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# Geotechnical Grouting

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# Transportation Research Record 1104

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# Grouting on the Run

MARTIN PRAGER

**A pressure-grouting technique was used to repair a highway section damaged by erosion in the base course from water infiltrating through joints and cracks. The grouting objectives for this project were established in an effort to underseal the highway section, preventing further erosion. To accomplish this task, new techniques were created to overcome material-handling and time constraints. A farm tractor was rebuilt and converted to a drilling machine that allowed a series of holes to be drilled simultaneously in order to drill the more than 67,000 holes estimated for this project. This equipment, with grouting plant and compressors in tow, performed assembly line operations and all grouting work was completed in 80 working days. After experimenting with regular rock and carbide bits, carbide bits were chosen for their durability and superior strength characteristics. The Texas State Department of Highways and Public Transportation (TDHPT) developed specifications and other constraints for this work. Some of the most notable included the number of hours that could be worked per day, testing of grout mix characteristics, and clean-up requirements. All short-term objectives were achieved by grouting, that is, the stabilization of the slab and undersealing. In addition the scope of the project was found to be larger than most (36 mi or 58 km), and preliminary grout quantities were hard to obtain, though independent estimates by TDHPT were within roughly 20 percent of actual quantities of grout used. The experience gained by completion of this project was considered valuable for future state highway rehabilitation projects.**

Pressure grouting is an effective and labor-saving means for extending the life expectancy of a highway system that is experiencing problems created by subsurface conditions. It is used by many state highway and transportation agencies to accomplish three primary functions: undersealing existing pavement, raising deflected pavement, and filling voids.

A section of Interstate highway 35E between Waxahachie and Hillsboro, Texas, was deteriorating rapidly due to increased and heavy load-bearing traffic. Moisture was infiltrating the pavement and accumulating underneath. Over time, the cycling of heavy loads and resulting hydraulic pressures had created pocket voids between the pavement and base material. To help minimize further deterioration, the Texas State Department of Highways and Public Transportation (TDHPT) had installed a series of subdrains. The drains were successful in removing some water from beneath the pavement; however, the continued infiltration of water combined with heavy truck loads led to a worsening pavement condition.

The grouting objectives for this project were established to underseal the highway section to prevent further infiltration of moisture and subsequent pavement deterioration. The job description did not include the raising or leveling of pavement sections, or both.

To repair this highway section the scope of work included replacing broken slab sections, undersealing the pavement, replacing the shoulders, and overlaying the entire pavement with an asphaltic concrete leveling course. The project consisted of approximately 4.5 million square feet (418 500 $m^2$) of pavement. The work area for grouting was almost 2.8 million square feet (260 400 $m^2$), an area 18 mi (29 km) of one lane northbound and 18 mi (29 km) of one lane southbound.

The concrete was 8-in. (20-cm) and 10-in. (25-cm) thick continuous reinforced pavement laid upon a 3-in. (15-cm) stabilized crushed limestone base course. The grout holes were to be drilled to a maximum depth of 12 in. (30 cm). This limitation was specified by the TDHPT to restrict the grout placement between the bottom of the concrete and the top of the subbase, as the primary problem was water infiltrating through joints and cracks causing erosion in the base course.

The grouting portion of this project was originally designed for 120 working days and estimated time for completing the entire project was 360 days. In order to save a paving season, the general contractor decided to push the completion date ahead 180 days. By another agreement it was established that the grouting portion would be completed in 80 working days. Some other notable restrictions in the scope of work required careful planning. Traffic could not be blocked. This meant that traffic had to move in at least one lane at all times and only 2 mi (3.2 km) of highway (in each lane) could be barricaded during work progress. Strict traffic patterns had to be followed in moving equipment and materials, and no U-turns, crossing of median strips, or parkways were allowed. Although six turnaround points were located inside the 18-mi (29-km) highway work area, logistical problems still remained on the length of supply routes. The spacing of the exits (including the beginning and ending exits) was such that it was common to travel a 25-mi (45-km) round trip to get water or materials and it could easily take more than 1 hr for each trip. The limitation to one lane for working eliminated larger off-road equipment and facilities normally available at a dam work site. For example, it is not uncommon to have silos for bulk material storage and distribution, water pipelines for grouting and clean-up, and large stationary grout plants, all located on the work site. The limited area and the requirement to completely grout more than 3,000 linear ft (900 m) per day made bulk material handling and storage infeasible. Therefore, all materials were supplied in bags and handled with forklifts and by personnel. Italy, Texas, was selected to be the storage and staging area for the job. Although several miles off the highway, it was still the most accessible site for the project.

The most difficult problem encountered posed an interesting challenge. Because extended periods of rain prevented the holes from being pre-drilled, the Hydraulic Ram Effect, caused

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by loading and the action of water in the holes, could accelerate the deterioration of the subbase.

Mobilization at the staging area began in mid-May 1984 in anticipation of a starting date the last week of the same month.

The grout specifications called for a mixture of 3 parts pozzolan (fly ash) to 1 part cement (Type I) and a superplasticizer as needed. A typical batch consisted of six bags (450 lb or 204 kg) of the pozzolan to two bags (188 lb or 85.3 kg) of portland cement Type I, and 1 oz (30 ml) of superplasticizer. The amount of water required was to be sufficient to create a flow that would meet the requirements of the TDHPT Flow Cone Method. In this case, the average volume of water used per batch was 27 gal.

The Flow Cone Method is a field test used for determining the flow of grout mixtures by measuring the time of efflux of a specified volume of grout from a standardized flow cone. The apparatus is a simple cone device normally constructed of cast aluminum. The procedure is to moisten the inside surface of the flow cone and to place a finger over the outlet of the discharge tube while grout is introduced into the cone. A specific quantity of grout is measured when the grout surface is in contact with the point gauge. The stop watch is started and the finger removed simultaneously. The elapsed time is noted at the clean out of grout from the discharge tube, and the time indicated by the stop watch is the time of efflux of the grout. At least two tests were conducted for any grout mixture and the average time was used to verify compliance with specifications.

The grouting equipment consisted of one shop-made plant, mounted on a trailer, and a commercial trailer-mounted plant. The shop-made plant was a standard grout unit consisting of a high-speed colloidal mixer, agitating tank, water tank, and two Moyno pumping units. The smaller commercial plant was to serve as a standby or filler for small areas that had been passed for one reason or another. A 600 $ft^3$/min (17 $m^3$/min) compressor was used to power the air-operated equipment.

A unique method was devised to meet the challenge of drilling more than 67,000 holes estimated for the project. Normally all holes are hand drilled but there are exceptions, as in this special case where time constraints were important. Consequently, design and reliability of the drilling machine were critical to the successful completion of the project.

A farm tractor was converted to a drilling machine that allowed a series of holes to be drilled simultaneously. Air-operated drills were attached to a carrier beam, which in turn was controlled by the tractor hydraulics. The beam was constructed to allow rotational ability to perform varied drilling patterns along latitudinal and longitudinal axes. Preconstruction experimentation revealed that for this project four drills driven by a 450-$ft^3$/min (13-$m^3$/min) compressor were the most practical means to obtain the desired cost-effective production. The drill tractor towed the compressor and was equipped with a guide rod attached to the front to enable the operator to maintain the proper spacing and alignment of the holes. The machine was able to drill four holes and move into position for the next set within 90 sec. One man was required to operate the drill machine. Another man accompanied the drill to change drill steel and rock bits when necessary. Whenever the drills ran into reinforcing steel the operator withdrew that particular drill and continued on. The assistant, using a hand drill (which was connected to the air compressor) would then drill another hole adjacent to the blocked hole.

The drilling methodology was established in the beginning by trial and error and also by observation of time trials when using the equipment. For example, regular rock bits were used at first, but later were changed to hard carbide bits when it was observed that only 40 holes could be drilled per regular bit to about 200 holes for each carbide bit. Another problem encountered at high drilling speeds was fatigue and cracking in the drill steel. The problem was solved by changing drill steel every 30 holes and rotating material to increase the longevity of the steel, reducing steel replacement by more than 70 percent.

A normal work day would begin with the drilling crew at work at 8:00 a.m. (time restriction by state), while the grout train was still being prepared for operation. About an hour later, after drilling ahead, the actual grouting procedure would begin. The Flow Cone test was used to determine the amount of water required for the first batch. It was run any time the plant started operations and at several times throughout the shift.

The grouting procedure used 100 ft (30 m) of 1½-in. (4-cm) high-pressure grout hose connected to a rubber nozzle. The nozzle had mechanical attachments so it could also work as an expandable packer. An on-off valve was installed at the nozzle head to ensure complete control over the pumping. A pressure gauge was located on the discharge side of the pump. The necessity of grouting all the drilled holes during the same day limited the available drilling time to 7 hr each day. The drill crew spent the remaining time marking holes ahead or going behind the grouting operation to assist in cleaning up and patching holes. The holes were patched with the same grout mix, which meant placing grout from a bucket into each hole.

A flatbed truck towed the grout plant, with the air compressor connected to the rear of the grout plant. A water truck was used to supply the bulk water for the tank on the grout trailer and for cleaning the pavement behind the grouting operation. An additional flatbed truck was used to shuttle materials from the staging area to the job site. The materials were transferred to the truck towing the grout plant where they could easily be loaded into the machine by one or two people, depending on the rate of grout pumped at the time of operation.

No absolute conclusions are evident when considering the long-term objectives. The number of years of life expectancy added to the pavement will only be known at some future date. The immediate objectives, however, were satisfied. The short-term objectives, the stabilization of the slab at the outer boundary where water was infiltrating and the sealing of this area by grout, were accomplished. Actual observations were made to verify this by the TDHPT. The success of this procedure will be determined, ultimately, by the TDHPT, but the results look promising. It is important to note that pavement sections that had failed were replaced, making it hard to determine what part grouting played in extending actual highway longevity. It can only be suggested that its contribution was significant. Other significant points were: *(a)* the scope of the project was larger than most (36 mi or 58 km), *(b)* it was hard to obtain an accurate approximation of grout quantities needed, *(c)* TDHPT estimates came within roughly 20 percent of actual quantities of grout used, and *(d)* no significant improvements to the estimating process were established as a result of this project. It is concluded that the experience gained through this project will be significant to both the TDHPT and grouting contractors when repair of this type is needed in the future.

# Report and Review of a Major Slabjacking Case History

SAMSON W. BANDIMERE

**A review is presented of a major case history of lifting and leveling portland cement concrete highway slabs by injection of a fluid cement/pozzolan material between the subgrade and the slab. The paper includes history and background data as well as current practices, advantages, disadvantages, and specific procedures. Grout materials, mixtures, equipment, and lifting controls are discussed.**

U.S. Interstate highways constructed under the 1956 Federal-Aid Highway Act are aging to the extent that general maintenance is no longer acceptable for their serviceability. Major innovative rehabilitation methods are required to restore the wearing surfaces, and, in many cases, the stability of the subgrades. The intent of this paper is to illustrate what is being done with the problem of substrate stability. Although not as visually evident as the resurfacing, it will play a major role in the longevity of the overlayment.

For many years it has been standard practice to relevel highway settlements and dips by merely adjusting the thickness of the resurfacing material, only to have it prematurely spall and break up. A soft substrate or voids below the original slabs perpetuate the problem under the loading and unloading effect of traffic. This condition, known as pumping, can be effectively eliminated by a system known as slabjacking *(1)*.

The modern-day process of slabjacking evolved mainly from work that was performed during the past 50 years under the term "mudjacking." While technically both terms are synonymous, the main reasons for the change in terminology are as follows: In the mid-1920s a machine was invented for the specific purpose of mixing soil-cement in a slurry and pumping it to fill voids. By over-pumping, pressure was applied, resulting in the lifting of a structural element. The manufacturer named this machine the Mudjack. Because of its widespread use over many years, the entire process adopted the term of mudjacking. Slabjacking more correctly describes the effect of the process on the component. As the technology improved, mudjacking was no longer limited to the use of soil-cements alone, as the following discussion will demonstrate.

In June of 1981 the Wyoming State Highway Department solicited bids for Statewide Slabjacking Project SMP-7892. It was to involve the lifting and re-leveling of concrete bridge approach slabs, highway dips caused by improper compaction over underground structures, and filling of voided areas behind sloped concrete paving around bridge abutments. There were more than 167 separate locations listed, from Green River to Gillette, Wyoming, on state and Interstate highways 373, 80, 25, and 90.

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## PROJECT DESCRIPTION

The work was to consist of raising concrete pavement slabs by drilling 2-in. (50-mm) diameter holes and pumping grout into the holes under sufficient pressure to bring the slabs to the line grades established by the engineer. This differs from a similar process known as slab undersealing where lifting of the slab is not a consideration. The slabjacking process requires additional operator expertise, time, and materials. The operator must work with the structural capacity of the slab so that it is raised without cracking or breaking.

Some advantages of the slabjacking process are:

1. By lifting the slab or structural member close to its original elevation, the stresses that have built up are relieved, thereby restoring its structural integrity.
2. The lifting process allows grout to travel and seek out additional voids that might otherwise be missed, resulting in a more stabile component.
3. It requires less overlayment, or restores the ride over a section that is not scheduled for resurfacing.

### Equipment Specifications

The contractor was to furnish a grout plant, which consisted of an injection pump and a high-speed colloidal mixer. The colloidal mixer had to operate at a minimum speed of 1,200 RPM and was to have a rotor operating in close proximity to a stator. This creates a high shearing action and subsequent pressure to make a homogeneous grout mixture. A water meter or scale was required to record the day's water consumption.

### Materials Specifications

The standard mix design for slabjacking on project SMP-7892 was to meet minimum and maximum cone effluent efflux times of 10 to 15 sec (Corps of Engineer's CRD-C 611-80), and attain a minimum 7-day strength of 300 psi (2.1 MPa) proportioned as follows:

1 part (by volume) portland cement type I or II, AASHTO M85
3 parts (by volume) pozzolan (fly ash) type C, ASTM C618
2.25 parts (by volume) water

The Wyoming State Highway Department has had experience with several alternative grout designs. including soil-cements, hot asphalts, neat-cements, and so forth, and considered these before choosing the design listed. Reasons for their choice for this project were fivefold:

1. Assured uniformity of materials,
2. Proven ability during underesealing operations to taper out and fill minute voids,

3. Non-shrink characteristics of the design,
4. Strength considerations,
5. The economics and availability of the materials *(2)*.

### Grout Holes

The contractor spaced the holes according to the particular site and the manner in which the slab was raised or tilted, but generally on a 4 ft (1.22 m) grid pattern. The holes were 2 in. (51 mm) in diameter and drilled by a pneumatic percussion method. Upon completion of grouting, each hole was cleaned and patched with a non-shrink 4,000 psi (28 MPa) grout.

### Method of Measurement

The quantity of slabjacking completed was to be measured by the cubic foot ($m^3$) of dry material only—portland cement and pozzolan—used in slabjacking.

### Basis of Payment

The accepted quantity of dry material used for slabjacking was paid for at an in place per $ft^3$ ($m^3$) unit price. The project was estimated at 21,900 $ft^3$ (620 $m^3$). The total project, however, required 24,600 $ft^3$ (697 $m^3$) because of site and change orders required by the state.

## DISCUSSION OF THE PROJECT

Before project No. SMP-7892 was initiated, to this author's knowledge, the combination of portland cement and pozzolan was used extensively for undersealing of concrete pavements but never for the lifting and releveling of slabs which had experienced settlements of 4 in. (102 mm) or more. The fact that the grout mix was to meet a maximum standard cone effluent efflux test of 15 sec meant working with a material that is fluid and subject to excessive flow. There were containment problems at adjacent slab edges, where the concrete slab met the outside bituminous pavement shoulder, and where there was no access to tamp a containment bank.

Control of grout flow and hydrostatic pressures on underground adjacent structures can become an important application consideration. For this reason correspondence and discussions took place between the highway department engineering staff and the contractors before bid opening. The highway department decided to maintain their original specifications.

While the contractor's operating technicians on this project had more than 45 combined years of experience using other slabjacking materials, the use of cement and pozzolan/fly ash required approximately one week of frustrating experience before they became proficient at controlling grout losses around bridge abutments, escape points, and so on. The art of controlling slab lifts also required some experimentation with flow rates, grout consistency, and pressures that were different from anything with which they were familiar.

However, once they learned the characteristics of the material they found that grout loss cutoff and slab lifting could be accomplished with as much accuracy as with any material previously used. The crews reported some of the following observations:

1. The dust from the combined materials causes skin burning and rashes if the slurry is allowed to make contact with the skin.
2. By prepacking known escape points with the dry materials very little if any grout loss was experienced.
3. Hydrostatic pressures against underground structures were less, due to the fast set time of this material resulting in less wait time between lifts when filling large voids against or near them.
4. This grout design often needed some additional material to accomplish the same results as other materials due to greater underslab coverage and because some previous subgrade materials absorbed the grout.

### Slabjacking Operations

The project was awarded to the bidder having the lowest cubic foot ($m^3$) unit cost, with a notice to proceed being issued July 16, 1981. The contractor began work September 5, 1981, shut down for the winter months between mid-November 1981, and April 1982, and completed the project by August 1982. Slabjacking operations were terminated when the ground was frozen or the ambient temperature fell below 40°F (4.4°C).

The contractor pumped in a pattern and with amounts required to raise the pavement to within 0.03 ft (9 mm) from a stringline or established grade. After a slab was raised to its desired elevation, every hole was pumped one more time to ensure that all voids were filled and that the slab had total support. Slabs were not allowed to be raised more than 0.25 in. (6 mm) while pumping in any one hole, nor was any part of a slab allowed to lead another part of an adjacent slab in order to avoid cracking.

### Equipment Assessment

The contractor chose to modify and use equipment that would facilitate the bulk handling of materials as opposed to the standard method of using bags (Figure 1). This required the following major equipment list:

1. A modified Daffin concrete mobile used as a volumetrically calibrated batch plant. It had a loaded capacity of 324 $ft^3$ (9.17 $m^3$) of bulk materials with a 300-gal (1136-L) water tank, all mounted on a tandem truck.
2. A custom-built pumping trailer consisting of a 7 $ft^3$ (0.20 $m^3$) agitating tank, a high-speed colloidal mixing unit, and a 6-in. (152-mm) cavity advancing Moyno pump.
3. Two auger feed bulk tankers, used to tender the concrete mobile with pozzolan and cement materials.

**FIGURE 1 Grout plant set up for bulk handling of materials.**

4. A truck-mounted auxiliary 1,000-gal (3.785-L) water tank.
5. A 175 $ft^3$/min (4.95 $m^3$/min) compressor.
6. Several 60-lb (27-kg) sinker drills with drill steels and bits.
7. Required traffic control equipment including two trailer-mounted sequentially lighted chevrons.

With current technological advances, it would be inappropriate to indicate that this equipment is the only way to set up for this type of operation. However, this project covered four highway districts involving seven different district construction engineers and the consensus was that this equipment best fulfilled requirements for the project specifications.

## PROJECT REVIEW

Due to temperature constraints the project was performed over three seasons, fall of 1981 and spring and summer of 1982. As a result one weather-related factor surfaced. The contractor's operating technicians learned at the start of the project that the set time of the designed mix was directly affected by temperature. For example, as the ambient temperature rose to 80°F (27°C) or more, a grout mix with an initial efflux of 10 sec would set hard in approximately 5 min. However, when the ambient temperature dropped to 50°F (10°C) or less, the same grout mix would begin to stiffen slowly at 20 min or more. This directly affected the slabjacking process in several ways.

High temperature and quick set have the desired characteristic of a fast cutoff, which limits the grout loss at slab edges or other areas of escape. It became apparent on this project that the operating engineer could begin pumping at the lowest point in the slab, and by the time the grout spread out and began oozing from escape points it would set and cut itself off. This happened when the initial fluid mix of the material was pumped at rates of up to 5 $ft^3$ (0.14 $m^3$) per min.

In many cases this allowed the operator to lift an entire slab near to its required elevation from one or two holes. Complete support coverage was indicated by the escape of grout at adjacent holes and edges. Subsequently, the operator merely pumped the surrounding holes with minimum grout takes, resulting in less lifting stress to the slab than pumping hole to hole and lifting at different points. This was evidenced by the fact that less cracking or breaking of slabs occurred than on previous occasions when the contractor had used soil-cement materials.

The set time of some of the material was retarded by chemical additives. In the interest of limiting grout losses, however, the district engineers soon asked the contractor not to use them.

The problems encountered with the fast-setting grout were frustrating but not insurmountable. The operators' experience and expertise are always significant contributing factors in the success of a slabjacking project. If there was to be a delay in the grouting application the operators learned how to anticipate the material's flash set time. The material was discharged into waste areas before the hoses and equipment were locked up. For grout consistency changes a circulating process was accomplished in two ways:

1. When slabjacking in close proximity to the grout plant and pump, one hose was used for proper grout delivery into the holes or recirculating back into the agitating tank for additional mixing.
2. When working with longer pumping distances, a tee was inserted at the injection point with a return line. The grouting technician always kept a portion of the grout flowing back to the grout plant to keep the return line from locking up. For this reason the operators on this project preferred to use, and were much more efficient with, the one line system.

Cooler temperatures and longer set times resulted in delays because of containment and grout-control problems. These were overcome in two ways:

1. The grouting technician moved around the slab using more drilled holes for lifting and filling purposes.
2. Water in the grout mix was reduced resulting in cone test effluxes above specification (18 to 21 sec) to reduce its flowing characteristics. This resulted in cooler temperature jelling set times of approximately 5 min.

Typical grouting operations are shown in Figure 2.

### Safety

The most dangerous aspect of this project was that the grouting technicians were working close to high-speed traffic (Figure 3). Despite repeated attempts by the traffic controllers to slow traffic to the posted 40 mph (64 km/hr) the speed of a passing vehicle in the adjacent lane averaged 57 mph (92 km/hr). This meant that the grouting technicians and engineers had to be alert at all times, especially when working a row of holes along the center dividing line. Semitrailer trucks passing at high speeds would cause a rush of air followed by a suction effect that threatened to pull workers under the truck unless they were

FIGURE 2 Typical grouting operations.

