LIQUEFACTION REMEDIATION NEAR EXISTING LIFELINE STRUCTURES

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ABSTRACT

Organized and systematic studies on the phenomenon of liquefaction began after the 1964 Niigata earthquake. Great progress has been made through research, much of it conducted by researchers in Japan and U.S., to develop a fundamental understanding of the mechanism of liquefaction. A number of in-situ ground improvement methods have been developed to reduce the vulnerability of ground susceptible to liquefaction. Many of these methods were developed empirically, and some are very costly to implement.

This paper examines the critical factors that influence the effectiveness of five ground improvement techniques which are most suitable for remedial work near existing lifeline structures. Expected cost of using these methods are given, even though the cost data from cases examined is scarce. Advantages and constraints of each of these methods are presented. Eight case histories of remedial work near existing lifeline structures are reviewed.

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INTRODUCTION

A major factor of lifeline damage in earthquakes is lateral ground displacement caused by liquefaction of loose granular soils, as illustrated in the case studies for many past earthquakes in the United States and Japan (O'Rourke and Hamada 1992; Hamada and O'Rourke 1992). For example, lateral ground displacement damaged many pipelines, bridges, roads, and buildings during the 1906 San Francisco, California, earthquake. Broken water lines made fighting fires after the earthquake impossible, and much of San Francisco burned. During the 1989 Loma Prieta earthquake, liquefaction and lateral ground movement resulted in major pipeline damage and fires at virtually the same locations in San Francisco. Soil liquefaction during the January 17, 1995 Hyogo-Ken Nanbu, Japan, earthquake completely destroyed Kobe port, which consists primarily of three man-made islands. Numerous breaks caused by liquefaction in Kobe City and the surrounding area water, wastewater, and gas supply systems, led to a number of fires and the total loss of water supply for fighting fires and for domestic use. Many transportation systems were disrupted as the result of liquefaction (Chung et al. 1995). Other important factors of lifeline damage include subsidence associated with densification of the soil and ejection of the water and soil, and flotation of buried structures.

Many lifeline structures lie in regions of high liquefaction and ground displacement potential. While it may be feasible to relocate some support facilities to sites which are not susceptible, similar precautions are not always possible for the long linear element of lifeline systems such as pipelines, electrical transmission lines, communication lines, highways, and rail lines. For some pipe systems, such as gas lines, it may be economical to replace old pipes with modern welded steel pipes that are less vulnerable to break or leak, even after moderate deformation (O'Rourke and Palmer 1994). For other pipe systems, such as water and sewage lines, the segmented pipe used can accommodate very little deformation. Ground improvement may be the only practical solution for these types of systems, and for all types of systems in areas where large ground displacement is anticipated. An important consideration is the lateral extent of improved ground beyond the perimeter of the lifeline. Recent significant earthquakes where ground improvement has been used for liquefaction prevention appear to indicate a distance equal to the liquefiable thickness (Hayden and Baez 1994; Mitchell et al. 1995).

This paper presents the state-of-practice of ground improvement for liquefaction remediation near existing lifeline structures.

LOW VIBRATION GROUND IMPROVEMENT TECHNIQUES

Ground improvement near existing lifeline structures requires special considerations (after Glaser and Chung 1995) because of the following:

- Work vibrations may damage lifeline, which could have very serious consequences;

- Soil needing improvement is obstructed by the lifeline;
- Scope of work is of large areal extent, yet may be limited to a narrow right-of-way;
- Subsurface conditions will vary greatly along alignment;
- Extent of treatment required to protect lifeline is not known;
- Exact location and condition of buried utilities might not be known; and
- Improvement might adversely affect regional hydrology.

Based on these considerations, this section presents five low vibration ground improvement techniques which are appropriate for liquefaction remediation near existing lifeline structures. The advantages and constraints for each technique are summarized in Table 1.

Compaction Grouting

Compaction grouting is the injection of a thick, low mobility grout that remains in a homogenous mass without entering soil pores. As the grout mass expands, the surrounding soil is displaced and densified.

There are many factors that can influence the effectiveness of compaction grouting (Graf 1992; Warner et al. 1992; Rubright and Welsh 1993):

1. *Soil Being Compacted.* Cohesive soils are harder to compact than cohesionless soils. The technique is not effective in thick, saturated clayey soils, and may be marginally effective in silt deposits.

2. *Earth Pressures.* Overlying ground will heave if overburden pressure is low, and injection pressure and rate are too high.

3. *Grout Mix.* Typical grout mixes consist of silty sand, cement, fly ash, and water. Grout slump is usually set at about 25 mm. It has been recommended that the use of bentonite and other clay materials be restricted, since hydraulic fracturing and limited compaction will occur if grout contains sufficient clay irrespective of slump. Cement may not be needed for just soil densification, but its use has had limited success.

4. *Grout Injection Pressure and Rate.* Excessive injection rates and pressures will result in premature heaving of overlying ground. The maximum pressure also depends on the sensitivity of adjacent structures.

5. *Grout Injection Volume.* Uneven distribution of grout will likely result in uneven improvement. Injection volumes range from as low as 4% of the treated volume to as high as 20% for sinkhole areas.

