view

Figure 4.55 Grout masses exposed from the soil specimens of lower density having 25 % of kaolinite (water/solids ratio = 1.0)

The water/solids ratio also affected the shape of the exposed grout masses. When the amount of water in the grout was increased beyond a certain limit (i.e. water/solids ratio of 2.0) higher filtration percentages are obtained regardless of the kaolinite content and the grout volume tends to expand around injection point rather than to fracture. This result was obtained in some of the soil specimens containing 25 % kaolinite when they were injected with a grout having a water/solids ratio of 2.0 (Figure 4.56). Therefore, it is recommended that the water/solids ratio of the ordinary Portland cement grout, that will be used in fracture grouting application of a fine sandy soil, should not be increased excessively. The test results and the exposed grout masses show that a water/solids ratio between 1.0 and 1.5 is the optimum value for the fracture grouting.



(a) Side view



(b) Top view

Figure 4.56 Grout masses exposed from the soil specimens of lower density having 25 % of kaolinite (water/solids ratio = 2.0)

CHAPTER 5

CONCLUSIONS

5.1 General

In this study, fracture grouting technique of saturated, loose, fine sandy soils of different fine content was investigated. Basically, relationships were obtained between soil conditions (grain size distribution, relative density, overburden stress) and grouting parameters (grouting pressure, injected volume of grout, rheological properties of the grout or water/solids ratio) both for micro fine cement and ordinary Portland cement grouts.

Model tests were carried out to simulate fracture grouting technique. During the tests grouting pressures and the injected volumes were recorded and at the end of the tests grouted soil specimens were exposed carefully and final grout shapes were observed and correlated with the grouting parameters. Amount of heave (lift) occurred at the top of the specimen during injections was recorded at each test.

Micro fine cement grout and ordinary Portland cement grout showed significant differences rheologically. Micro fine cement grout, with much higher Blaine fineness, lower specific gravity, lower viscosity and cohesion, lower bleed and filtration coefficients, made it possible to fracture the sandy soils of different fine content. On the other hand, grouting tests performed in pure Yumurtalık sand specimens showed that it is not possible and thus practicable to fracture a clean fine sandy soil (with 6 % fine content) with ordinary Portland cement grouts. The reason is the occurrence of very high percentages of pressure filtration of water in the grout. Water filtration causes the water to be squeezed out of the grout increasing the grout viscosity. This is followed by a rapid pressure increase and further injection into the ground could not be possible. Addition of 8 % of kaolinite reduced the filtration.

Increased kaolinite contents (fine content of the soil specimen) resulted in fracture or compaction grouting of the sand specimens.

5.2 Comparison of conclusions for micro fine cement and ordinary Portland cement grouting

Effect of grout mixture

The results of the tests performed with both micro fine cement and ordinary Portland grouts show that fracturing pressure (or the maximum observed pressure measured during grouting) generally decreases with an increase in the water content of the grout. However, significantly higher fracturing pressures are required for ordinary Portland cement grouts due to considerable amounts of water filtration.

Effect of soil conditions

It is observed that fracturing pressure for micro fine cement grouts generally increases as the fine content of the soil increases. However, for ordinary Portland cement grouts, fracturing pressures tend to decrease as the fine content of the soil increases. This is because of reduction in filtration percentages.

It is seen that relative density of the soil affects the fracturing pressure significantly both for micro fine and ordinary Portland cement grouts.

5.3 Other conclusions regarding micro fine cement grouting

For micro fine cement grouts, the volumes of grout injected into soil specimens until fracturing indicate an increasing tendency as the water/solids ratio decreases. When the grout gets stiffer, the mechanism turns into the compaction grouting and tends to accumulate at the tip of the injection pipe. When the grout becomes more mobile (grout of higher water/solids ratio) then it is more easy and possible to fracture the soil.

The volumes of grout injected into soil specimens until fracturing are higher for soil specimens with lower fine content. This behavior shows the effect of fine content on the penetration of grouts into the soil specimens. It is probable that some amount of permeation grouting occurs at the initial stages of pressure increase.

Injected volume until fracturing becomes significantly less as the relative density increases.

Effect of relative density on the volume of grout injected until fracturing is more influencial than that of fine content.

It was observed that an increase in overburden stress applied on top of the specimen results in an increase in the fracturing pressures.