FIGURE 3 Wide loads and high-speed trucks are a constant safety hazard.

secured by tie lines attached to the grouting equipment or were holding securely to the grout injection nozzle.

Due to the many varied locations of this project, it was not feasible to consider installing median barriers for protection; therefore, requesting a patrol car to work with the traffic controllers became the most effective way of holding speeds to a minimum.

## CONCLUSIONS

Despite the problems encountered using a cement, pozzolan/fly ash material for slabjacking, and the contractor's initial concerns about controlling lifts with a fluid, these materials should be considered for future uses for the following reasons:

1. The material shows uniformity.
2. Shrinkage is lessened.
3. The material exhibits greater strength than other slabjacking materials.
4. The material can be used to seek out small voids and penetrate some yielding subgrade materials.
5. It provides more support while lifting or tilting slabs, resulting in less cracking or breaking.
6. The fast set time of the grout mix can be a major asset to the slabjacking process although it requires experienced operators.
7. A maximum cone efflux test of 21 sec is required to perform slabjacking with these materials in cool temperatures.

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# Injection Stabilization of Failed Highway Embankments

JAMES R. BLACKLOCK AND PAUL J. WRIGHT

**Restoration of failed soil embankments along the Interstate highway system is a costly and time-consuming maintenance problem for many state highway departments. Unfortunately, few economical or easy solutions are available for repair and improvement of these failed earth embankment sections. In extreme cases highway bridges have been removed and the earth cross-sectional designs changed because attempts at solving existing bridge embankment slope failures were unsuccessful. In related studies to evaluate a new solution for this problem, Alabama, Arkansas, and Missouri have recently installed full-scale highway embankment test sections using the lime and lime/fly ash injection method of in situ soil stabilization. Presented in this paper is a discussion of the lime and lime/fly ash injection technology necessary for stabilization and restoration of typical failed embankment slopes. Also addressed are lime and lime/fly ash laboratory testing, injection materials selection, injection construction technology, and site evaluation. The first two highway embankment injection-stabilization projects were installed in Alabama and Arkansas in 1983. These are presented in a detailed case study format and their relative degree of success is documented after 2 years of service. The Missouri site stabilized in 1984 is also discussed and a preliminary assessment of its success potential is included. Discussed are the injection-stabilization evaluation techniques that have been derived from these demonstration projects, and suggestions for improvement of future projects are offered.**

Lime and lime/fly ash (L/FA) injection stabilization for improving the engineering properties of embankment soil mass materials is currently being evaluated by highway maintenance engineers in several states. Pressure-injection stabilization with hydrated lime slurry has been used for more than 20 years to stabilize expansive clay soils, and within the past 8 years the addition of fly ash to the process has created numerous new applications for injection stabilization. Because the injection method uses hydraulically inserted injectors rather than predrilled grout holes, it is considerably faster and less expensive than most conventional grouting methods. The two main reasons for the present favorable economics of the L/FA injection-stabilization method are the ready availability of an inexpensive supply of fly ash and the development of new injection equipment.The anticipated continued use of large quantities of coal promises a steady supply of fly ash for future stabilization projects, and the continued development of new and better equipment for injection should promote future improvements in construction economics and performance.

Injection stabilization using lime and L/FA is now an accepted procedure used throughout the United States by most major railroads to stabilize roadbeds and embankments to reduce chronic high-maintenance track problems. Stabilization of expansive clays for building foundations and pavement structures using injection stabilization has also grown rapidly during the past decade. Although use of this system is perhaps most prevalent in the southern and middle states, injection-stabilization is being used increasingly in the eastern and western states as more emphasis and construction dollars are shifting to maintenance and rehabilitation of the transportation infrastructure. There are few alternatives for in-place soil treatment, and injection stabilization is suitable for both pretreatment and repair and renovation. In almost every case injection stabilization is the most economical method available.

The availability of fly ash as an inexpensive grout material has encouraged contractor research and development of equipment and procedures for its use in the L/FA-injection method of soil stabilization. A U.S. patent for injection stabilization with lime and fly ash slurry mixtures was issued to the Woodbine Corporation in 1978. Initially, injection stabilization with L/FA slurry was an alternative stabilization method to be used when lime slurry pressure injection (LSPI) was not appropriate. Gradually, however, it has become obvious that there are many uses for L/FA injection that are not merely alternatives to LSPI but an improvement over other alternatives. The stabilization of highway embankments, discussed in the case histories portion of this paper, is one important use of injection stabilization using lime and L/FA that is currently under development. Limited research is in progress to generate geotechnical engineering data and to promote improved performance and economics of the method.

## INJECTION TECHNOLOGY

The most noticeable difference between pressure-injection stabilization and conventional grouting is in the equipment technology. With pressure injection, typically, large volumes of slurry grout, up to 23,000 gal, are bulk mixed and injected into the soil using various types of hydraulic and mechanical injectors capable of penetrating to depths of 40 ft or more. Conventional grouting more often mixes small batches of cement grout that is pumped through stationary grout pipes that have been placed in predrilled holes. Consequently it is a slower and usually more costly technique than injection stabilization.

### Equipment

A truck-mounted injection vehicle with three 40-ft injectors is shown in Figure 1. This self-contained unit has a 2,000-gal slurry tank with a mechanical agitation system and a high-pressure pump capable of pumping more than 3,000 gal/hr at

J. R. Blacklock, School of Engineering Technology, University of Arkansas at Little Rock, Little Rock, Ark. 72204. P. J. Wright, Woodbine Corporation, 2510 Decatur Ave., Fort Worth, Tex. 76106.

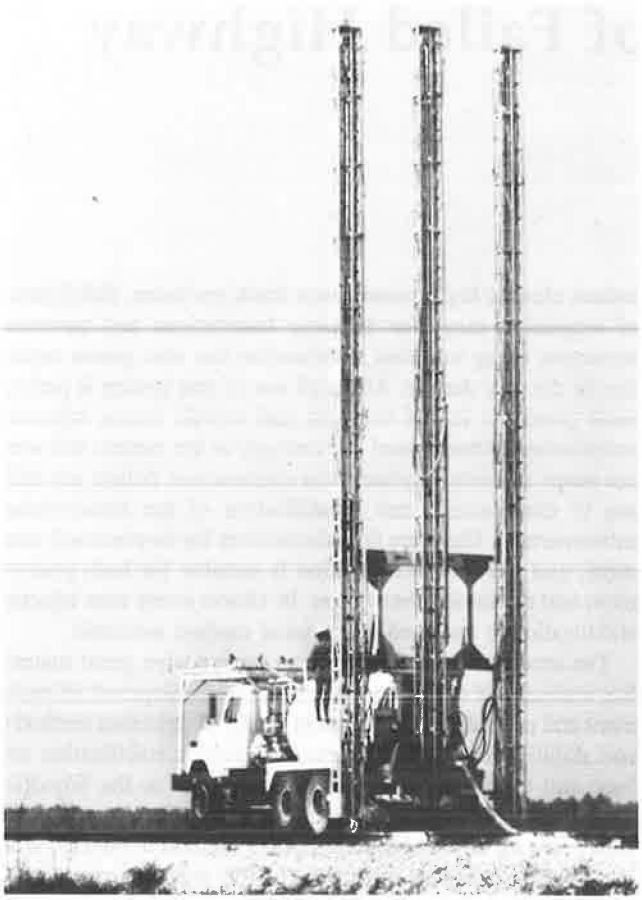

FIGURE 1 Injection truck, 40 ft.

50 to 200 psi. It will operate on railroad tracks, paved roads, or compacted surfaces. Other injection vehicles are mounted on rubber-tired or crawler tractor machines (see Figures 2 and 3) for off-road capability such as embankments or construction sites. The bulk slurry mixing tanks are usually 10 ft in diameter by 30 to 40 ft long and are used for mixing either lime or L/FA slurry. These tanks are portable and are easily transported from one site to another with a tractor truck. Some tanks (Figure 4) are equipped with high-pressure pumps so that slurry can be

FIGURE 2 Rubber-tired off-road injectio.. machine operating on an embankment.

FIGURE 3 Trac-powered injection equipment on highway embankment.

pumped directly to the off-road injection machines without going into a secondary holding tank.

The economy and convenience of jobsite lime slaking is now possible with new portable batch slakers, shown in Figure 5. Although these are relatively new to the marketplace, the system has been developed and proven over the past 5 years and is in daily use on stabilization projects. This portable batch slaker is a high-capacity lime slaker that can convert up to 25 tons of quicklime into 30 tons or more of hydrated lime slurry


FIGURE 4 Slurry mixing tank on site in Arkansas.

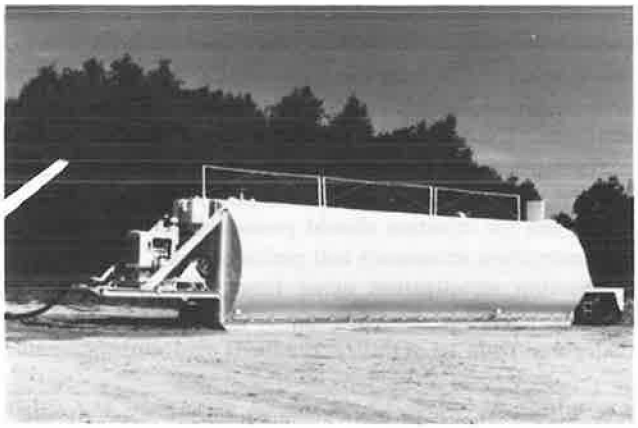

FIGURE 5 Porta batch high-capacity lime slaker.

in less than 1 hour. It is a totally enclosed, dust-free system that is simple and safe to operate.

In addition to the above mentioned equipment, there are many other ancillary pieces of equipment such as rollers, scarifiers, slurry transport trailers, slurry transfer pumps, conveyors, and vacuum material handlers that are needed to support the logistics of injection-stabilization projects.

## Injection Materials

The basic materials for injection stabilization are lime, fly ash, water, and additives. Quality control and design of slurry mixes are of prime importance. All materials should be purchased according to specifications and tested before use.

### *Lime*

In this paper the term lime refers to oxides and hydroxides of calcium. Two types of commercially available lime, calcitic quicklime (CaO) and high-calcium hydrate [$Ca(OH)_2$], are used on injection jobs. The quicklime must be slaked before mixing, whereas the hydrated lime, which comes in a dry powder form, is ready for immediate mixing. Laboratory testing can be used to indicate effectiveness of any particular commercial source of lime, but it should be emphasized that the quality of the fly ash has a much greater influence on L/FA pozzolanic reaction than does the lime. It can be stated that most commercially available limes meeting ASTM C977 are appropriate for L/FA injection if quality reactive fly ash, which meets the laboratory test series for strength and durability criteria, can be economically obtained.

The portable batching system of lime slaking allows the use of calcium oxide (quicklime) as the raw material that is converted into hydrated lime slurry at the jobsite. According to Boynton *(1)*, there are several advantages to using this system:

> Slaking quicklime at the job site with a generous excess of water improves dispersion of the hydrate particles, contributing to finer particle size and slower settling qualities.... In addition...equal importance is attached to reasonably high hydration temperature and rapid agitation in achieving fineness in particle size.... As a consequence, high surface area exerts a profound effect on chemical reactivity, settling rate, putty yield, plasticity, and the generally desired qualities of hydrates for most purposes.... The consensus among authorities is that surface area is the most reliable criterion on reactivity of hydrates; the higher this value, the greater the reactivity.

### *Fly Ash*

Fly ash is "the finely divided residue that results from the combustion of ground or powdered coal and is transported from boilers by flue gases" (ASTM Specification C593). Fly ash is collected from the flue gases by either mechanical or electrostatic precipitation devices.

Fly ash is a pozzolan and is defined as "a siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious value, but which will, in finely divided form and in the presence of moisture, economically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties" *(2)*.

### *Water*

Water used in mixing lime and L/FA slurry should be clean and free from injurious amounts of oils, acids, alkalis, salts, organic materials, or other substances that may be deleterious to the soil reactions desired. If nonpotable water is proposed for use, and if there is any doubt concerning compliance with the preceding statement, laboratory tests should be conducted to compare the reactions of similar specimens incorporating potable water.

### *Additives*

It is well known that the normal curing of L/FA slurry is dependent on time, temperature, and moisture variables. The chemically accelerated curing of L/FA grout is dependent not only on these three factors, but also on the type and amount of added chemical accelerator. L/FA accelerator can be batch mixed in a slurry tank with 20 to 30 tons of L/FA dry solids and water to increase the early strength of Type C fly ashes. Also, in some instances, an accelerator will increase the pozzolanic reactivity of Type F fly ashes so that they can be used when Type C fly ash is not available. All fly ash should be tested with proposed mix ratios in the laboratory before use. Best results are obtained by mixing the L/FA slurry continuously for 4 hours and withdrawing a sample every hour to evaluate mixing effects. Some fly ash mixtures will require an additive to retard the initial set of the slurry. Some fly ash is so reactive that it will flash set in the mixing tank and some will lose strength with continuous mixing. Proper use of retarders will delay the initial set until the slurry is pumped into the ground.

## EVALUATION OF CANDIDATE SITES

The evaluation of candidate embankment sites for injection stabilization is best accomplished through the joint efforts of highway engineers and the injection contractor. During the past 20 years, several participating research and development engineers have worked to develop soil tests for site evaluation and for predicting success of potential injection-stabilization applications. These tests are modifications of standard soil tests to measure the stabilizing effects of lime and L/FA seams and supernate penetration. As a result of these efforts a test methodology that satisfies current needs for a soil test program has been developed. The new soil tests are relatively inexpensive and straightforward so that numerous tests can be performed. They have been found to give consistent, repetitive results that can be related directly to engineering soil properties. As a rule *(a)* compression and shear strength tests should be used to evaluate sites with low-strength soils, *(b)* swell tests should be used to evaluate sites with expansive clays, and *(c)* consolidation tests should be used to evaluate sites with potential settlement problems. Other standard classification tests that give an

indirect indication of soil properties, such as Atterberg limits, are not recommended.

### Testing

Soil testing for lime injection stabilization is an important part of this technology. The testing program is used to help determine whether lime slurry pressure injection (LSPI) improves the problem sites adequately and it can also be used as a guide in preparing injection specifications. The tests provide data to help quantify the degree of site improvement that might be expected from injection stabilization; however, it is obviously not possible to obtain a one-to-one correlation between laboratory tests and field results.

Engineers have made a significant contribution to LSPI testing by developing and refining evaluation tests. These test procedures, which simulate the LSPI field condition, involve treating soil samples with lime slurry to form a glaze or seam, then curing and testing. The test results of the lime glaze and seam-stabilized test samples are then compared with test results from nontreated control samples. The amount of dry lime solids used in LSPI evaluation testing is usually 1 percent of the soil dry weight. This has been determined to be the maximum amount of dry lime injected during a single-stage LSPI injection spaced on 5-ft centers. The laboratory tests can also be used to evaluate the benefits of a second injection pass or even a third injection. The test results can then be used as input for preparing appropriate job specifications. Lime glaze and seam-stabilized samples can be used in swell, consolidation, and compression testing. This method of sample testing was developed jointly by researchers at Woodbine Corporation and the University of Arkansas. The lime-glazed and seam-stabilized method can be used with either undisturbed or remolded soil samples. As the lime-treated samples are to be compared with the untreated control samples, both will serve the purpose of evaluating lime/soil reactivity and predicting strength, swell, and stiffness improvements.

### Lime/Fly Ash Soil Testing

The purpose of the L/FA soil-testing program is to determine the potential improvement L/FA-slurry injection will produce in the candidate site and to guide in preparing appropriate specifications. These test procedures, which attempt to simulate L/FA-injection results, involve treating soil samples with the L/FA slurry to form seams, then curing and testing. Test results from the L/FA-treated samples are compared with control samples to evaluate the potential benefits of L/FA injection stabilization and with LSPI results when appropriate, to aid in selecting the most appropriate injection material.

### Investigation Plan

The investigation plan for each site should include a preliminary surface investigation followed by development of a plan for detailed subsurface investigation and laboratory tests. The subsurface investigation should be scheduled to allow ample time for sample preparation, curing, and testing. The actual injection project should not proceed until all necessary laboratory tests of soil and materials are satisfactorily completed. As presented later in this paper under case histories, failure to test the actual materials to be used can result in unsatisfactory material performance. Time and money saved by omitting necessary engineering, testing, and planning steps is soon forgotten if a material failure occurs.

## EVALUATION TEST

The laboratory testing program for injection stabilization currently uses several test procedures. These tests are described in the sections that follow.

### Glaze-Stabilized Compression Test

The glaze-stabilized compression specimen is shown in Figure 6. The purpose of the glaze-stabilized compression test is to determine the increase in sample compression strength provided by the reinforcement from the glaze-stabilized coating. The lime-glaze compression test was first reported by Blacklock *(3)*. Test samples can be prepared from either undisturbed or remolded soil. Control test samples are prepared and cured identically to the treated samples.

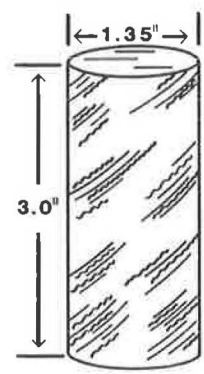


**FIGURE 6 Glaze-stabilized compression specimen.**

Detailed instructions for lime-glaze stabilized compression tests are given by Boynton and Blacklock *(4)*. The instructions for L/FA glaze are identical except for the substitution of L/FA slurry for lime slurry.

### Seam-Stabilized Compression Test

The seam-stabilized compression specimens are of two types, straight seam (Figure 7) and angle seam (Figure 8). The straight-seam sample is designed for evaluation of the compression strength-reinforcement component of the stabilized seam, and the angle-seam sample is designed for evaluation of the shear-reinforcement component of the stabilized seam. Typically, the contribution of both compression and shear will be used in repairing cracks in embankment failures. These

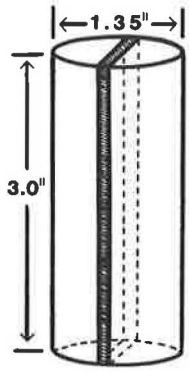


**FIGURE 7 Straight split-seam-stabilized compression specimen.**

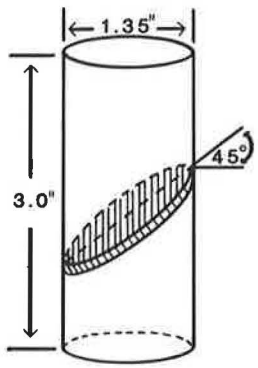


**FIGURE 8 Angle split-seam-stabilized compression specimen, shear reinforcement.**

samples can be prepared from undisturbed soil samples bu. experience indicates a preference for remolded samples. These can also be glaze-coated to allow evaluation of combinations of shear, tension, and compression strength reinforcement. Seam-stabilized compression test instructions are given by Boynton and Blacklock *(4)*.

### Glaze-Stabilized Consolidation Test

The glaze-stabilized consolidation specimen shown in Figure 9 is for the purpose of evaluating the settlement improvement provided by lime-injection stabilization of natural embankment soils. This sample is prepared by cutting undisturbed samples and then applying a glaze-stabilization coating of lime or L/FA slurry to both the top and bottom surfaces of the samples.

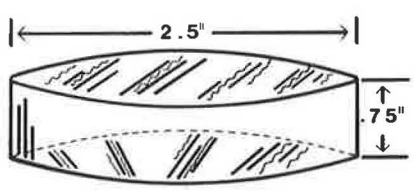


**FIGURE 9 Glaze-stabilized consolidation specimen, settlement reinforcement.**

### Seam-Stabilized Swell Test

The seam-stabilized swell specimen shown in Figure 10 is for the purpose of evaluating the swell-reduction function of lime or L/FA seams. This sample is prepared by remolding soil and placing a lime slurry seam in the center. Seam-stabilized swell test instructions are given by Boynton and Blacklock *(4)*.

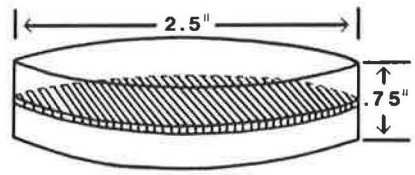


**FIGURE 10 Seam-stabilized swell specimen, expansion neutralization.**

### Material Test

In addition to the soil-stabilization tests discussed earlier, testing of all source materials is necessary. It is well known that there is considerable variation in fly ash reactivity and performance. The seam and glaze tests will help evaluate these performance properties; however, it is always best to evaluate the materials separately by performing a series of cube tests or compression cylinder tests. These tests should evaluate time, temperature, and strength variables for different mixing times, mix ratios, and material suppliers.

## EMBANKMENT FAILURE MECHANISMS

Embankment failures can be divided into two general groups, those occurring in embankments built on foundations of soft clay and silt, and those built on stiff soil foundations *(5)*. Embankments built on foundations of soft clay and silt are typified by cracks originating in the vicinity of the bottom of the interface between the fill and the top of the foundation, as shown in Figure 11; whereas, those built on hard or stiff foundations are typified by surface failures originating with surface cracks, as shown in Figure 12. A study of the origination, location, and growth pattern of embankment cracks is paramount to understanding the need for different renovation techniques because the inherent soil strength may not contribute to the stability of the embankment slope if the embankment fill is substantially cracked. Therefore, crack mending can be critical to embankment renovation.

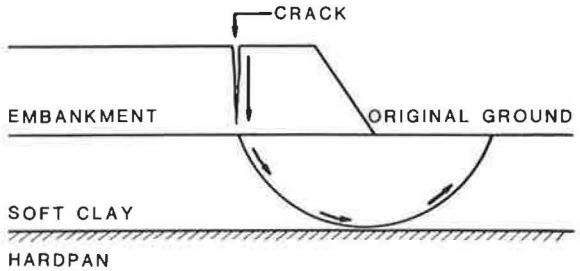


**FIGURE 11 Failure surface passing through crack in embankment.**

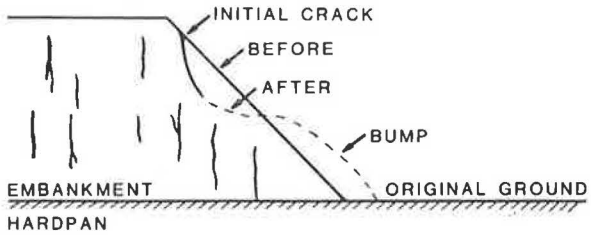


**FIGURE 12 Failure surface passing through crack.**

There are certain characteristics of lime and L/FA seams that should be recognized. Figures 13–16 illustrate the concepts of how crack repair is made by LSPI and L/FA seams, respectively. These seam-stabilization concepts were given important consideration in the design of the seam- and glaze-stabilized laboratory evaluation tests previously discussed.

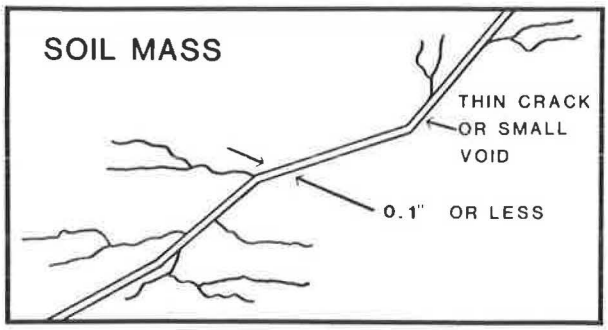


**FIGURE 13 Soil crack before stabilization with lime slurry pressure injection.**

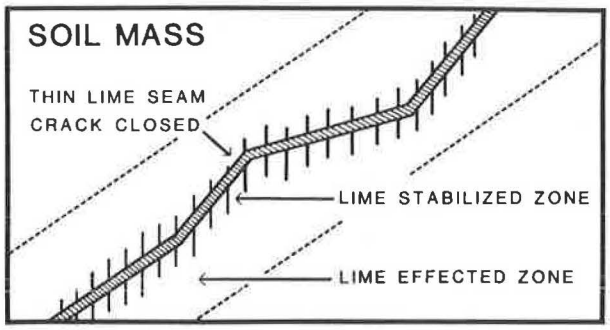


**FIGURE 14 Soil crack after stabilization with lime slurry pressure injection.**

Injection stabilization increases the strength of embankments by adding reinforcing strength and mending existing cracks, allowing peak strength of the embankment fill and the foundation subsoil to be mobilized simultaneously, thus reducing progressive failure effects. Cracks may develop in embankments because of excessive tensile stresses in the underlying fill due to differential settlements or because of shrinkage stresses due to drying. Many tension cracks frequently begin at the bottom of the fill, progress upward, and may not be detected until the embankment is seriously failing. The injection-stabilization method has been developed to treat cracks and planes of weakness in situ, even those cracks that are not visible from the surface (Figure 17). In general, cut and replace does not mend

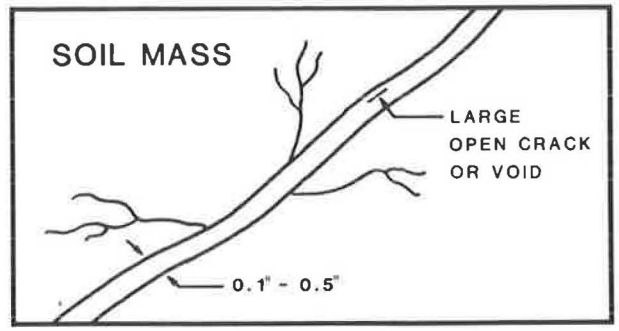


**FIGURE 15 Large soil crack before stabilization with lime/fly ash injection.**

existing tension cracks in the undisturbed mass below the cut and those cracks can continue to grow, propagating into the newly placed material. Because of the impact of embankment cracking on the stability of fills, the laboratory-testing program must evaluate the effectiveness of injection grouts for seam reinforcement, crack filling, and prevention of crack growth. Split-glazed lime or L/FA compression tests can be used to evaluate the benefits of slurry to repair cracks and increase embankment strength by adding tensile, compression, and shear reinforcing strength. Seam tests can be used to evaluate the benefits of hardened seams to stop crack growth, mend existing cracks, and prevent formation of new cracks.

Safety factors can be shown to increase rapidly with increase of fill strength. Computerized structural analysis methods can be used to analyze the strength effects of stabilized lime-soil or L/FA-soil seams, given the properties of the soil mass. Slope-

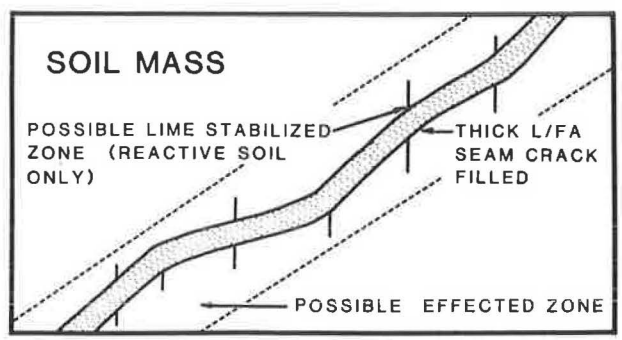


**FIGURE 16 Large crack after stabilization with lime/fly ash injection.**

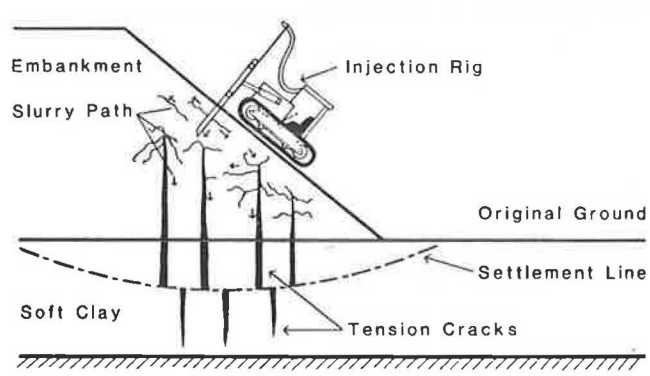


**FIGURE 17 Tension cracks filling with lime/fly ash slurry.**

stability analysis can be used to calculate safety factors for both cracked and uncracked embankment fills.

## EMBANKMENT STABILIZATION PROCEDURES

The procedures for injection stabilization of highway embankments have evolved over the past several years with changes and improvements being made during the recent highway case studies programs. The procedures currently recommended for highway embankment stabilization using injection stabilization methods are as follows:

1. The failed embankment, as shown in Figure 18, is bladed and compacted into shape and drainage is corrected.
2. The surface is proof rolled and compacted at prescribed limits of moisture and density, perpendicular to the face of the slope.

**FIGURE 18 Slope failure on highway embankment.**

3. The injection pattern is graphically planned to allow for injection variables of depth, single injection, double injection, lime or L/FA, or a combination of both. The injection sketches should then be prepared, as shown in Figures 19 and 20.
4. Each site is injected in a prescribed sequence usually beginning with the longitudinal rows at the toe of the slope and then progressing to transverse rows along the face of the slope, as shown in Figure 21. Double-injected areas require a short stage wait before injecting the diagonal off-set hole pattern.
5. Following the injection, the lime and fly ash mixture on the surface of the site is scarified, or disked, into the top 6 in. and then compacted to seal the surface.
6. Grass should be planted when the proper growing season arrives. The mixing and compaction, described in item 5 above, should prevent any excessive surface erosion from occurring until the new vegetation is established.

## CASE HISTORIES

The following three highway embankment case histories of injection-stabilization projects are presented to illustrate the adaptability and versatility of the method. Injection stabilization has many other geotechnical applications. It has been used successfully on: *(a)* railroads, *(b)* dikes and levees, *(c)* runways, *(d)* streets and parking lots, *(e)* pre-treatment, of building foundations, *(f)* construction dewatering, and *(g)* renovation of building foundations. Several of these projects have already been presented in case studies *(4, 6, 7, 8)*.

### Case 1: Evaluation of Lime/Fly Ash Pressure-Injection of an Alabama Roadway Embankment

#### *Preliminary Investigation*

The site is located in Lowndes County, Alabama, along I-65 about 30 mi south of Montgomery *(9)*. The area is located at the approximate contact of the Ripley Formation and the cretaceous prairie Bluff Chalk. Both of these formations are in the Black Belt or Black Prairie physiographic district and contain a large amount of calcium carbonate and a high percentage of smectite in the clay fraction. The slide was located in a side-hill section and was approximately 385 ft long as measured along the toe of the slope. The distressed embankment section traverses the outlet end of a roadway culvert. Once the stream discharges from the culvert the water runs parallel to the toe of the slope for approximately 300 ft; however, the significance of this orientation was not recognized until the area was cleared of undergrowth during the construction phase. This section of Alabama experiences approximately 50 in. of rainfall per year so the toe of the slope is usually wet.

#### *Design Criteria and Procedure*

Woodbine Corporation investigated the site and proposed using L/FA injection stabilization with Type C fly ash produced in Texas for the project. However, it was subsequently pointed out by Alabama Highway Department engineers that the transportation cost of shipping Type C fly ash would offset any eco-

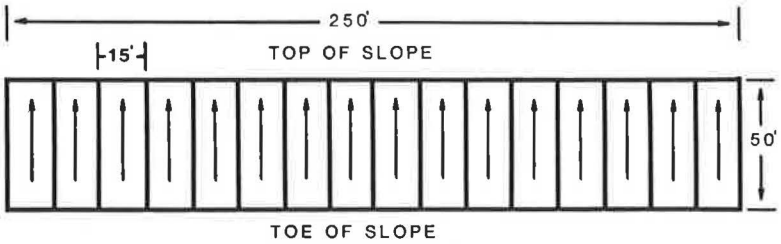


**FIGURE 19 Injection equipment progress chart.**

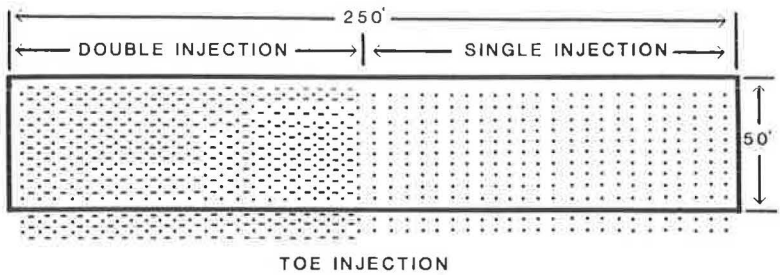


**FIGURE 20 Embankment injection patterns.**

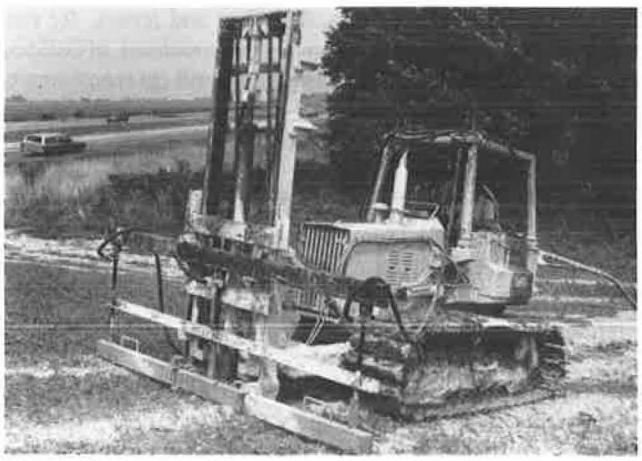

**FIGURE 21 Injection rig advancing on injection pattern.**

nomic advantage of this type of treatment. Therefore, the decision was made to use locally produced Type F fly ash, and to add 1 percent Type I portland cement to enhance the strength gain. The lime used in this project was high calcium hydrate.

### *Construction Criteria and Procedure*

State personnel prepared the site by removing vegetation and smoothing and dressing the face of the slope. This included obliterating the slide scrap and backfilling the small stream course along the toe of the slope. The culvert's headwall and wingwall were also removed (one wingwall had previously been removed by the force of a slide) to better accommodate the injection equipment. The slide area was approximately 385 ft long and the face of the slope was 105 ft long.

After the site was prepared, the contractor moved an 18,000-gal slurry mixing tank to the site. Water, purchased from the city of Greenville, was hauled to the site and pumped into the tank. Lime and fly ash were then transported in bulk to the site and pneumatically unloaded into the slurry tank. The lime was pumped into the mixing tank first to help suspend the heavier fly ash. After the slurry was sufficiently mixed it was pumped to a smaller holding tank mounted on an injection rig capable of injecting to a depth of 40 ft. The cement was added to the slurry in this holding tank. Injections were made at the edge of the pavement to depths up to 40 ft and on the face and toe of the slope to a depth of 10 ft using a crawler-tractor injection machine.

The injection pattern on the first injection was 5 ft on center and 10 ft deep on the face and toe of the slope. The injection was performed by pushing the injector rods and pumping the slurry to refusal at 18- to 24-in. intervals. Refusal was defined as that point at which slurry began to run freely at the surface from previous injection holes or from areas where the surface soils were fractured. At times the slurry would erupt from the ground more than 50 ft from the injection point. A total of 122 tons of fly ash and 45 tons of hydrated lime were initially injected. Based on the previous evaluation of the soils at the site by the contractor and the results of the laboratory tests, the decision was made to use hydrated lime slurry for the second injection. The same injection pattern was used as on the first injection, and placed diagonally between the first injection points. A total of 76 tons of hydrated lime were injected during the second injection. This work was completed in June 1983.

Throughout the injection process continued movement of the slope occurred, evidenced by the appearance of numerous surface cracks over the face of the slope, some as wide as 3 in. at the surface. Because of the continued earth movement during injection and the apparent unstable condition of the slope, Alabama Highway Department personnel elected to add a rock buttress at the toe of the slope along the stream course soon after injection, thereby disqualifying it as a viable injection-stabilization demonstration project. After the buttress was constructed, the embankment slope was benched. During this benching operation, the presence of lime seams was evident in the face of the benches. Seams were also evident in the extruded Shelby tube samples.

### *Conclusions and Recommendations*

It is the opinion of the authors that a lack of appropriate engineering material testing and site evaluation contributed to the lack of success of this project. The following points and recommendations are made:

1. The decision to change from Type C to Type F fly ash was based on economics rather than engineering principles.
2. This was not a proper site for an injection-stabilization demonstration because of the presence of the discharge culvert at the toe of the slope. This was not detected during site selection because of the heavy vegetation present.
3. It is recommended that in future research projects of this type a site be chosen that is entirely made up of embankment with no complicating structures in the immediate area.
4. The duration of the research should be for a period long

enough to allow monitoring of curing and measurement of performance over several years.

### Case 2: Arkansas Highway Earth Embankment Lime/Fly Ash Stabilization

Highway embankment surface slides are a recurring maintenance problem throughout the state of Arkansas. Approximately 65 slides occur each year in District 1 in eastern Arkansas. In this region the soil type is a clay or silty-clay alluvial river deposit of high montmorillonite content. The slope failures are usually shallow, surface-type slides 10 ft deep or less with a classical configuration. They generally do not destroy the pavement surface, but if left untreated, they can eventually lead to a complete roadway failure. Highway maintenance economics have dictated that very few of these slides can be treated to achieve correction because economical permanent methods have not been available. Thus, a majority have been temporarily repaired by simply pushing the failed material back in place. In many instances, this has resulted in repeated failures at a single site within a given construction season.

#### *Demonstration Specifics*

In 1982 a failed site was selected at the Bolling Road bridge overpassing I-40 near West Memphis, Arkansas, to be used in a federally sponsored L/FA pressure-injection demonstration project. The soil used to construct this embankment originally was a grey, highly plastic, partially organic clay, classified A-7. The slope had failed repeatedly over the past few years, including two failures that occurred in the months before injection. The principal area of failure was near the bridge abutment, where a large slide had occurred that was 8- to 10-ft deep with a ripple effect that went down to the toe of the slope (Figure 18). The length of the slope selected for the demonstration was approximately 500 ft. The entire face, toe, and cone of the slope containing a surface area of 28,778 $ft^2$ was treated with a double injection of L/FA slurry on a 1½-ft diagonal offset blanket-grid pattern. The injected mixture contained 248,700 lb of Type C fly ash and 85,600 lb of hydrated lime mixed with water and surfactant to form 101,320 gal of L/FA slurry. The total cost for injection was $14,000, which is $4.66/$yd^2$ *(10)*.

#### *Construction Procedure*

The L/FA injection procedure was conducted in stages, preceded by the highway department's maintenance forces pushing the slope back into place in order for the contractor to have a smooth workable surface. As the project began, the contractor double-injected the toe of the slope to a depth of 7 ft. The first pass was spaced on 5-ft centers and the second was spaced on the diagonals between the first injections. Next, the contractor double-injected the face of the slope to a depth of 10 ft in the same way as on the toe of the slope.

After the injection operation was completed, the L/FA slurry left on the surface of the slope was scheduled to be mixed with the soil, and recompacted by others. Later inspection visits to the site revealed that final proof rolling was never completed; therefore, no compaction was accomplished. This was required to ensure that there were no soft spots in the near-surface materials and that the surface was properly sealed. The omission of this last important step was determined to be the cause of the shallow surface slough that occurred 18 months later.

#### *Monitoring Equipment Installation*

The monitoring equipment installed by the Arkansas Highway Department after injection consisted of two slope inclinometer tubes and 10 temperature and moisture sensors for both the L/FA-injected slope and the control slope situated across the road. The moisture sensors were placed at three separate locations at various depths on both the injected and control slopes. Each sensor was to give the temperature and electrical resistance from which the soil moisture content could be determined. These sensors did not function properly and were later abandoned. The slope inclinometer tubes were installed at two locations to a depth of 40 ft on both slopes. The slope inclinometers were to measure subsurface movement that could not be detected by visual surface measurements. It is not known if any data were obtained from these installations.

#### *Evaluation Period*

The embankment performance-monitoring plan called for inspection four times a year for 5 years, including evaluations during both dry and wet periods. Inclinometer and moisture gauge readings were planned, and visual indications of surface and subsurface failures were to be noted on both the control and injected embankments. Soil samples were to be taken once a year for 5 years to note any changes from initial conditions. Conclusions and recommendations were to be based on data obtained during the yearly evaluations.

#### *Performance to Date*

The L/FA injection stabilization was performed in July 1983. On the first day the injection work was performed, a regional demonstration was held in West Memphis, Arkansas, and attended by personnel from state and federal agencies throughout the region. During the demonstration one of the maintenance engineers from Arkansas, who was familiar with the history of this slope, stated that if the slope was still standing after the next spring he would consider the job a success. The slope to be treated had failed twice in the spring before injection. Shortly after completion of the injection one of the slopes on the opposite side of the Interstate failed. This slope had been repaired at the same time the test slope and adjacent control were repaired. Early in December 1984, the control section failed. This was 18 months after injection of the test section. In January 1985, inspection of the test slope revealed that a shallow surface slough had occurred approximately 185 ft from the bridge end. The slough is 12 to 18 in. deep and extends for about 60 to 70 ft. Photographs of the treated slope showing the original area of failure before injec-

**FIGURE 22 Bolling Road embankment: primary slide zone, 18 months after lime/fly ash stabilization.**

tion and the shallow surface slough are shown in Figures 22 and 23.

In February 1985, Arkansas issued a Research Informer declaring that the injected slope had failed and concluding that the method of treatment performed at this location was not successful. However, as of this date, 2½ years after treatment, the primary failure area of the slope shows no signs of movement and is performing satisfactorily.

In the authors' opinion the shallow surface slough does not constitute a failure of this demonstration and the research project should not have been terminated. No funds were recovered because of early termination of the demonstration.

*Conclusions and Recommendations*

The following conclusions and recommendations are presented:

1. When a failed slope is pushed back into place before injection the entire face must be proof rolled or compacted to ensure that there are no soft spots or areas of loosely compacted material.
2. Immediately after injection the lime or L/FA slurry on the surface should be mixed into the top 6 in. and properly recompacted. This was not done on this job.


**FIGURE 23 Bolling Road embankment: observed shallow surface slough, March 1985.**

3. All proof rolling and compaction work should be done perpendicular to the face of the slope and never parallel to it. A long, shallow surface slough such as the one that occurred here can actually be caused by loosening the material on the downhill side by a heavy track machine running parallel to the face. This could have been done by the machine that redressed the slope just before injection.
4. Inspection of the surface slough in June 1985 revealed that the failed material contained grass and rocks and was dry and uncompacted, and the surface was still showing L/FA that had never been mixed and recompacted.
5. Deep-seated embankment slope failures can be repaired by multiple injections of L/FA slurry.
6. The evaluation of any demonstration should be continued for the full period to learn as much as possible about the process and its results. Premature abandonment does not provide necessary long-term information about the process under evaluation.
7. When any failure does occur a complete evaluation should be made so that useful new technological information can be generated to aid in future design and implementation. In the authors' opinion the purpose of any demonstration or research project should be to learn as much as possible from both successes and failures, so that future projects can benefit from the accumulated knowledge and experience.
8. All L/FA-injection projects should include the 4-hr mixing test already mentioned to evaluate quick-setting injection materials. The fly ash used on this embankment was later found to suffer excessive strength loss during the 4-hr mixing strength test. This material problem did not surface during the standard tests then used to evaluate the L/FA soil reactions.