At lower relative densities no vertical heave is observed until fracturing. When the relative density of the soil specimens are increased some amount of heave is observed as the volume of grout until fracturing is injected with increasing grouting pressure. In case of soils with lower relative densities initial injections provide the preconditioning phase.

5.4 Other conclusions regarding ordinary Portland cement grouting

The soil specimens placed at lower densities (except soil specimens mixed with 8% of kaolinite) were fractured more than once with grouts having water/solids ratio of 1.5 and 2.0. As the grout gets stiffer, it becomes more viscous and tends to compact rather than to fracture the soil. In soil specimens mixed with 8 % of kaolinite, there occurs a continuous pressure build up regardless of the water/solids ratio of the grout due to the high percentages of water filtration.

An increase in the amount of water filtrated out of grout during injection affects the grouting pressure significantly.

Filtration percentages during grouting increase as the water/solids ratio of the grout increases.

The dry densities of the exposed grout masses vary and tend to increase as the water/solids ratio of the injected grout increases.

Placement of the soil specimen at a higher density increased both the fracturing pressure and the filtration percentages during grouting.
For the soil specimens placed at a higher density some amount of heaving were measured at the top of the specimen after a certain volume of grout were injected. However, tests performed in soil specimens placed at a lower density produced almost no or a negligible amount of heave at the top of the specimen.

For soil specimens of higher relative density more heaving is recorded as the kaolinite content is incerased from 8 % to 25 %.

The shape of the exposed grout mass is affected by both the kaolinite content (or fine content of the soil specimen) and the water/solids ratio of the grout material.

In soil specimens having 25 % kaolinite content, the exposed grout masses were generally of fractured type. On the other hand, in soil specimens with lower kaolinite content, especially in specimens of 8 % kaolinite, exposed grout masses were in a globular shape expanding around the injection point.

When the amount of water in the grout was increased beyond a certain limit higher filtration percentages are obtained regardless of the kaolinite content and the grout volume tends to expand around injection point rather than to fracture. Therefore, it is recommended that the water/solids ratio of the ordinary Portland cement grout that will be used in fracture grouting application of a fine sandy soil should not be increased excessively. A water/solids ratio between 1.0 and 1.5 seems to be the optimum value for the fracture grouting.

5.5 Recommendations for future studies and practical aspects

In this study, fracture grouting of fine sand by both micro fine cement and ordinary Portland cement grouts was investigated. Basically, relationships were obtained between soil conditions (grain size distribution, relative density, overburden stress) and grouting parameters (type of grout, grouting pressure, amount of injected grout, rheological properties of the grout or water/solids ratio). Effect of each parameter on the response of soil to grouting operation was determined.

It was aimed to provide guidance for grouting engineers such that which grouting parameters could be used in a soil layer of certain characteristics for fracture grouting to be applied successfully. In other words, it was tried to make clear whether fracture grouting of a certain soil site could be performed efficiently with a certain type of grout or not.

However, control of fracture grouting was not studied in this research. Improvement levels obtained are not discussed. There exists improvement in the soil layer in the form of reinforcement. Therefore, volumetric approaches (injection of a certain volume of grout into a fixed volume of soil layer) could be utilized for quality control. Tests to determine the change in ground parameters (e.g. the decrease in ground permeability or increase in mass strength) or the controlled ground movements also enable the success of the grouting to be quantified. In this way, a further research could be performed to assess the improvement level obtained by fracture grouting with certain grouting parameters.

Other improvement techniques such as dynamic compaction or vibro compaction cause high vibrations under existing buildings and can not be performed within the buildings. Therefore, grouting technique with its small, compact equipment, becomes the alternative in this case and it could be performed within the buildings without interrupting the operations of these facilities. Cost of micro fine cement is approximately three times that of ordinary Portland cement. This is due to the requirement of higher grinding action to be employed during the production stage. However, in small scale projects, where fracture grouting with ordinary Portland cement grouts could not be utilized efficiently due to soil characteristics, use of micro fine cement grouts could provide a viable solution.

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PUBLICATIONS

1. Tunçdemir, F., Ergun, U., 'Fracture Grouting of Sand by Micro Fine Cement Grouts', Journal of Ground Improvement, Proceedings of the Institution of Civil Engineers, ICE, Thomas Telford, 2008.

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HOBBIES

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