## Case 3: Missouri Highway Earth Embankment Lime/Fly Ash Stabilization

This slide-repair project is located on Route 77, Cape Girardeau County, Missouri. The embankment had a history of previous failures and in the past several construction procedures had been tried with little success to stabilize the recurring slides. All evaluation tests on the project were conducted by the Missouri Highway Department using both standard soil tests and lime-glaze and L/FA-seam tests.

*Soil Tests*

The slide zone is composed of two soils. The fill was originally constructed of Sharkey clay, which is alluvial in origin. The Sharkey clay is a highly plastic, grey, waxy clay and was the cause, in conjunction with the steep slopes, of the slides. The demonstration section was approximately a 3 to 1 slope; however, slopes as flat as 6 to 1 have suffered slide failures in this area.

Test results on soils obtained from the fill show liquid limits exceeding 50, with the maximum value determined at 67. The Sharkey clay soils are usually lime reactive. The second soil type found within the slide zone is the Memphis soil which is a loess material. The Memphis soil was imported by maintenance personnel for slide repair. Memphis soils are also lime

reactive, but the improvements are less dramatic than in the more plastic clays.

### *Injection Procedure*

Lime was mixed into a slurry at the rate of 2½ to 3 lb/gal of water while the lime and fly ash were mixed at a 1 to 3 ratio by weight and slurried at a rate of 4 lb L/FA/gal of water. Injection pressures ranged from 50 to 200 psi. A total of 139 tons of lime and 301 tons of fly ash were injected into the 9,322 $yd^2$ embankment slope area. During the site selection and evaluation phase the decision was made to single-inject a portion of the slide on 5-ft centers and to double-inject the balance of the slide. L/FA was used for the single-injected area, and lime slurry followed by L/FA was used for the double-injected area. Because a single-injection pattern is more economical than a double one, the purpose was to evaluate the performance of both patterns to determine the most effective and economical method to use for future work.

The injection work was accomplished during the period September to October 1984. A 100-ft section was double injected and the balance of 700 ft was single injected. All injections were made to a depth of 10 ft, except for one area that was injected 12 ft deep.

### *Performance to Date*

The injection stabilization was completed in October 1984, and through December 1985, 14 months later, no movement was observed in the slope. The slope was visually inspected in November 1985 by Missouri Highway Department personnel who reported the slope to be in good shape. The guardrail was straight and no movement or tension cracks were observed.

A row of iron fence posts was set 3 ft into the ground and 4 ft above the ground, approximately ⅓ of the distance up from the bottom of the slope. No movement of these posts has been noted since installation. Site monitoring will be continued for 5 years. The three noninjected control slopes at this location failed within the first 6 months of this demonstration. Plans call for injection of these slopes in 1986.

### *Conclusions and Recommendations*

1. Both the single L/FA injection and the double combination lime and L/FA injection have prevented any recurring slides for approximately 15 months after installation.
2. Additional installations should be made to optimize the most economical, effective method to stabilize embankment slopes in this area.

## RESEARCH NEEDS

The rate of future progress in injection stabilization will be substantially increased if additional funding is allocated for technology research and development for construction renovation of highway soils. Although currently funded demonstration projects are of considerable value, they do not address basic injection-technology needs of admixture development, material characterization, full-scale strength testing, design methodology, and nondestructive site evaluation.

## SUMMARY

Injection stabilization is an emerging technology with increasing application opportunities. To date, the largest markets for injection stabilization have been stabilization of existing railroads to reduce maintenance, increase line speeds, and improve safety; and stabilization of building sites and pavement structures in expansive clays. This same injection technology developed over the past 20 years can now be applied to existing highway pavement and embankment stabilization, as well as civil construction projects, as emphasis shifts from new construction to maintenance and rehabilitation of the infrastructure.

The three completed injection projects discussed in the case histories section of this paper show specific examples of how lime and fly ash can be used to stabilize highway earth embankments. Of the three projects discussed, the Alabama project was considered unacceptable because of poor material design, improper site selection, and additional work that was performed on the slope. The Arkansas site remained standing after 2 years, with the exception of one shallow surface slough, which occurred because of a lack of adequate pre-injection compaction and post-injection mixing and compaction. The primary area of deep failure, which had failed repeatedly before injection, has resisted any further movement for 2½ years. The Missouri project is performing 100 percent successfully, with no observed movement to date.

A fourth site in northern Louisiana is scheduled for injection in 1986. This site will include injecting all four quadrants of a failed Interstate crossing. Two quadrants will be injected with lime slurry and two with L/FA slurry. The planning at this site has included recommended injection improvements and necessary construction changes supported by the generation of data from Alabama, Arkansas, and Missouri.

In addition to the three reported case histories, numerous other slides have been successfully injected for private industry and other agencies, including the Corps of Engineers, over the past 10 years.

Much progress has been made to date on the use of injection stabilization of embankments; however, improvements are needed in engineering, material testing, mix design, and soil-density control. Stabilization of surface slope failures requires good moisture and compaction control before injection coupled with post-injection mixing and compaction. It is recommended that additional demonstrations be conducted to further develop injection stabilization as a viable, economically feasible method for correcting highway embankment failures.

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# Underpinning Considerations for Design Unit A-140, Metro Rail Transit Project

DELON HAMPTON AND J. SCOTT JIN

**The proposed rail transit system, Section A-140 of the Southern California Rapid Transit District, consists of two cut-and-cover stations and approximately 1.5 mi of twin bore tunnel. Along the proposed alignment are numerous structures many of whose foundations rest above the invert of the proposed tunnels or adjacent to the proposed station excavations. Consideration is given to protection of structures along the proposed route. The influence zones for tunnel mining and station excavation, based on design criteria, available literature, and past experience, are established. Next, settlements of the buildings within the influence zone are predicted and compared with the estimated allowable settlements. Those buildings whose predicted settlements exceed allowable settlements are thereby identified. Technically sound and economically feasible underpinning methods are considered for protection of those structures whose predicted settlement exceeds the allowable settlement, and the most effective underpinning scheme is proposed for each structure. Finally, the current project status is briefly outlined.**

The Southern California Rapid Transit District (SCRTD) is in the process of building a rail transit system to serve the people of metropolitan Los Angeles. The initial line (see Figure 1) will begin at Union Station, travel west, pass the Civic Center and the Jewelry Mart and then travel north, approximately parallel to Wilshire Boulevard, to the San Fernando Valley, a distance of approximately 18.5 mi.

Section A-140, of the proposed rail transit system, the subject of this paper, consists of two cut-and-cover stations and almost 1.5 mi of twin bore tunnel. It begins at approximately Station AR 112+30 in the Union Station parking area and extends to approximately Station AR 199+47 in the vicinity of the intersection of 7th and Hope Streets. The approximate locations of each major type of construction are given in Table 1.

Along the proposed alignment are numerous structures many of whose foundations rest above the invert of the proposed tunnels or within the zone of influence of the proposed station excavations. Therefore, consideration has to be given to protection of structures along the route. The purpose of this paper is to discuss the options considered for protecting these structures.

## SUBSURFACE CONDITIONS

In general terms, the subsurface conditions along Design Unit A-140 consist of alluvium over weak claystones and siltstones *(1, 2)*. The general subsurface conditions are shown in Figure 2. The alluvium largely consists of clean sands and gravels, but may also contain some silt, clay, and boulders. The thickness of

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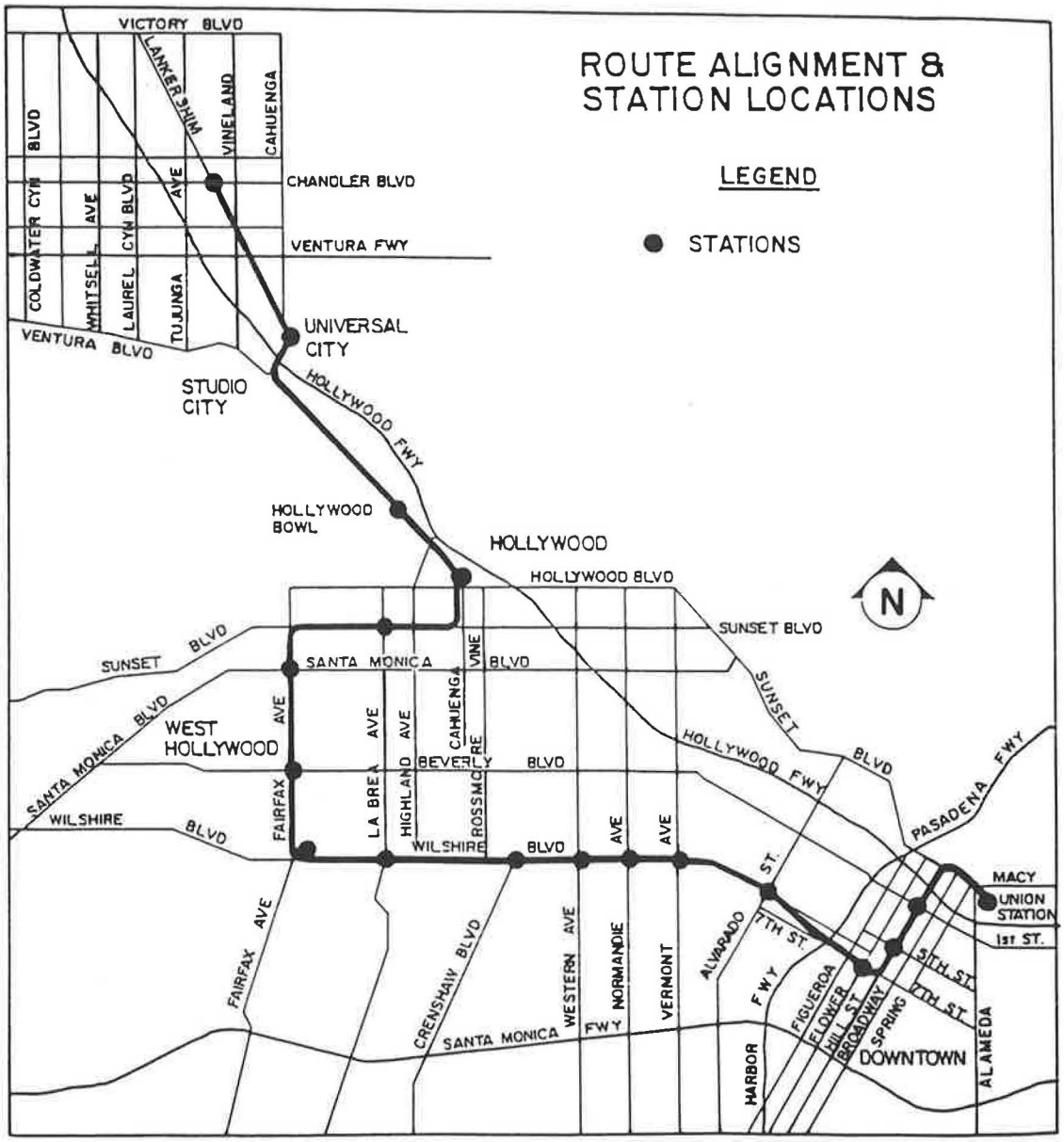


**FIGURE 1 Schematic alignment.**

the alluvium ranges from less than 10 ft to more than 100 ft within the limits of Section A-140. The variation in thickness of alluvium is related to the alignment within the margin of the Los Angeles basin. In general, the regional groundwater occurs within the bedrock. However, local areas of perched water can occur within the alluvium. These areas can include a substantial thickness of saturated soil.

The soil and bedrock materials have been categorized by name. The named units include:

1. The young alluvium, or granular alluvium, which consists primarily of clean sands and gravels with numerous layers of sandy silt, sandy clay, and silt. It also contains boulders up to 4 ft in diameter. The compactness ranges from loose to very dense.
2. The old alluvium, or fine-grained alluvium, which consists primarily of silts and clayey silts. The consistency varies from soft to stiff.
3. Bedrock, of the Fernando and Puente Formations, which consists of claystone and siltstone. The bedrock possesses a strength that would characterize it not as a rock but as a stiff to hard soil.

Two channels, tributaries of the Los Angeles River, will be traversed by the tunnel line. These channels contain both young and old alluvium, but at the depth of the tunnels only the young alluvium will be encountered. The first is between Stations 113 and 125, and the other is between Stations 177 and 200. Weak bedrock of the Puente and Fernando Formations will be encountered in the tunnel line from approximately Stations 125 to 177. Mixed-face conditions, that is, alluvium and weak bedrock contacts, should be anticipated in the vicinity of Stations 125 and 177.

TABLE 1 OUTLINE OF APPROXIMATE LOCATIONS OF MAJOR TYPES OF CONSTRUCTION

| AR STATION | | TYPE OF CONSTRUCTION | LENGTH (FT) |
|---|---|---|---|
| FROM | TO | | |
| 112+30 | 146+53 | TWIN-BORE TUNNEL | 3,423 |
| 146+53 | 152+48 | CUT-AND-COVER STATION (CIVIC CENTER) | 595 |
| 152+48 | 170+00 | TWIN-BORE TUNNEL | 1,752 |
| 170+00 | 178+21 | CUT-AND-COVER STATION (5th/HILL) | 821 |
| 178+21 | 199+47 | TWIN-BORE TUNNEL | 2,126 |
| TOTAL | | | 8,717 |

At the two station sites, the alluvium is generally less than 30 ft thick. Within the northern two-thirds of the Civic Center Station, artificial fill, 2 to 3 ft thick, directly overlies fresh bedrock. At the south end of the station, the bedrock surface drops off steeply and a zone of weathered bedrock thickens to the southeast. In this area, the weathered bedrock varies in thickness from about 8 to 17 ft and is overlain by young alluvium and fill. At the extreme southeast corner of the station, the fill is approximately 18 ft thick and the old alluvium is about 5 ft thick.

The 5th and Hill Street Station is located on the edge of the alluvial basin with the surface of the bedrock forming a subsurface ridge dropping off sharply toward the southeast. The depth of bedrock ranges from approximately 20 ft under the northern two-thirds of the station to as much as 50 ft south of 5th Street. There may also be as much as a 10-ft drop in the bedrock surface across the width of the station excavation. Overlying the bedrock is the young alluvium with a thin veneer of fill.

The young alluvium is the material that will be most problematical for tunneling, particularly where the tunnels pass beneath, or alongside, foundations of buildings. The ground here is expected to be slow ravelling, with fast raveling to running ground to be found in lenses of cohesionless sand.

## EFFECT ON ADJACENT STRUCTURES

### Underpinning Guidelines

The basic criteria to determine the zone of influence as a result of station construction are proposed in Figure 3. Fifteen buildings were found to be located within the defined influence zone (Table 2).

Underpinning guidelines for tunnel construction are shown in Figures 4 and 5. Application of these guidelines resulted in more than 30 buildings deemed to be within the zone of influence of tunneling and, therefore, considered to be at risk from ground movement. The most economically important buildings located within the defined influence zone are given in Table 3. The necessity of protective measures for these struc-

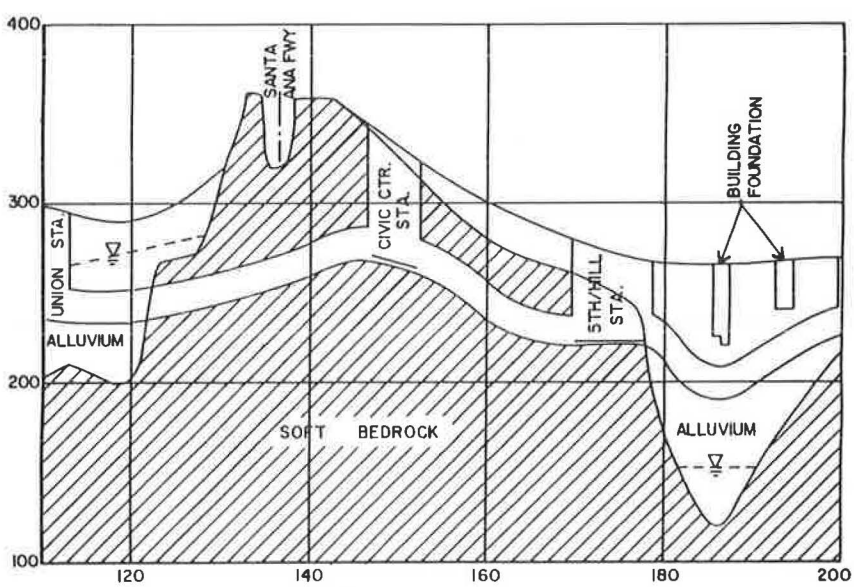


FIGURE 2 General subsurface conditions.

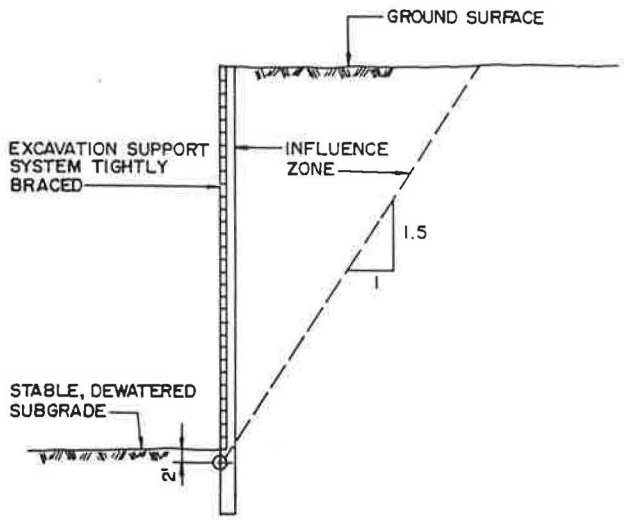


**FIGURE 3 Underpinning guidelines for station construction.**

tures will be evaluated, subsequently, based on their subsurface conditions, estimated and allowable settlements, and economic value.

## Allowable Settlement of Buildings

The allowable building settlement criteria contained herein are based on work performed by Skempton and MacDonald *(3)* and by Polshin and Tokar *(4)*. Skempton and MacDonald summarized settlement and damage observations on 98 buildings, 40 of which exhibited signs of damage. The study included both steel and reinforced concrete frame structures and structures with load-bearing walls. Buildings supported on spread footings, mats, and piles were included in the study. Skempton and MacDonald used angular distortion as their criterion for damage, as most of the damage appeared to be related to distortional deformations. Angular distortion was defined as the ratio of the differential settlement between two points ($\delta$) to the distance separating the two points (L). It was concluded that cracking of load-bearing walls or panel walls in frame structures will occur when $\delta/L$ exceeds 1/300 (0.33 percent) and that structural damage is probable when $\delta/L$ exceeds 1/150 (0.67 percent). Skempton and MacDonald recommended $\delta/L$ = 1/500 (0.2 percent) as a design criterion to provide an adequate factor of safety against damage due to settlement in buildings. The Skempton and MacDonald criteria were subsequently incorporated into recommendations that relate the magnitude of $\delta/L$ to various types of damage by Bjerrum *(5)*.

In a more recent study, Grant et al. *(6)* reviewed the settlement and damage data reported by Skempton and MacDonald in conjunction with data on an additional 95 buildings, 56 of which reportedly suffered some damage. This study supports the Skempton and MacDonald conclusion that cracking of walls can be anticipated when $\delta/L$ exceeds 1/300 (0.33 percent). Furthermore, Grant et al. recommended criteria for architectural damage based on the deflection ratio. The deflection ratio is defined as the maximum displacement, $\Delta$, relative to a straight line between two points to the distance, L, separating the two points. According to the Grant et al. study, cracks in panel walls of frame buildings and load-bearing walls are likely to occur if the deflection ratio exceeds 1/100 (1.0 percent). Note that this ratio is equivalent to an angular distortion of 1/500.

**TABLE 2 STRUCTURES LOCATED WITHIN THE INFLUENCE ZONE OF STATION CONSTRUCTION**

| APPROXIMATE STATION NO. | GROUND TYPE | CONSTRUCTION TYPE | NUMBER OF STORIES | RELATIVE VALUE | REMARKS |
|---|---|---|---|---|---|
| AR 146 + 75 | Rock | Flexible | 3 | M | Court of Flags |
| AR 146 + 75 | Rock | Flexible | 2 | M | County Mall Parking |
| AR 150 + 00 | Rock | Rigid | 6 | H | County Courthouse |
| AR 150 + 00 | Rock | Flexible | 2 | M | County Law Library Parking |
| AR 173 + 25 | Rock | Rigid | 12 | H [a] | Clark Hotel |
| AR 173 + 25 | Rock | Rigid | 12 | H | Subway Terminal Bldg. |
| AR 174 + 50 | Soil | Rigid | 2 | L | Abandoned Commercial |
| AR 175 + 00 | Soil | Rigid | 10 | M | Commercial/Warehouse |
| AR 175 + 00 | Soil | Rigid | 3 | L | Zody's |
| AR 175 + 50 | Soil | Rigid | 2 | L | American Barber College |
| AR 175 + 75 | Soil | Rigid | 2 | L | Pussycat Theater |
| AR 176 + 25 | Soil | Flexible | 1 | L | Kal's Burger |
| AR 176 + 50 | Soil | Rigid | 3 | H | Pershing Square Bldg. |
| AR 176 + 75 | Soil | Rigid | 12 | H | Thrifty Drug |
| AR 178 + 00 | Soil | Flexible | 3 | M | Pershing Square Parking |

Note: H = high, M = medium, and L = low.
[a]High value due to status as historic landmark building, only.

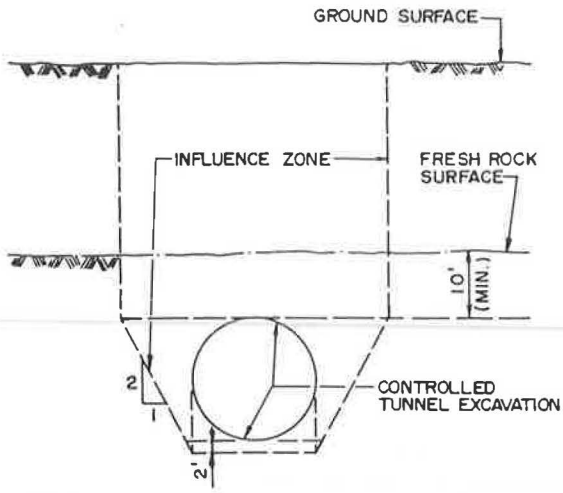


**FIGURE 4 Underpinning guidelines for tunnel construction in rock.**

Polshin and Tokar *(4)* discussed allowable deformations and settlements and defined criteria similar to those of Skempton and MacDonald. In this study, Polshin and Tokar treated frame structures and load-bearing walls separately. However, their limiting values of angular distortion for frame structures ranged from 1/500 to 1/200 and are approximately the same as those of Skempton and MacDonald.

Polshin and Tokar's treatment of load-bearing walls introduced the concept of allowable settlement defined in terms of the deflection ratio, and assured a relationship between maximum allowable deflection ratio and a critical level of tensile strain in the wall. Using this concept, the deflection ratio at which cracking occurs in a (brick) wall is related theoretically to the length-to-height ratio of the wall. A larger deflection ratio is allowed for structures on plastic clay than on sand or stiff clay. It is presumed that the slower rate of settlement allows time for creep within the structure, thus increasing the level of tensile strain, and therefore increasing the deflection ratio at which cracking begins. It should be noted that the critical tensile strain applies only to visible damage and not to structural damage.

In summary, the essential criteria used to define allowable settlement of buildings are based on the works of Skempton and MacDonald, and Polshin and Tokar. Extensions of these works were provided by Grant et al., and Burland and Wroth *(7)*. From these works, general conclusions on allowable settlements may be drawn as follows:

1. For frame structures with panel walls,
   *(a)* An angular distortion of 1/500 is allowable for buildings in which cracking is not acceptable,
   *(b)* An angular distortion of 1/300 is allowable in buildings in which some cracking is acceptable, and
   *(c)* An angular distortion of 1/150 is allowable in buildings in which severe cracking is acceptable but structural damage is not acceptable.
2. For load-bearing walls, the allowable differential settlement is reduced to 75 percent of that for frame structures.
3. The allowable differential settlement is reduced to 75 percent of the values in Items 1 and 2 when the settlement pattern is concave downward.

## SETTLEMENT ESTIMATES

Estimates of settlements of foundations within the zone of influence were made. These estimates were based principally on the 1975 paper by Cording and Hansmire *(8)*, which relied heavily on a paper by Peck *(9)*. Settlements for existing conditions were made assuming 3 percent ground loss as the best estimate, based on a review of available literature of reported case histories for similar ground conditions, the estimated settlement trough, and the geometric position of each particular foundation within the zone of influence of the settlement trough.

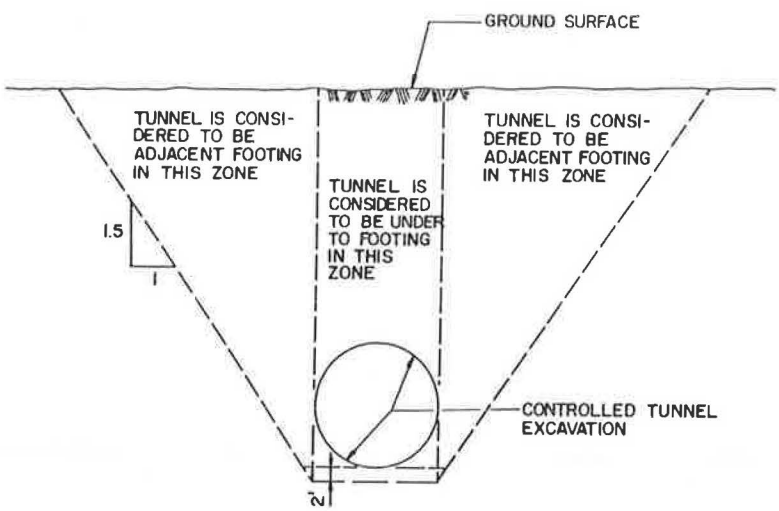


**FIGURE 5 Underpinning guidelines for tunnel construction in soft ground.**

**TABLE 3 STRUCTURES LOCATED WITHIN THE INFLUENCE ZONE OF TUNNEL CONSTRUCTION**

| APPROXIMATE STATION NO. | GROUND TYPE | CONSTRUCTION TYPE | NUMBER OF STORIES | RELATIVE VALUE | TUNNEL LOCATION | REMARKS |
|---|---|---|---|---|---|---|
| AR 167 + 25 | Rock | Rigid | 4 | M | Adjacent | Myrick Hotel |
| AR 181 + 00 | Soil | Rigid | 14 | H | Adjacent | International Jewelry Ctr. |
| AR 182 + 00 | Soil | Flexible | 3 | M | Under | Pershing Square Garage |
| AR 185 + 00 | Soil | Rigid | 9 | H | Under | Jewelry Mart |
| AR 186 + 00 | Soil | Rigid | 14 | H | Under | Park Center Bldg. |
| AR 187 + 50 | Soil | Rigid | 12 | H | Adjacent | L.A. Jewelry Center |
| AR 189 + 00 | Soil | Flexible | 5 | M | Under | Athletic Club Parking |
| AR 189 + 75 | Soil | Rigid | 10 | H | Adjacent | LA Athletic Club |
| AR 190 + 50 | Soil | Rigid | 9 | H | Under | Olive Center Bldg. |
| AR 191 + 00 | Soil | Rigid | 14 | H | Under | Bank of America Bldg. |
| AR 192 + 00 | Soil | Rigid | 4 | M | Under | Clifton's |
| AR 192 + 25 | Soil | Rigid | 4 | M | Under | Okada Restaurant |
| AR 193 + 00 | Soil | Rigid | 13 | H | Under | Brack Shop Bldg. |
| AR 193 + 75 | Soil | Rigid | 13 | H | Under | Quindy Bldg. |
| AR 195 + 00 | Soil | Rigid | 21 | H | Adjacent | Wilshire Grand Bldg. |
| AR 196 + 50 | Soil | Rigid | 11 | H | Adjacent | Robinson Dept. Store |
| AR 197 + 00 | Soil | Rigid | 12 | H | Adjacent | Kyowa Bank |
| AR 199 + 00 | Soil | Rigid | 8 | H | Adjacent | Central Bank |

Note: H = high and M = medium.

## METHOD FOR CONTROL OF SETTLEMENT

### Buildings Adjacent to Station Excavation

#### *Techniques Available*

Four methods feasible for underpinning structures adjacent to station construction are *(a)* jacked piles, *(b)* slant-drilled piles, *(c)* hand-dug pit, and *(d)* column pickup. Other methods that may be considered feasible for protecting such structures are compaction and consolidation grouting.

In the column pickup method, foundation loads are not transferred below the influence zone to a lower stratum; rather specific structural elements are releveled in the event that excessive settlements occur.

In the hand-dug pit method, a pit is excavated below the influence zone to an alternative bearing stratum. Because the distance from footings to the bottom of the influence zone is approximately 40 ft for most buildings near stations, this method is not considered feasible for the situations under consideration.

Both jacked piles and slant-drilled piles are considered feasible for underpinning structures adjacent to station construction. As both methods will carry the column loads to a lower stratum, their effectiveness is practically independent of the performance of the excavation support system. Also, both jacked piles and drilled piles can be preloaded to minimize potential settlements. The cost of these two techniques is relatively high, and accessibility to the bottom of the foundation that needs underpinning may be questionable.

Compaction and consolidation grouting techniques have been used in subway station construction in Baltimore and in Pittsburgh to underpin adjacent buildings. These techniques are relatively economical and direct access to the bottom of the foundation is not required. They are not positive structural underpinning methods and should be used in conjunction with a conservative soldier pile wall. Also, quality control of the grouting operation, as well as an adequate instrumentation program, are required to ensure the effectiveness of this technique.

#### *Predicted Settlement of Buildings*

The need to underpin, and the selection of the appropriate type of underpinning for specific buildings located adjacent to station excavations, depend on many factors. Each structure must be evaluated independently. However, the basic approach was to select a relatively rigid excavation support system that would minimize ground movement to such a low value that the need to underpin adjacent buildings would be reduced or eliminated.

If a conservative soldier pile wall is constructed as proposed, the maximum lateral and vertical movements should be limited to approximately 0.1 percent of the excavation depth; the angu-

lar rotation of adjacent buildings caused by ground movements should be approximately 1 to 1,000. Thus, a 60-ft excavation would result in approximately 0.75 in. of settlement over a distance of approximately 60 ft. However, the adjacent buildings are approximately 25 to 50 ft away from the excavation. The angular rotation of adjacent buildings may be greater than 1 to 1,000 when these buildings are located approximately 10 ft from the face of excavation. There are four high-rise buildings (Subway Terminal Building, Clark Hotel, Thrifty Drug, and Pershing Square Building) in the conditions previously described, along the 5th and Hill Street Station excavation.

Installation of a properly designed and adequately constructed slurry wall would also likely eliminate the need for underpinning of existing buildings close to the station excavation. Therefore, the installation of a slurry wall is considered to be equivalent to the conservative soldier pile wall as a method for control of settlement.

## Buildings Along Tunnel Line

### *Techniques Available*

There are four classes of techniques available for reduction of settlements caused by nearby tunneling:

1. Control of tunneling practices,
2. Consolidation grouting,
3. Compaction grouting, and
4. Underpinning.

**Control of Tunneling Practices** Refers to the limitation of loss of ground during tunneling; this is somewhat analogous to providing a conservative soldier pile or slurry wall in open cuts. It may be accomplished by exercise of a combination of controls including

1. Careful metering of spoil to assure that its volume corresponds to the advance of the bore,
2. Installation of a noncompressible lining, and
3. Rapid backfilling and grouting of the space between the lining and excavated surface.

All of these operations would normally be executed and controlled to a certain extent regardless of whether the tunnel lies adjacent to or beneath a structure subject to settlement. The additional effort required to exercise careful control is considered to have a negligible effect on the cost and progress of the work. Therefore, it is taken for granted that these controls will be adopted in the construction contract and that they will be carefully enforced.

**Consolidation Grouting** Consists of the injection of chemical grout into the voids in the soil ahead of, above, and to the sides of the tunnel. Its purpose is to create a zone of soil with sufficient strength to distribute the load from an overlying footing to columns of grouted soil in the sidewalls of the tunnel.

It is doubtful that the grouted soil would possess sufficient strength to carry the load of an overlying column to the grouted soil columns adjacent to the tunnel. The function of the grouting is to lengthen the time for the subsidence of the soil into the annular space around the shield and lining, thus providing time for the cavity to be grouted by routine contact grouting operations.

Consolidation grouting can be executed from the ground surface (through holes inclined so as to reach underneath buildings), from basements, from a completed adjacent tunnel, or from the heading of the tunnel itself. Grouting from the tunnel heading is extremely disruptive to ongoing tunneling operations and would normally be used only when access from other locations is impossible. Because of the alternative locations from which the work can be done, consolidation grouting lends itself well to reduction of settlement under the conditions that exist in Design Unit A-140.

Another advantage of consolidation grouting is that it will cause essentially no disruption to current use of buildings whenever work can be executed from outside the building or from within an adjacent tunnel or the tunnel under construction. At other times, when the work must be done from basements, the materials are relatively clean and amenable to containment, and the equipment is small enough to be handled easily.

**Compaction or Displacement Grouting** Consists of the injection of a stiff grout or mortar into the ground at sufficiently high pressure to displace the soil and cause heave of overlying materials. The use of this method here would not be to initiate heave but rather to arrest the settlement or actually raise a footing or column that has started to settle.

The principal advantage of compaction grouting is that it can be called on for use when needed. It is therefore economical in the sense that costs are expended only where instrumentation has indicated that preventive or remedial work is actually needed. Its disadvantage is that it must usually be done from the basements of buildings, thereby disrupting day-to-day use.

**Underpinning** Consists of the installation of structural support beneath a column or footing in order to carry the load to an area below the zone of influence of the tunneling operations. Four techniques are used for installing underpinning:

1. Jack piles: costly; require that construction operations be conducted within the building; are of little practical value when the tunnel passes directly beneath the column to be supported.
2. Slant piles: must generally be driven by standard percussive techniques. This operation must often be carried out from within basements of buildings below which tunnels must pass.
3. Hand-dug pits: like jack piles, are useful only when the underpinning can be extended vertically downward from the column to be supported. This is not generally practical for tunnels passing directly beneath structures.
4. Column pickups: require no work to be performed beneath the existing footings, which is a significant advantage over the aforementioned methods of underpinning. The installation and operation of column pickups does, however,

require work to be done in basements of buildings. Although the equipment for this work is small, and would cause less disruption of other uses of the building than would be caused by pile driving or jacking, it is the opinion of the authors that the use of column pickups would have little practical application here.

## CONCLUSIONS

Extensive discussions took place within and among the design teams concerning the best approach to use to protect the structures along the proposed alignment for Section A-140. In general, these discussions can be summarized as follows:

1. Good tunneling practices must be required and enforced through compliance with a well-documented tunneling specification.
2. Structural underpinning methods are economically infeasible and would cause too much disruption to building occupants.
3. It was agreed that 20 structures identified as needing protection during construction could be adequately protected using grouting techniques.
4. For six buildings (Jewelry Mart, Park Center Building, Oliver Center Building, Bank of America, Brack Shop Building and Quinby Building), where the proposed tunnel is directly under the foundation with only one- to one-and-a-half-diameter clearance, the designer and the General Engineering Consultants have not reached agreement on the technique to be used to underpin these structures.

## ACKNOWLEDGMENTS

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# Mitigating Artesian Water Flow by Pressure Grouting on U.S. 101 in San Jose, California

KEN JACKURA AND ED GRAF

**Severe wet weather cycles in northern California during the winters of 1981 to 1982 and 1982 to 1983 created near-historic high groundwater levels in at least one area of San Jose where a depressed portion of U.S. Route 101 exists. The high groundwater level induced artesian flow onto U.S. 101 in excess of 1,100 gal/min. The flow led to piping of fines from underneath the freeway, slab uplift in excess of 10 in., and buoyancy of an underground water storage vault. Installation of temporary dewatering wells and implementation of an effective grouting program and chemically grouted structure tie-down anchors stabilized the area without shutting down traffic.**

Over the past 30 years, the California Department of Transportation (Caltrans) has designed and constructed a number of depressed freeways through urban areas to balance cut and fill, reduce its visual impact, lower costs, provide for cross-traffic flow on unelevated bridge overcrossings, and more recently reduce traffic noise. High groundwater tables required special design and construction techniques to prevent bouyancy of underground structures and to mitigate the impact of seepage pressures on slopes and the roadway.

Discussed in this paper are problems associated with a rising groundwater pressure within a confined aquifer lying immediately below a depressed section of U.S. 101 in San Jose, California. The aquifer pressure head was low before construction and had been low for years due to below-normal rainfall, heavy agricultural pumping, and domestic water needs. Since 1970, increased urbanization and receding agricultural development have resulted in less shallow groundwater withdrawal. Coupling this with the heavy rains of 1981 to 1983, groundwater levels and the recharging of the confined aquifer are approaching a historic high.

Problems in the depressed section became apparent in early March 1983 when Caltrans' maintenance crews noticed water spouts emanating from around the median timber posts separating the northbound and southbound lanes. These water spouts, estimated at 3 ft high, were distracting motorists and affecting travel.

A 10- to 12-in. slab uplift in the southbound concrete pavement was also observed and water was streaming from around a pump house adjacent to the southbound lanes. Further investigation revealed bouyancy of an underground storm-water collection box and washing of fines.

The immediate problems to be solved were

1. Lowering the water table,
2. Sealing the upward flow of water where an impervious soil barrier above the aquifer had been pierced,
3. Anchoring buried structures to resist imminent flotation, and
4. Filling of voids caused by the washing of fines from under and around the buried structures.

In addition to these immediate problems, this section of U.S. 101 is the major north-south corridor feeding the San Francisco and Silicon Valley areas and is located in a highly urbanized district. Closing only one of its six lanes would cause a backup during midday and an intolerable situation during commuting hours. One of the underground structures, a concrete cistern, was completely under all of the southbound lanes. Quick installation of five dewatering wells followed by pressure grouting and structure tie-down anchors resulted in stabilizing the areas without shutting down traffic.

## PROJECT LOCATION AND GEOLOGY

The limits of the depressed section, which is located at the northern end of the city of San Jose at the junction of U.S. 101 and State Route 17, are shown in Figure 1. Original ground elevation was 53 ft, whereas finished grade elevation within the depressed portion is 33 ft, a finished cut of about 20 ft below adjacent ground.

Geologically the site lies within the Santa Clara Valley, which is a large structural trough extending from Hollister (south of San Jose) to San Francisco (north of San Jose). Unconsolidated alluvial and bay deposits of clay, sand, and gravel fill this trough and make up the valley floor. The subject site area is bounded on the east and west by the Coyote River and the Guadalupe River, respectively. During their historical development, layers of sand and gravel and then finer-grained materials were laid in alternating sequence to form confined water-bearing aquifers.

Four distinct aquifers lie below the site. The upper or most shallow aquifer, which is of primary concern in this project, is approximately 30 ft thick and starts at a depth of about 40 ft below original ground surface. Sediments overlying the aquifer in this area are impervious and are composed of clays and clay-silt mixtures.

Underlying this upper aquifer are three others at depths of about 150, 350, and 550 ft. All are significant water-bearing strata. The aquifers are separated by impervious boundaries of clays, mixtures of clay, silts, sands, and cobbles. A generalized cross section of the subsurface conditions is shown in Figure 2. The illustration has no scale, but provides a general idea of the underground conditions showing water transport and recharge behavior. Recharge of the aquifer systems is primarily through the Santa Clara formation lying west of the site. Some recharge is also developed through infiltration and is most predominant in the uppermost aquifer.

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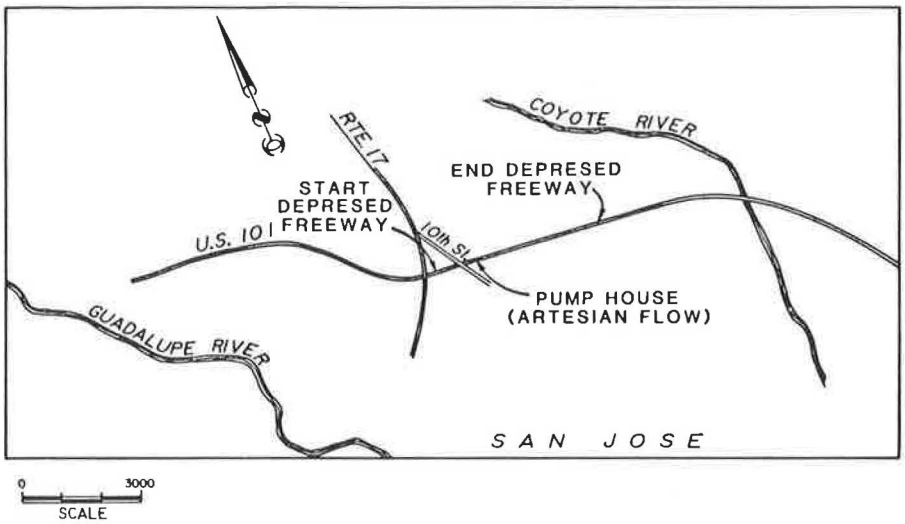


**FIGURE 1 Plan map for a portion of U.S. 101 in San Jose showing location of artesian flow.**

## Original Construction

The depressed highway section of U.S. 101 was constructed over a 1½-year period ending in July 1960. The original contract called for widening old State Route 68 (now U.S. 101) to freeway standards for a distance of about 3 mi. A pump house, storm water storage box (cistern), one major interchange, a number of overcrossings, and reconstruction of old State Route 69 (now State Route 17) crossing U.S. 101 were part of the contract. The depressed section is 1 mi long.

Borings conducted in the early 1950s revealed silts and clays to a depth of 40 ft within the major portion of the depressed section, as illustrated on the profile map (Figure 3).

A perched static water table during that time was at elevation +43 ft, about 11 ft above finished profile grade in the depressed area. Groundwater studies conducted in the mid-1950s indicated that lateral drainage into the depressed freeway would be from 1,500 to 3,000 gal/min (gpm). Most of this water would be coming from the depressed portion of the freeway where a 2- to 7-ft-thick clayey silty sand layer exists near elevation +36. Due to the rather slow draining nature of the soils, freeway design incorporating interceptor trenches along both sides of the freeway and a pervious 2-ft-thick sand blanket over the bottom of the cut was considered adequate to intercept and transport the perched groundwater.

Collected groundwater and storm water runoff was to be handled by a pump house and a 50-ft × 50-ft × 8-ft-high cistern as shown in Figure 4. Uplift pressures on the pump house and cistern were developed on the assumption that the phreatic line would be no higher than 2 ft below the finished profile grade elevation of 31 ft. Hence, buoyancy of these features was predicated on that assumption.

Pump house construction began before freeway excavation by excavating within the confines of sheet pilings. Pile sheet tip

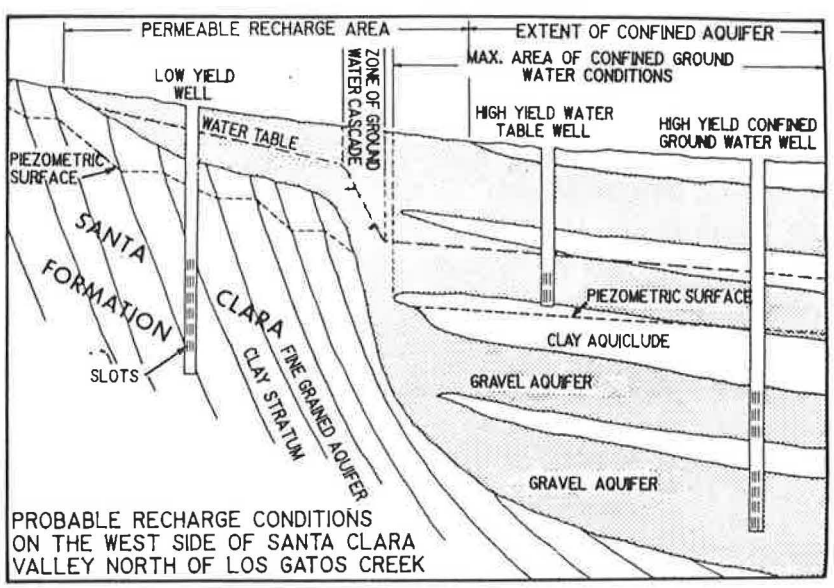


**FIGURE 2 Generalized soil cross section of northwestern San Jose area showing aquifers and predominant method of natural recharge through the Santa Clara formation.**

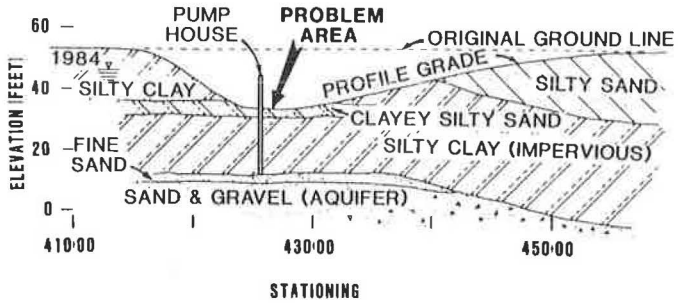


**FIGURE 3 U.S. 101 profile of grade and soil types between Stations 410 and 460.**

elevation was about 10+ ft or near the contact of a 3-ft fine sand layer lying immediately above the sand and gravel aquifer. Freeway excavation proceeded simultaneously with construction of the pump house.

Following full freeway excavation, construction of the cistern began. Material was excavated to elevation +21 ft (about 12 ft below grade) for construction of the cistern. This left about 9 ft of impervious material above the aquifer, but as this excavation was directly connected to the pump house excavation, it essentially punctured the impervious material.

Only minor sump pumping was necessary to maintain a water-free working platform in the excavations for the pump house and cistern. A stable working platform was constructed by placing a 6- to 12-in.-perimeter crushed-gravel base at the bottom of the excavations.

After the construction of the pump house and cistern, a clean pea-gravel backfill was used to fill the approximate 2- to 3-ft space between soil and structure.

### Artesian flow

In the early spring of 1983, maintenance crews noticed 3-ft-high water spouts emanating from around several timber median posts separating the northbound and southbound lanes of U.S. 101. Further investigation revealed a bubble type of slab uplift of about 10 to 12 in. over several hundred feet of the southbound lanes and occurring in the area overlying the underground cistern.

Believing that a water main had burst, Caltrans workers contacted city maintenance personnel for verification, while other Caltrans crews started relieving the hydrostatic pressure by trenching along the southbound lane shoulder. Trenching produced immediate relief of the slab uplift pressure by releasing large water flows estimated in excess of 1,000 gpm (Figure 5). Simultaneously, several workers descended into the cistern for inspection. Water flows into the cistern from the original drainage system were negligible and the crews decided to core into the 18-in. thick concrete walls to help relieve the large surface flows. Sixteen 2-in.-diameter core holes were drilled during the next several days; each hole produced large streams of water estimated at between 50 to 100 gpm.

Because of the high velocities of the discharging water, another problem developed rapidly: scouring of fines from beneath the pavement and, presumably, beneath the pump house and cistern. Two days after the initial trenching to relieve the slab uplift pressure had been conducted, pavement sag started developing in the inner two fast lanes of the northbound traffic flow and the southbound fast lane. During the following 4 weeks, in excess of 190 tons of asphalt concrete had to be placed over the sagging area, resembling a trough approximately 20 ft wide × 80 ft long.

Caltrans geologists and geotechnical engineers reviewed the site immediately after the maintenance superintendent warned them about the seriousness of the situation. After reviewing early site plans and construction cross sections along with the soil profile, it soon became apparent that the pump-house construction of 1960 passed through the impervious clay stratum and bottomed out on the sand layer immediately overlying the sand and gravel aquifer stratum. Since construction in 1960 and the 20 plus years following produced no flowing water, it was concluded that this sand and gravel aquifer later became

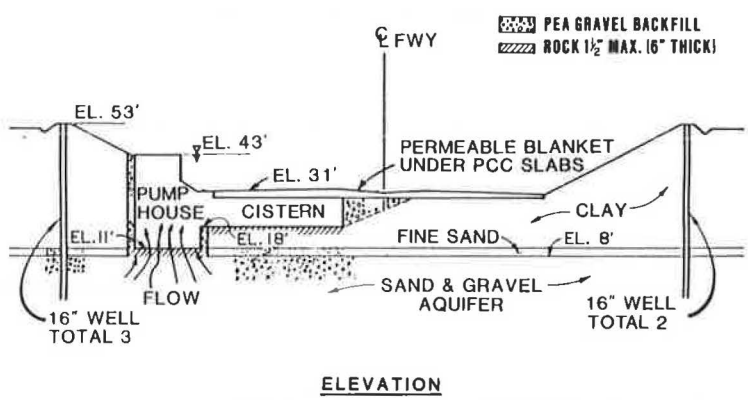


**FIGURE 4 San Jose 10th Street Pump House, U.S. 101 (looking north).**

**FIGURE 5 Temporary repair of shoulder where a relief trench was cut to allow the water to flow from the highway subgrade.**

charged to a pressure head that in 1983 was 12 ft above freeway grade.

After reaching some critical pressure head, possibly 5 to 8 ft above freeway grade, piping of the fine sand layer overlying the sand and gravel aquifer into the clean gravel backfill developed opening channels for large water flows to the surface (see Figure 4). Inducing quick release of the pressure head at the surface aggravated the scouring action of the sand blanket just beneath the pavement and resulted in the further undermining of the pump house and cistern.

### Dewatering

Once the water source flow was assessed, plans were made to develop several wells at freeway grade and to use the cistern and pump house for water storage and removal. These plans were quickly abandoned after a futile attempt was made to bucket-auger a 12-in.-diameter well to the aquifer from freeway grade. Artesian pressure had to be resisted by a heavy drill mud that was difficult to work together with further complications arising from normal drilling problems.

Subsequently, plans were made to install wells on both sides of the freeway at dike (original ground) level. Initially only two wells were planned, one on each side of the freeway. Holes were augered with a 36-in.-diameter bucket to a depth of 40 ft and cased with a 30-in. corrugated metal pipe with the annular space backfilled with concrete. Following this, a 16-in.-diameter 45-slot (0.045-in. openings) stainless-steel well screen was placed. Well-screen length was 30 ft and the screen was positioned within the limits of the aquifer. Gravel packing around the screen consisted of No. 6 × No. 12 Monterey sand (Figure 6). Gravel pack and well-screen size used were the responsibility of the experienced well-drilling contractor.

Soil borings made by Caltrans before the well drilling indicated the aquifer was made up of the least 60 percent gravel with less than 10 percent minus No. 200 sieve sizes. Sampling techniques limited the maximum material size to 2 in.; hence, larger sampling diameters would have undoubtedly yielded a higher gravel percentage. Figure 6 shows the gradation of the aquifer and the No. 6 × No. 12 Monterey sand used as gravel pack.

Wells were developed by jetting with water and back flushing. Peak discharge flows were a disappointing 500 to 700 gpm per well that soon leveled off to about 400 to 600 gpm. Renewed back flushing increased the flow to earlier, peak values but, after 2 or 3 days pumping, it again leveled off to the lower flows.

Drawdown characteristics were measured over a period of

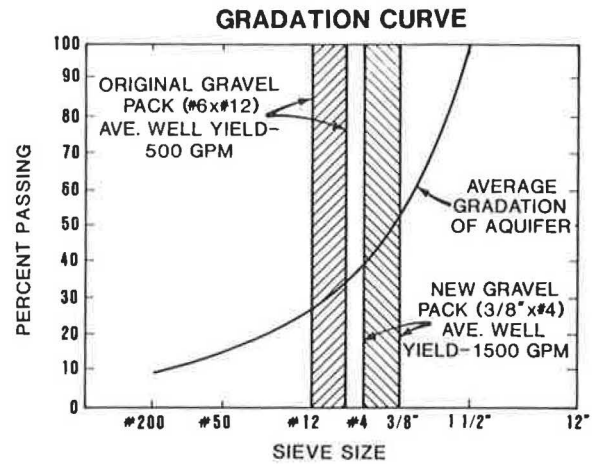


**FIGURE 6 Gradation averages of aquifer and gravel packs. Original gravel pack incorporated 0.045-in. slotted screen; new gravel pack incorporated 0.100-in. slotted screen.**

several weeks and indicated an approximate 5-ft drawdown at 100 ft under a combined pumping rate of 1,000 gpm (Figure 7). Because the wells were located approximately 110 ft from freeway centerline, a combined long-term pumping rate of at least 3,000 gpm was estimated to develop a theoretical 15-ft drawdown at centerline. A 15-ft drawdown was considered about the minimum required for an economic grouting program.

As a result, a decision was made to install three additional wells, two on the south side of the freeway and one on the north side. This time Caltrans engineers decided to install a 100-slot screen (0.100-in. opening) and increase the gravel pack to ⅜ in. × No. 4 (see Figure 6) to increase well yield and risk migration of fines from the aquifer.

Once these new wells were in and flushed, pumping began with significant increases in flow. The new north-side well had a long-term capacity in excess of 2,500 gpm, whereas the two south-side wells had long-term capacities of about 1,500 gpm. The belief that a significant increase in the migration of fines would occur was unfounded as pumped water indicated turbidity almost the same as that in the first two wells. With all pumps turned on, more than 5,000 gpm could now be pumped resulting in a drawdown of about 19 ft (Elevation 24) in the vicinity of the cistern. Figures 8 and 9 show the wells after installation and water discharge during pumping.

## GROUTING

In order to *(a)* seal the primary source of water from the pump house area that connected to the aquifer, *(b)* shut off water flow

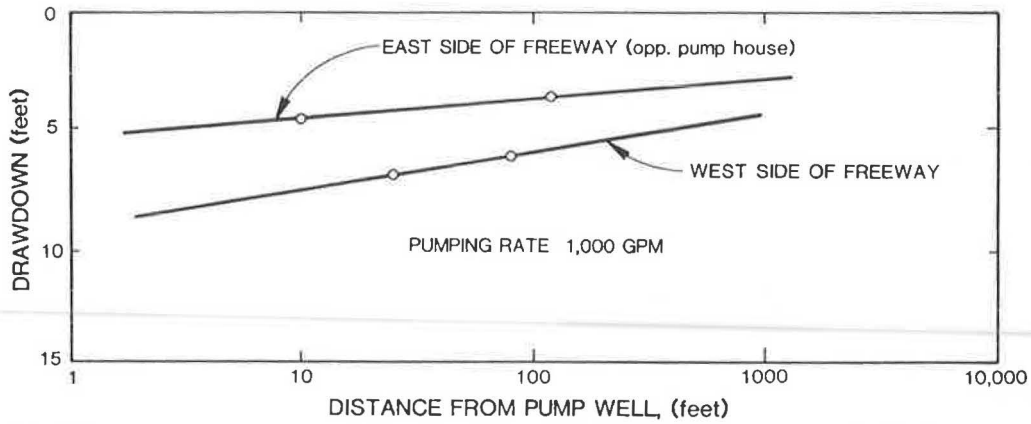


**FIGURE 7 Drawdown characteristics of aquifer on east and west side of freeway. East side curve typified area drawdown when full dewatering was implemented.**

in the clean gravel fill under and around the pump house and cistern, *(c)* fill voids created by the washing of fines, and *(d)* anchor the structures to resist buoyancy, a contract was let to Pressure Grout Company whose personnel worked with Caltrans to develop, implement, and modify the procedures.

Phase I grouting was to seal the perimeter of the pump house. This was accomplished from the sloping surface around the pump house using grout injection points driven to Elevation 10 (Figure 4) and staged up.

**FIGURE 8 Bucket rig used in drilling wells. Upper 40 ft lined with 30-in. corrugated metal pipe and concrete backfilled. Water jet spray used for cleaning.**

Phase II was to grout the pump house invert. This was accomplished by drilling through the pump house floor, setting packers (about 12 ft of groundwater head) with pipes attached that injected the grout at the base of the 2 ft of gravel fill underlying the slab. Phase III of grouting was to fill the gravel and voids under the cistern using the same techniques used in the pump house.

Phase IV was the grout filling of the gravel and voids around the perimeter of the cistern. Because of the three lanes of freeway above the cistern, this was accomplished through grout holes drilled through the walls.

The high water velocities through the 2-in. holes in the

**FIGURE 9 Discharge from one of the wells during well development.**

cistern (previously drilled to relieve water pressure from the highway pavement) scoured fines from the subgrade, the underlying silt and clay layer, and presumably from the aquifer itself, and ended up in the cistern, which acted as a settlement pond. During the pump-house grouting (Phases I and II) almost 150 $yd^3$ of silt was removed from the cistern using a propane-powered Bobcat loader and hoisting it to the surface for transportation to a disposal site. Figure 10 shows the drilled relief holes and silt line within the cistern.

Because of the potential voids beneath the pump house, it was feared that any large decrease in the water level could result in settling of the pump house with resulting shearing of connections and supports to the cistern. At the same time, there was the concern that grout pressure could cause uplift. Using "dangerous" grout pressures (40 to 50 psi), techniques were used that essentially precluded uplift. Precise elevation monitoring was maintained during both dewatering and pressure grouting. Maximum differential movement of the pump house during both dewatering and grouting was less than 0.015 ft.

Grout mix used throughout this filling and sealing operation consisted of nine sacks of cement and three sacks of bentonite per cubic yard admixed to eliminate setting shrinkage and


**FIGURE 10 Inside 8-ft tall cistern. Note *(a)* drilled water pressure relief holes, *(b)* high water line, and *(c)* silt line.**

yielding a full volume set. No sand was used because of potential filter plugging in the pea gravels; the goal was a complete filling of all voids.

Grout was pumped as follows:

| *Phase* | | *Cubic Yards* |
|---|---|---|
| I | Pump house perimeter | 62 |
| II | Under pump house | 15 |
| III | Under cistern | 10 |
| IV | Cistern perimeter | 55 |

This work was completed in 4 working days.

## SOIL ANCHORS

To resist buoyancy, it was assumed that the pump house and cistern would behave like a piston within the clay layer or cylinder. It was not known how effective the slurry grout seal was at depth; therefore, it was assumed that an effective seal existed only around the perimeter of each structure. Because of potentially large displacements in the soil as a result of earthquake loading breaking any bond between grout and the clay layer surrounding the structures, frictional resistance between structure and soil was conservatively assumed to be zero.

Resistance to uplift was then based on the dead weight of the structures plus any overlying soil. Uplift forces were computed on a water elevation rise to 48 ft (15 ft above freeway grade). Water elevation rises to a maximum of 53 ft (20 ft above freeway grade) were assumed as an ultimate condition. Water rises above this point would cause rupture of the overlying clay layer somewhere near freeway centerline, therefore it was considered fruitless to provide safety factors of the structures significantly higher than incipient failure of the freeway itself.

Under the design water elevation rise of 15 ft above freeway grade, 2,270 kips of resisting force was required to resist buoyancy of the cistern and 455 kips was required for the pump house. Several alternatives were evaluated to determine the best way to provide resistance to the bouyancy forces. Of the alternatives discussed, the more practical ones, beside soil anchors, were tie-beams over the top of the cistern held down by large-diameter friction piles external to the structure; and for the pump house a girth strap with friction piles or dead weight added to the top of the structure. The soil anchors were preferred primarily because of the almost total lack of impact on the traffic flow. All the anchors could be installed from within the structures. The potential disadvantage was long-term stability due to corrosion of the pipe. However, this was of negligible concern because future freeway widening plans were no more than 10 years away and at the minimum, a new pump house was required. In addition corrosion estimates were very low for a 20-year design life based on the nonoxygenated water, pile depth, absence of chlorides and sulfates, and the highly alkaline environment of the grout around the anchors.

To determine pile tip elevation and anchor pullout resistance, soil anchors were driven into the ground at freeway surface and load tested. Soil anchors consisted of perforated 2-in.-diameter Schedule 80 steel pipe in 5-ft lengths, coupled with steel pipe sleeves (40-kip rupture). The anchors were driven by a pneumatic 90-lb pavement breaker (Figure 11) to depths of 30, 40, and 50 ft into the sand and gravel and then grouted in place. A minimum holding capacity of 30 kips was necessary, which was developed with the 50-ft penetration depth. Figure 12 shows the loading and reaction-frame apparatus used for the pullout resistance tests.

**FIGURE 11 Air hammer used in driving soil anchors to depth. Location where test soil anchor is driven.**

Figure 13 shows the soil anchors installed within the cistern before grouting and cutoff. Because the dewatering reduced the head, there was only a small inflow from the aquifer through these anchors. Casings were set from the top of slab to above the water table owing to the high head in the pump house, and all work progressed through the casings until the anchors were grouted.

The grout used was CemChem, a two-solution cement grout using set times of 30 to 45 sec. Longer set times would travel long distances in the gravel aquifer without benefit to the project. CemChem has the characteristics of full-volume set, no syneresis, and controllable final set times from less than 10 sec to more than an hour.

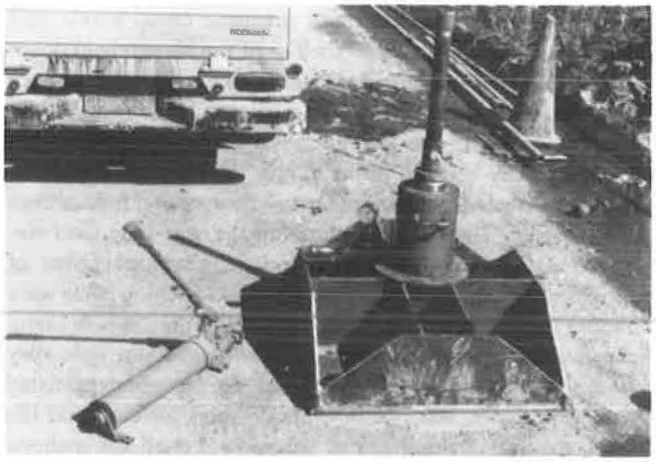

**FIGURE 12 Hydraulic jack and reaction frame used for testing pile anchor capacity.**

Once the technique was developed for anchor placement and grouting, the contractor began placement of the anchors within the cistern on September 7, 1985. A total of 105 soil anchors were placed within the cistern and 33 were placed within the pump house. The job was completed on November 1, 1983, 43 working days after driving the first anchor.

Design working load of each anchor was 20 kips with the ultimate estimated load between 30 and 35 kips. During the grouting process, anchors were periodically load tested for pullout capacity. Approximately 10 percent were tested with the first few indicating low pullout capacities of between 25 and 27 kips. Sandblasting the pipe and a change in the grouting procedure before placement resulted in the pullout resistance exceeding the 30-kip capacity requirement.

## CONCLUSIONS

The dewatering, grouting, and soil anchor program was developed on an emergency basis. In particular, the soil anchor system had never been used before to the authors' knowledge and was an extrapolation.

In retrospect, the current problems could easily have been prevented if the pump house had not terminated on the sand layer immediately above the sand and gravel aquifer. Only a few feet of impervious material would have been sufficient to preclude piping. Alternatively, excavation of the sand layer and recompaction of an overlying impervious material with or without a membrane seal would also have prevented the piping problem.

Future geotechnical investigations for such facilities will examine past historic peak water table elevations and design the facility accordingly unless strong evidence suggests otherwise. Caltrans has constructed a significant number of depressed sections in a variety of soil and water table conditions. In virtually every instance these facilities are trouble free as a result of the design accommodating site conditions.

**FIGURE 13 Soil anchors installed in cistern before grouting and cutoff.**

However, long lulls in high rainfall and the increased demand on virtually all usable water sources can produce a false sense of security, as happened in this instance. The lesson learned here is valuable and will be used as an excellent teaching guide for the future.

## ACKNOWLEDGMENTS

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# Modeling Grouted Sand Under Torsional Loading

CUMARASWAMY VIPULANANDAN AND RAYMOND J. KRIZEK

**The mechanical behavior of grouted sand is controlled by some combination of the properties of grout, sand, and their interaction. In this study, chemically grouted sand is considered a two-phase particulate composite, and the mechanical properties under pure torsional loading are examined at both the particulate and composite levels. Both the adhesive and cohesive properties of grout are believed to influence the behavior of grouted sand, and an experimental program was conducted to quantify the particular relationship for each. These data, together with the porosity of the sand, are employed to formulate strength, shear modulus, and failure strain models for predicting the behavior of grouted sand from a knowledge of the properties of the constituents. The most critical mode of failure for grout and grouted sand is tension, but comparisons are also made between the shear and compressive properties.**

The extensive use of chemical grouts in recent years to solve a multitude of geotechnical engineering problems has dictated the need for a better understanding of the behavior of the grouted sand under different loading conditions. Most studies have been limited to the behavior of grouted sand in compression with very little emphasis on understanding the mechanisms responsible. As one step toward better understanding the complicated interaction between the sand and the grout, a detailed experimental study was undertaken of the adhesive and cohesive properties of grout and the mechanical properties of grouted sand under torsional loading. The mechanical properties of interest are the strength, failure strain, modulus, and critical mode of failure. Adhesive tests described in an earlier study are used to evaluate the adhesive properties, and torsional tests are used in this study to investigate the mechanical properties of grout and grouted sand.

## OBJECTIVES

The objectives of this study are *(a)* to evaluate the respective contributions of sand and grout, as well as their interaction on the overall behavior of grouted sand under torsional loading, *(b)* to use the properties of the constituents in the formulation of models for predicting the shear response of grouted sand, and *(c)* to study the variation in the mechanical properties of grout and grouted sand with curing time.

## MATERIAL SELECTION

Because the purpose of this investigation was to improve understanding of the mechanisms influencing the behavior of grouted sand under torsional loading, the materials were limited to one chemical grout and one sand. The grout mix consisted of hydrated sodium silicate ($Na_2SiO_3 \cdot nH_2O$), water ($H_2O$), formamide ($HCONH_2$) and ethyl acetate ($CH_3COOC_2H_5$) proportioned according to volume in the ratio of 10:8:1:1; the gel time of this grout mix was about 15 min at room temperature (20°C ± 2°C). The sand was Ottawa 20-30, which is composed of almost pure quartz and has a uniformity coefficient of 1.08.

## METHOD OF TESTING

In the absence of standard tests to evaluate the shear properties of either grout or grouted sand, testing techniques were developed to obtain the required measurements. Two types of tests were employed: one to measure the shear properties of the grout and grouted sand and the other to measure the adhesive strength (bond strength).

### Hollow Cylinder Torsional Test

The torsional test used herein subjects a specimen to all three forms of stress simultaneously, and the manner of failure will be governed by the critical strength. Materials that are weaker in shear than in compression or tension will fail in shear, whereas materials that are weaker in tension than in compression or shear will fail in tension. When a specimen is loaded in pure torsion, the material will be subjected to equal magnitudes of shear, compression, and tensile stresses, and the specimen will fail in its critical mode, as shown in Figure 1. During pure torsional loading, the directions of the principal stresses are unchanged. A torsional test allows measurement of the applied torque and angular deflection, and hence constitutive relationships can be developed. In this experimental program, a torsional test on a hollow cylindrical specimen was employed, as shown in Figure 2.

The stress distribution within a hollow cylindrical specimen

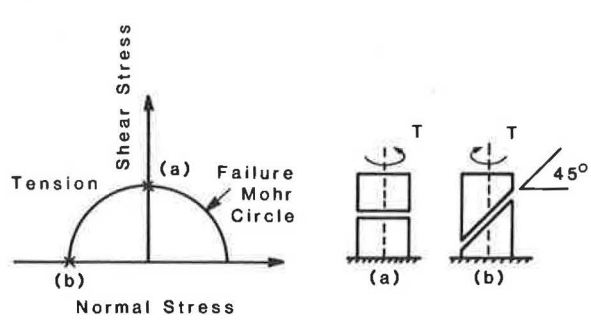


**FIGURE 1 Critical modes of failure under pure torsional loading *(a)* shear failure and *(b)* tensile failure.**

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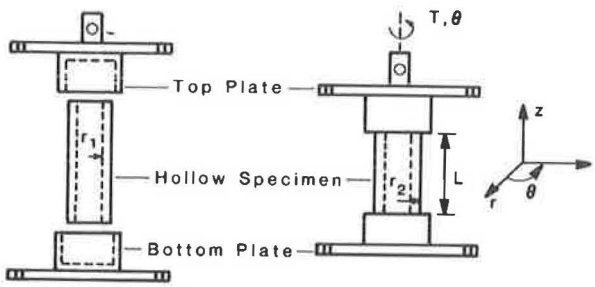


**FIGURE 2 Torsional test configuration.**

will depend on its material properties, specimen geometry (inner and outer radii and height), and end restraints *(1, 2)*. Because of the low strength and brittleness of the grout and grouted sand limitations associated with specimen preparation and handling, it was decided to adopt a ratio of inner to outer radius of 0.5. Figure 3 compares the selected dimensions with geometries used by other researchers *(2)*. The length of the specimen should be as long as possible so that a large region will exist that will not be affected by the end restraints. In general, however, the stability of a specimen and experimental difficulties will govern its height. The effective height, L, of the specimens used in these tests varied between $3.5r_2$ to $4r_2$.

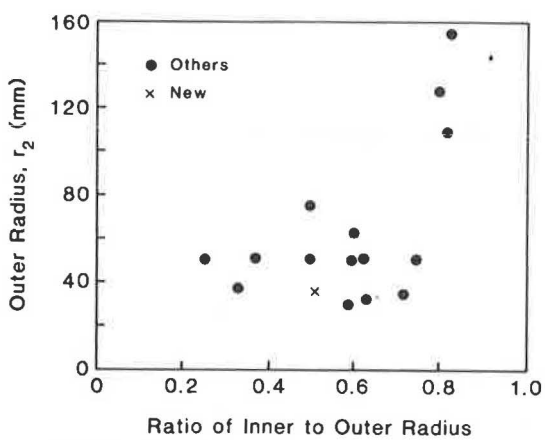


**FIGURE 3 Geometries of specimen used in earlier hollow cylinder apparatuses.**

Simple averaging will be used in reducing the data. Hence, the average shear strain is given by

$$\gamma_{\theta z} = [(r_1 + r_2)/2]\ \theta/L \qquad (1)$$

where θ is the angle of twist and $r_1$, $r_2$, and L are the specimen dimensions, as shown in Figure 2. If T is the twisting moment, equilibrium demands that

$$T = \int_A \tau_{z\theta}(r)\, dA\, r = 2_\pi \int_{r_1}^{r_2} \tau_{z\theta}(r)\, r^2 dr \qquad (2)$$

Assuming the stress to be uniform, which is generally justified for a thin-wall cylindrical specimen, will result in

$$\tau_{z\theta} = 3T/[2_\pi (r_2^3 - r_1^3)] \qquad (3)$$

This uniform shear stress and average shear strain will be used in all calculations.

## Adhesion Test

The physical and chemical interaction of two or more materials at their interface is known as adhesion or bonding. In this case the sand surface will be modeled by a quartz rock with a chemical composition similar to that of sand, and simple tests proposed by Krizek and Vipulanandan *(3)* will be used to determine the adhesive strength. Both tensile and shear failures are possible under torsional loading, and the adhesive tensile and shear strengths will be of concern in this study.

The tensile test consisted of sandwiching a layer of grout between flat surfaces of quartz rock, as shown in Figure 4*a*. For this type of test, equilibrium considerations dictate that the average adhesive tensile (AT) stress at failure, $\sigma_{tf}^{AT}$, across the interface in the axial direction must be P/A where P is the maximum applied load and A is the cross-sectional area. The adhesive shear strength was evaluated by use of a torsional test, which consisted of subjecting an inner rock core to torsion while holding fixed an outer surface, with grout filling the intermediate annulus, as shown in Figure 4*b*. Because the radial displacement is zero within the grout and there is no warping of the cross section, the grout area will remain constant and this loading will induce a relatively pure shear condition within the grout. When this configuration is subjected to a torque, T, equilibrium at the inner surface demands that

$$T = \int_0^L (2\pi r_1\, dz)\, \tau_{r\theta}(z)\, r_1 = 2\pi r_1^2 \int_0^L \tau_{r\theta}(z)\, dz \qquad (4)$$

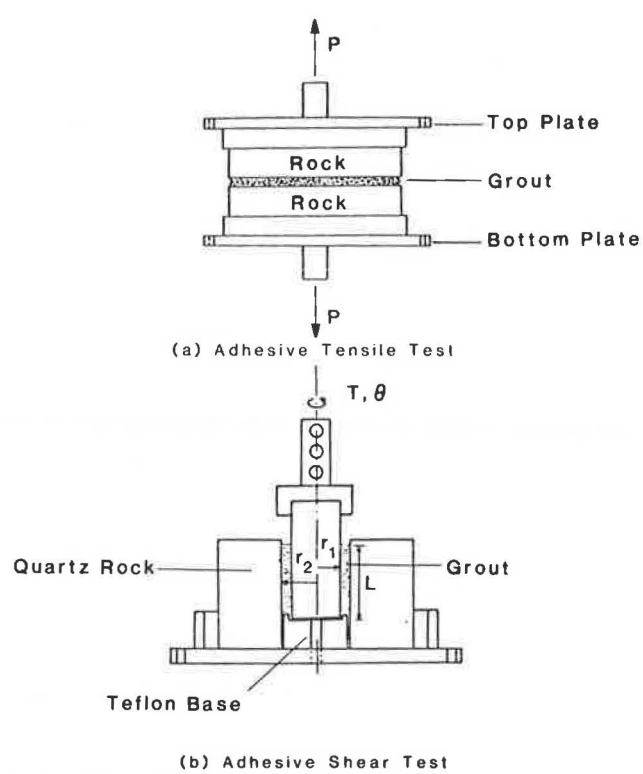


**FIGURE 4 Adhesive test configurations.**

Because the rock is much stiffer than the grout with a no-slip condition assumed at the interface, all points on the interface are subjected to the same angular displacement, which implies a uniform stress distribution as follows:

$$\tau_{r\theta} = T/2\pi r_1^2 L \quad (5)$$

As the gap between the rock surfaces is small (about 1.8 mm), the shear stress at the inner and outer interfaces, as well as within the grout, will be almost equal.

## PROPERTIES OF GROUT

Pure grout (PG) specimens were tested in unconfined torsion after different periods of curing in a humid temperature-controlled room. Hollow specimens were made by inserting a nylon rod concentrically into a 6-in. long, 1.5-in. diameter PVC tube that was closed with rubber stoppers at the bottom and sealed at the top with waxed paper. These rods were subsequently removed with a small twist before the specimens were glued to the plates, as shown in Figure 2, and assembled in the torsional device *(4)*. This was a load-controlled test and the specimens were loaded at 0.42 N·m/min, so that failure occurred in about 15 min.

Typical test results are shown in Figure 5. The stress-strain relationship for pure grout is nonlinear initially, but with increased loading it becomes linear. Because all specimens failed in tension, the most critical mode of failure for pure grout is obviously tension. The variation of strength, shear strain at failure, and shear modulus with curing time is shown in Figure 6. Because the failure was tensile, the failure stress corresponds to the tensile strength and these results are comparable with results reported from direct tension tests by Vipulanandan and Krizek *(5)*. The strength increased continuously at a reducing rate. The 7-day strength (0.2 MPa) increased by about 40 percent after 1 additional week of curing and a further 25 percent after 28 days of curing. The failure strain decreased rapidly with curing time until the change was negligible after 20 days. The 7-day failure strain (0.85 percent) reduced by 30 percent after 1 additional week and a further 10 percent after 28 days of curing. The shear modulus is defined as the gradient of the stress-strain curve at 50 percent of failure strength. The shear modulus increased rapidly during the initial 2 weeks, with the 7-day modulus doubling during the second week and increasing a further 25 percent after 28 days of curing.

## BONDING PROPERTIES

The adhesive tensile strengths obtained from several tests are summarized in Figure 7*a*. The strength increased rapidly during the first 8 days of curing and reached a maximum value of 5.2 kg/cm$^2$ (0.51 MPa). However, with increased curing the strength reduced to less than 4 kg/cm$^2$ (0.39 MPa) on the 20th day and to almost 3.5 kg/cm$^2$ (0.34 MPa) on the 30th day of curing. The variation of adhesive shear strength with curing time is summarized in Figure 7*b*. The adhesive shear strength, $\tau_f^{AS}$, increased rapidly during the first few days, reaching a peak of 3.5 kg/cm$^2$ (0.34 MPa) around the second week. However, with increased curing, the strength reduced to 2.8 kg/cm$^2$ (0.27 MPa) after about 3 weeks and 2.5 kg/cm$^2$ (0.25 MPa) after about 4 weeks. A similar trend in the adhesive

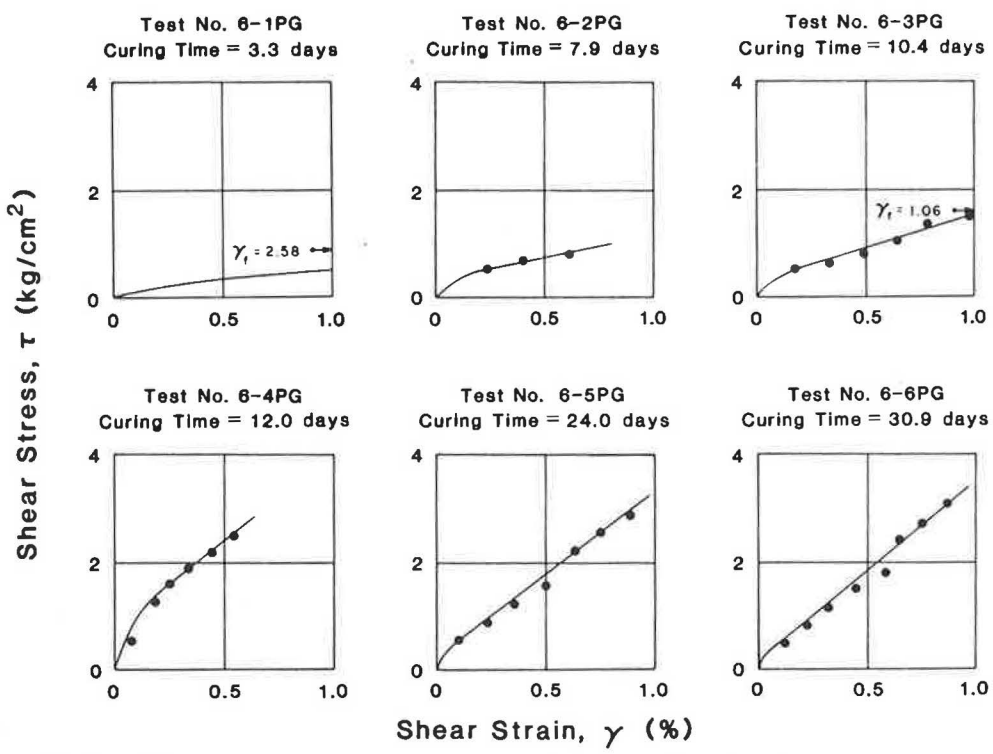


**FIGURE 5 Shear stress-strain relationships for pure grout at different curing times.**

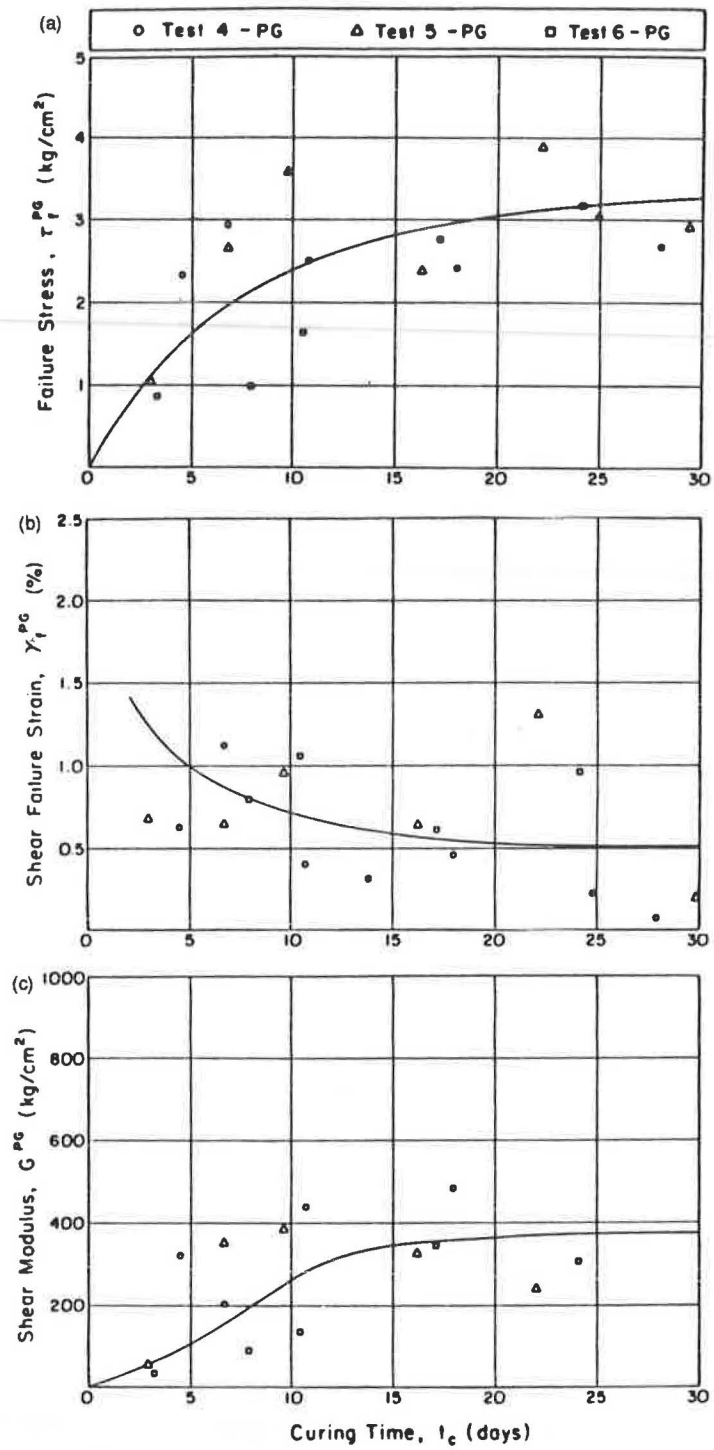


**FIGURE 6 Variation of failure stress, shear failure strain, and shear modulus with curing time for pure grout.**

tensile strength was observed earlier. The reduction in strength may be caused by *(a)* a continuous chemical reaction at the interface and *(b)* a partial debonding and development of high residual stresses as the grout shrinks and becomes brittle. Although the trend in strength development appears to be generally similar, the adhesive tensile strength is greater than the adhesive shear strength during the period under consideration.

## PROPERTIES OF GROUTED SAND

The grouted sand specimens were prepared by injecting grout into sand confined under a $K_o$ condition. As in the case of pure grout, a concentric nylon rod was placed in a plexiglass mold and a known amount of sand was placed and vibrated to obtain a porosity of 0.36 ± 0.02. The test configuration used for these tests was similar to that used for pure grout. Six specimens

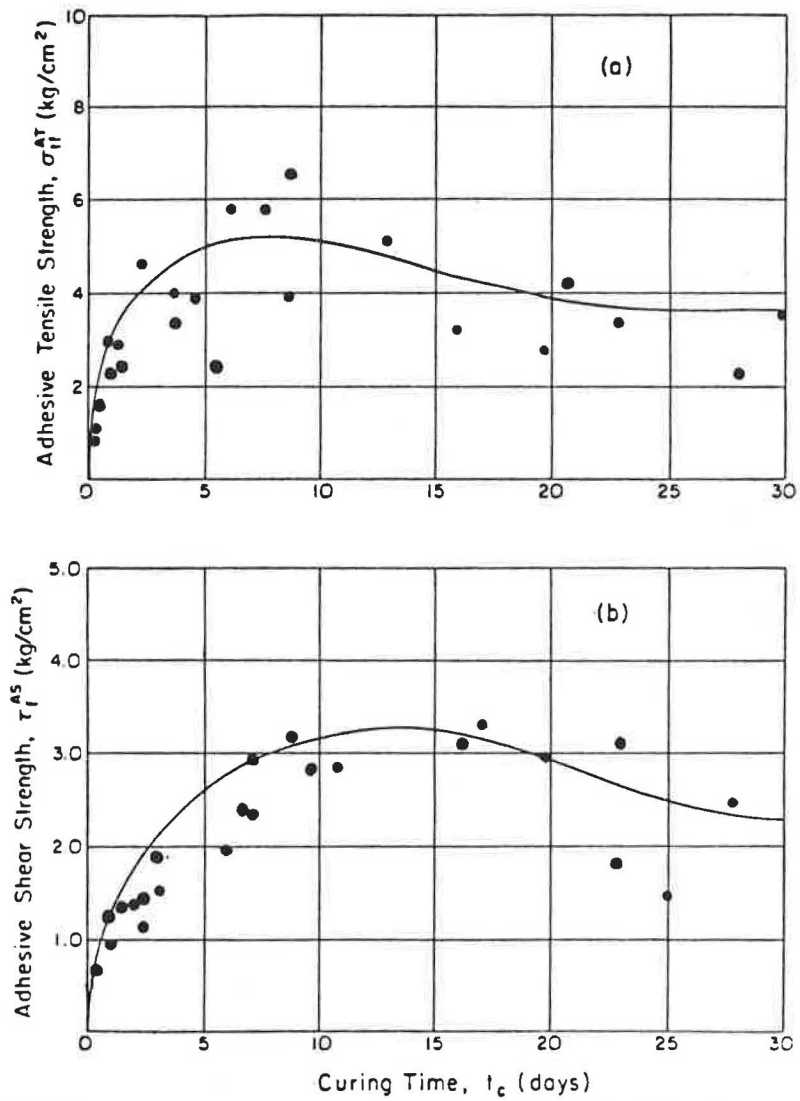


**FIGURE 7 Variation of adhesive tensile and shear strengths with curing time.**

were grouted in parallel at an injection pressure of approximately 13.8 KPa; about 6 void volumes of grout were passed through each specimen to achieve complete grout saturation *(6)*. Approximately 1 day after grouting, the molds were dismantled and the specimens were removed, sealed in moistened plastic bags, and stored in a humid room at a temperature of 20°C ± 2°C.

For specimens with short curing times, the stress-strain relationship under monotonic torsional loading was nonlinear initially and became linear with increased loading. Longer curing resulted in a linear stress-strain relationship, as shown in Figure 8. All failures were tensile, and hence the most critical mode of failure for grouted sand is also tension. Inspection of the failure surface with a magnifying glass with a magnification factor of 3 indicated both adhesive and cohesive failures.

As shown in Figure 9*a*, the failure stress increased continuously at a decreasing rate during the first 2 weeks and remained unchanged thereafter. The 7-day strength, 3.7 kg/cm$^2$ (0.36 MPa) increased by 25 percent during the second week of curing. The angular failure strain, as shown in Figure 9*b*, reduces rapidly during the first week and continued to reduce at a decreasing rate, approaching a constant value with longer curing time. The 7-day failure strain (0.43 percent) reduced by 50 percent after 14 days of curing and remained unchanged thereafter. The shear modulus (Figure 9*c*) increased almost linearly up to 14 days of curing and the trend thereafter was uncertain due to scatter in the data. The 7-day modulus, 1200 kg/cm$^2$ (117 MPa) almost doubled during the second week of curing and increased an additional 30 percent after 28 days.

## COMPARISONS

Most of the available mechanical property data in the literature are on the compressive properties of grouts and grouted sands so it is useful to advance some comparisons between shear and compressive properties. Toward this end, solid cylindrical specimens (38 mm in diameter and 85 mm in height) of pure grout and grouted sand were capped with sulfur compound and

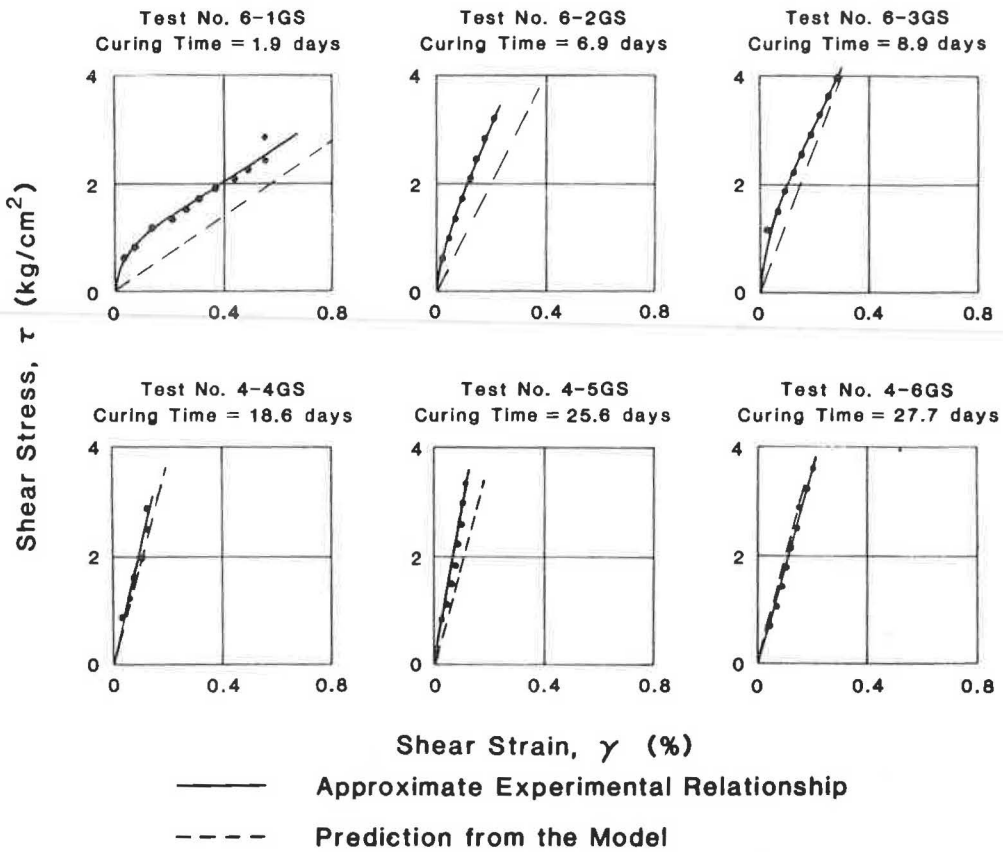


**FIGURE 8 Shear stress-strain relationships for grouted sand at different curing times.**

tested in unconfined compression at a strain rate of 0.15 percent per minute, and the comparisons in the form of ratios of compressive-to-shear values are given in Table 1.

### Grout

The ratio of compressive-to-shear strength increased with curing time and reached a value of 3.2 after 4 weeks. The ratio of failure strains, on the other hand, decreased and approached 2.2 after 4 weeks. The ratio of moduli increased with curing time and reached a value of 3.0 after 4 weeks of curing.

### Grouted Sand

The strength ratio decreased with an increase in curing time, from 4.1 after 7 days to 3.3 after 28 days of curing. The ratio of failure strains fluctuated between 1.5 and 2.0 during the period under consideration. The ratio of moduli decreased during the initial 14 days of curing, but remained almost constant thereafter at a ratio of 2.

## MODELING BEHAVIOR OF GROUTED SAND

Grouted sand can ideally be considered as consisting of two physically distinct materials. The sand phase consists of discrete particles in contact, while the grout phase fills the voids and holds the particles together. The resulting brittle particulate composite material generally has higher strength and stiffness, both of which change continuously during the process of curing. Available studies on particulate composites have been limited mainly to composites in which the particles are dispersed in the matrix (such as concrete), and hence the interactions between the particles have not been incorporated into the available theories.

During the process of grouting, the grout can be asssumed to fill the voids, although this may often not be the case. As a result of adhesion and shrinkage, self-equilibrating internal stresses, $\bar{P}_G$ and $\bar{P}_S$, are induced within the grouted sand, where $\bar{P}_S$ and $\bar{P}_G$ are defined as the volume-average hydrostatic stresses in the sand skeleton and the grout matrix and may be written as

$$\bar{P}_G = 1/V_G \int_{V_G} P_G(x)\, dV_G \tag{6a}$$

$$\bar{P}_S = 1/V_S \int_{V_S} P_S(x)\, dV_S \tag{6b}$$

where $P_S$ and $P_G$ represent the pointwise hydrostatic stresses in the sand and grout and x designates the position. As there is no external pressure during curing, equilibrium demands that

$$n\bar{P}_G + (1 - n)\,\bar{P}_S = 0 \tag{7}$$

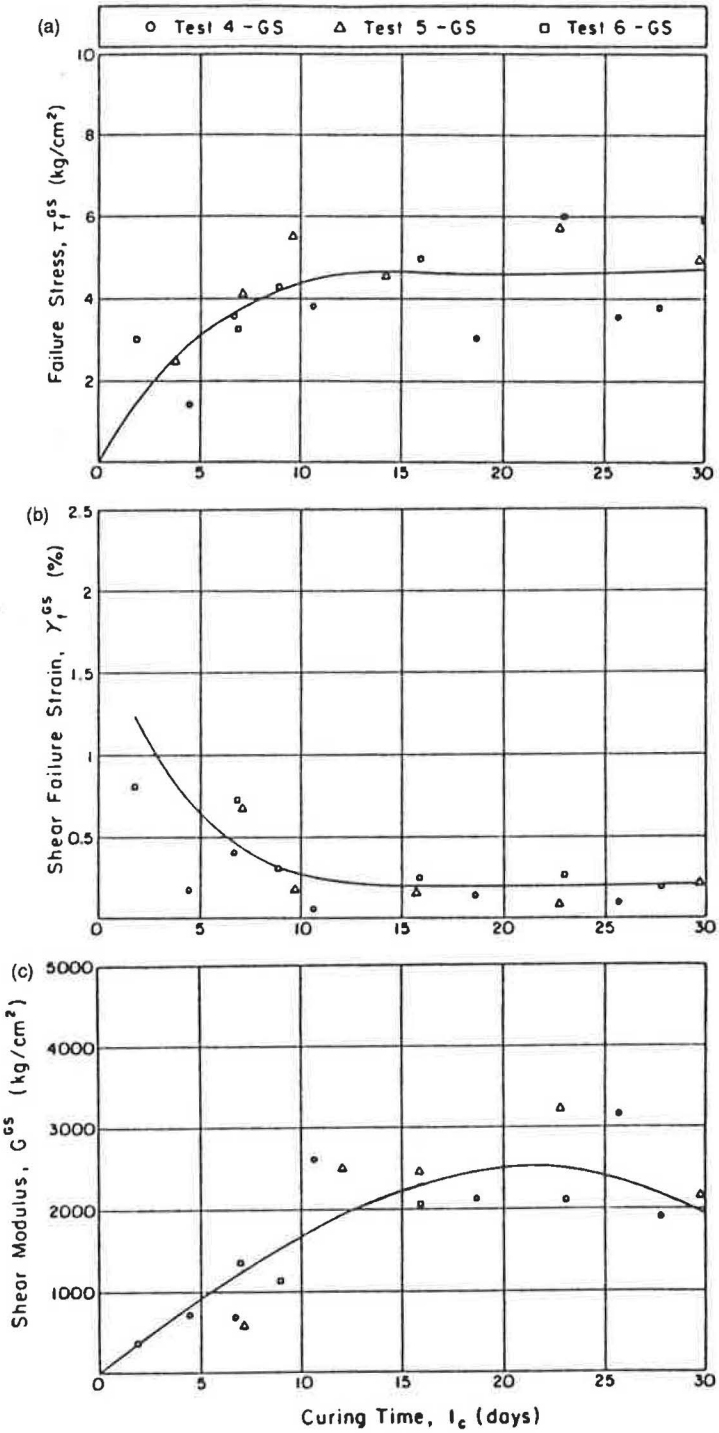


**FIGURE 9 Variation of shear properties with curing time for grouted sand.**

**TABLE 1 RATIOS OF COMPRESSIVE-TO-SHEAR PROPERTIES**

| Curing Time | Pure Grout (PG) | | | Grouted Sand (GS) | | |
|---|---|---|---|---|---|---|
| | Strength[a] | Failure Strain | Modulus | Strength[a] | Failure Strain | Modulus |
| 7 | 1.3 | – | 1.0 | 4.1 | 1.7 | 3.1 |
| 14 | 2.8 | 3.1 | 1.7 | 3.7 | 2.0 | 2.1 |
| 28 | 3.2 | 2.2 | 3.0 | 3.3 | 1.6 | 1.9 |

[a]Failure was tensile.

which yields

$$\bar{P}_S = -[n/(1 - n)]\ \bar{P}_G = -e\ \bar{P}_G \quad (8)$$

The induced stresses within the grout matrix will depend on the structure of the soil skeleton, the grout properties, and the method of curing. However, $\bar{P}_G$ cannot exceed the tensile strength of the grout, in which case there would be a failure in the grout matrix during curing. The maximum value of $\bar{P}_S$ will be controlled by the maximum stress transfer that can occur at the interface with no bond failure. In no case will $\bar{P}_S$ exceed the adhesive tensile strength at the sand-grout interface. As a result of $\bar{P}_S$, there is particle-to-particle interaction. Hence, the behavior of grouted sand can be represented as a combination of the behavior of grout and sand without particle interaction (similar to a two-phase dispersed particulate composite) and the behavior of the sand skeleton with particle interaction (the modification that must be made in available theories). The latter is influenced by the initial residual stresses and restraint to particle movement (rolling and sliding) caused by grout acting as a filler material.

## Strength Model

Under pure torsional loading the grouted sand failed in tension. At or close to tensile failure the intergranular forces on the failure surface will approach zero. Hence, the effect of particle interaction on such a failure can be neglected. Experimental observations show both adhesive and cohesive failures on the failure surface. Because the failure is tensile, the adhesive tensile strength will play an important role, and it has been shown by Vipulanandan and Krizek *(5)* that the tensile stress on the failure surface can be represented as

$$\sigma_{tf}^{GS}/\sigma_{tf}^{PG} = (1 - n)\ (\sigma_{tf}^{AT}/\sigma_{tf}^{PG}) + n \quad (9)$$

where $\sigma_{tf}^{AT}$ is the adhesive tensile strength of grout, $\sigma_{tf}^{PG}$ and $\sigma_{tf}^{GS}$ are the tensile strengths of pure grout and grouted sand, respectively, and n is the porosity of the sand before grouting. As explained earlier, under pure torsional loading, it is expected that the magnitude of the applied shear stress will be equal to that of the tensile stress in the specimen. Hence, $\sigma_{tf}^{PG}$ and $\sigma_{tf}^{GS}$ can be replaced by $\tau_f^{PG}$ and $\tau_f^{GS}$, respectively, where $\tau_f^{PG}$ and $\tau_f^{GS}$ are the shear stresses at failure for pure grout and grouted sand, and Equation 9 can be rewritten as

$$\tau_f^{GS}/\tau_f^{PG} = (1 - n)\ (\sigma_{tf}^{AT}/\tau_f^{PG}) + n \quad (10)$$

Figure 10*a* compares the strength ratios (grouted sand to pure grout) predicted by this model with the experimental data, where values for $\sigma_{tf}^{AT}$ and $\tau_f^{PG}$ were obtained from Figures 7*a* and 6*a*, respectively. Although there is a scatter in the data, the trend in the variation of the strength ratio with curing time is quite adequately represented by the model. Figure 10*b* indicates a satisfactory agreement between the predicted shear strength of grouted sand and the experimental data. Thus, it appears that this model can be used in conjunction with the adhesive and cohesive properties of grout to predict the shear strength of grouted sand.

## Stiffness Model

A review of the literature revealed a number of formulas for predicting the modulus of composites based on the modulus and volume concentration of each constituent. However, all of these models have been developed on the basis of very limited or no particle interaction. Because particle interaction plays an important role in the case of grouted sand, any such model must be used cautiously and with appropriate modification.

Hansen *(7)* found that the iso-stress model, also termed the Reuss model, is better than the iso-strain model when the aggregate is stiffer than the matrix, which is the case in grouted sand [the stiffness of quartz sand particles is about $7.5 \times 10^5$ kg/cm$^2$ ($7.4 \times 10^4$ MPa) *(8)* and that of the grout is on the order of $10^2$ kg/cm$^2$ (9.8 MPa) and changes somewhat with curing time]. Hence, with appropriate modification the iso-stress model can be expressed as

$$1/G^C = (1 - C)/G^m = n/G^m \quad (11)$$

where the modulus of the composite, $G^C$, can be represented in terms of its matrix modulus, $G^m$, and volume concentration of particles, C.

As mentioned earlier, the stiffness of grouted sand can be attributed to two factors—particle interaction and the arrangement of rigid particles in a grout matrix. Accordingly, the stiffness of the grouted sand, $G^{GS}$, can be represented as a linear combination of the two factors, which can be written as

$$G^{GS} = G^S + G^C \quad (12)$$

where $G^S$ is the stiffness due to the particle interaction and $G^C$ is the stiffness of grouted sand without particle interaction. $G^S$ will be influenced by the initial confinement, $\bar{P}_S$, restriction to particle movement, and sand fabric. Because the failure strain of grouted sand is less than 1 percent and particle movement is restricted, $G^S$ can be represented by the initial tangent modulus of sand. When grouted sand is tested in an unconfined condition, the initial confining pressure that will influence $G^S$ is $\bar{P}_S$. As an approximation, we can assume that $\bar{P}_G$ is influenced (controlled) by the uniaxial tensile strength of the grout; hence, a limiting case can be represented as

$$\bar{P}_S = -e\ (\sigma_{tf}^{PG}/3) = -e\ (\tau_{tf}^{PG}/3) \quad (13)$$

Because particle movements (rolling and sliding) are restricted by the grout filler, it is reasonable to approximate the modulus of the sand skeleton, $G^S$, by the small strain dynamic modulus of sand. According to Richart et al. *(9)*, the dynamic shear modulus, G, for clean rounded sands ($e < 0.80$) can be expressed as

$$G = [700(2.17 - e)^2]/(1 + e)\ (\bar{\sigma}_o)^{0.5} \quad (14)$$

where $\bar{\sigma}_o$ is the mean stress (G and $\bar{\sigma}_o$ are expressed in kg/cm$^2$). Using $e = 0.56$ and $\bar{\sigma}_o = \bar{P}_S$, it is possible to write the shear modulus as

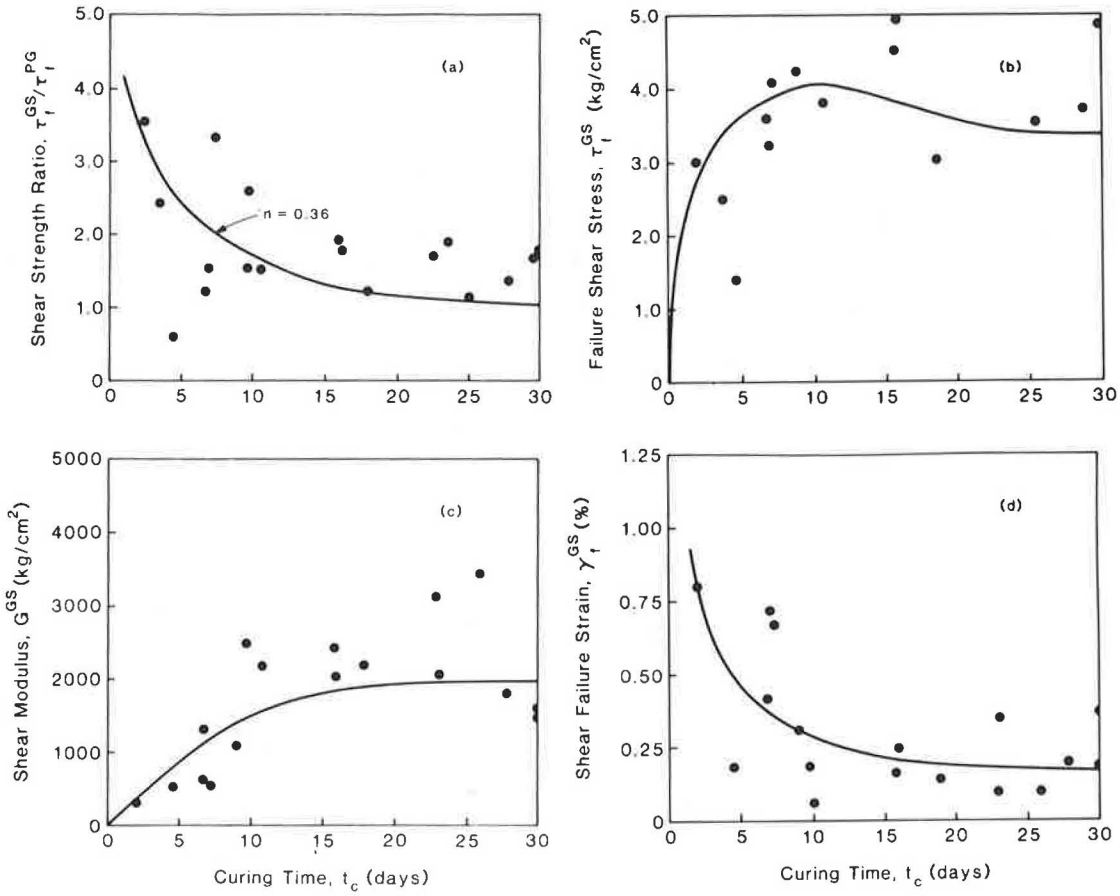


**FIGURE 10 Comparison of predicted and measured results for grouted sand.**

$$G^S = 500\,(\tau_f^{PG})^{0.5} \tag{15}$$

Therefore, the Reuss model can be modified and the stiffness model for grouted sand can be represented as

$$G^{GS} = G^S + G^{PG}/n \tag{16}$$

Figure 10*c* compares the experimental and predicted shear modulus of grouted sand. The variation in the shear modulus is well represented and the prediction of the magnitude is satisfactory. Hence, this model can be used in conjunction with the properties of grout and sand to predict the stiffness of grouted sand.

### Failure Strain Model

It was observed that the stress-strain relationship for grouted sand is approximately linear, and the constitutive relationship can therefore be represented as

$$\tau = G^{GS}\,\gamma \tag{17}$$

where $\tau$ is the shear stress and $\gamma$ is the shear strain. Hence, the failure strain can be written as

$$\begin{aligned}\gamma_f^{GS} &= \tau_f^{GS}/G^{GS} \\ &= (1-n)\,(\sigma_{tf}^{AT} + n\,\tau_f^{PG})/(G^S + G^{PG}/n)\end{aligned} \tag{18}$$

Figure 10*d* compares the experimental results with those predicted by Equation 18. The model represents well the variation of failure strain with curing time, and the prediction of the magnitude is good. Hence, this model can be used in conjunction with the properties of grout and sand to predict the shear strain at failure.

Figure 8 compares the predicted and experimental stress-strain relationships for grouted sand. The predictions were based on the strength model (Equation 9) and stiffness model (Equation 16), which were developed independently based on the proposed theory for grouted sand. Although the agreement was not good for short curing times, the predictions improved with increased curing.

## CONCLUSIONS

Based on this complementary experimental and analytical study, the following conclusions can be advanced:

1. The most critical mode of failure for grout and grouted sand under unconfined torsional loading is tension. The strength, failure strain, and modulus of pure grout and grouted

sand change continuously at varying rates during the first month of curing; the strength and modulus were higher for grouted sand than for pure grout, but the inverse was true for the failure strain. Both adhesive and cohesive failures were observed on the failure plane of the grouted sand.

2. The stress-strain relationship of grout and grouted sand under monotonic torsional loading is essentially linear except during the early stages of curing, where it is nonlinear during initial loading but becomes linear with increased loading.

3. The adhesive tensile and shear strengths develop rapidly during initial curing, with the adhesive tensile strength always being greater than the adhesive shear strength. These tests have the potential to be developed as standard testing methods for evaluating the bonding properties of grouts.

4. The behavior of grouted sand can be considered as consisting of two components: one due to interaction of soil particles (such as in ungrouted sand) and the other due to the presence of a grout matrix surrounding the rigid soil particles (as in the behavior of particulate composites). Strength and stiffness models based on this concept give satisfactory results when predictions are compared with the test data. These models offer the potential for predicting the mechanical behavior of grouted sand under torsional loading from a knowledge of the adhesive and cohesive properties of the grout and the porosity of the sand.

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