6. *Grout Hole Spacing.* Holes spaced too far apart will leave zones of undensified soil. For deep injection (greater than about 3 m), final spacings of 2 to 4 m are frequently used. For shallow injection, final spacings usually range from 1 to 2 m.

7. *Injection Sequence.* Effective sequencing will utilize confinement created in previous work. Grouting can be performed from the top down (stage down) or from the bottom up (stage up). While stage up grouting is generally more economical, stage down grouting utilizes confinement created in previous work. Near the ground surface where confining pressures are low, stage down grouting may be required to achieve specified compaction levels. It is considered good practice to have at least primary and secondary grout holes, where secondary holes split the distance between primary holes. Injection stages or increments of 0.3 to 0.9 m have been used. In addition, splitting the injection depths will also contribute to greater uniformity.

The cost to mobilize and demobilize the compaction grouting equipment is between $8,000 and $15,000 per rig (Welsh 1995). To install 76-mm diameter grout pipe, the cost starts at about $50 per meter of pipe. This cost would double for low headroom work. The cost of injection labor and grout materials starts at about $20 per cubic meter of improved soil, assuming the volume of grout injected is 10% of the total volume of treated soil.

Permeation Grouting

Permeation grouting is the injection of low viscosity particulate or chemical fluids into soil pore space with little change to the physical structure of the soil. The major objective of permeation grouting is either to strengthen ground by cementing soil particles together or to reduce permeability by plugging soil pores.

There are several factors which influence the effectiveness of permeation grouting (Baker 1982; Perez et al. 1982; Littlejohn 1993; Greenwood 1994):

1. *Soil Being Permeated.* Clean granular soils are easier to permeate than fine-grained soils. Soil permeability is the single most useful index. Porosity dictates the amount of grout consumed. Other important parameters include grain size, soil fabric, and stratigraphy.

2. *Earth Pressures.* Ground fracture and heave occur when overburden pressure is low, and injection pressure and rate are high. Fractures can extend for great lengths since the grout is water-like, and there is little loss of pressure along them.

3. *Ground Water Conditions.* Grout could be leached out of soil by seepage, or attacked chemically or biologically. Some chemical grouts crack where water level fluctuates.

4. *Grout Mix.* Particulate grouts, or suspensions, may consist of micro-fine cement, fly ash, clay, and water. Chemical grout types, or solutions, include sodium silicates, acrylamides, lignosulfonates, and resins (Karol 1982). Sodium silicate grouts are the most widely used chemical grout for soil strengthening. The acrylamides in solution or powder form, and the catalyst used in lignosulfonates are highly toxic. Special handling and mixing procedures may be required to insure the health and safety of workers, and to protect the environment. Grout particle size, viscosity, temperature, setting time, stability, strength, creep, and durability must be considered. In general, micro-fine cement grouts will not permeate medium to fine sand, and chemical grouts will not permeate sands containing more than about 25% silt and clay.

5. *Grout Injection Pressure and Rate.* Excessive injection pressures and rates will result in ground fracture and heave. It has been recommended that injection pressures be kept to about 25% of the fracture pressure determined by field trial.

6. *Grout Injection Volume.* Uneven distribution of grout will likely result in uneven improvement.

7. *Grout Hole Spacing.* Holes spaced too far apart will leave zones of untreated soil. Typical final hole spacings range from 0.5 to 2 m.

8. *Injection Sequence.* Effective sequencing will utilize confinement created in previous work. Grout initially penetrates the more open soil leaving soils of lower permeability untreated. For a more uniform treatment, it has been recommended to inject predetermined grout quantities, and split spacings and depths of injection in successive phases.

The cost to mobilize and demobilize permeation grouting equipment ranges from $15,000 to $25,000 per rig for projects using micro-fine cement grout, and over $25,000 per rig for projects using sodium silicate grout (Welsh 1992, 1995). To install sleeve port grout pipes, the cost is over $50 per meter of pipe. This cost would double for low headroom work. The cost of injection labor and grout materials start at about $130 per cubic meter of improved soil for micro-fine cement grout, and about $200 per cubic meter of improved soil for sodium silicate grout. The cost of labor and materials is based on a 20% grout take, and a total grout volume greater than about 200 cubic meters.

Jet Grouting

In jet grouting, high pressure fluid jets are used to erode and mix/replace soil with grout. The general installation procedure begins with the drilling of a small hole, usually 90 to 150 mm in diameter, to the final depth. Grout is jetted into the soil through small nozzles as the drill rod is rotated and withdrawn. A continuous flow of cuttings from the jet points to the ground surface is required to prevent ground pressures from building up to the jet pressure, leading to ground deformation. The cuttings accumulate at the surface to form large spoil piles.

The main factors which influence the diameter and strength of jet grouted columns (Bell 1993; Covil and Skinner 1994; Stroud 1994) include:

1. *Soil Being Jetted.* Sand is easier to erode than silt. Thus, the width of the treated zone will be less in silt than in sand if no adjustments are made during the jetting operation. Irregular column geometries are likely in cobblely soils where larger particles limit the range of jetting, and in highly permeable, poorly graded gravel where grout may flow out of the jetted zone.

2. *Ground Water Conditions.* Grout could be leached out of soil by seepage, or attacked chemically or biologically.

3. *Grout Mix.* Grout, usually a water-cement mixture, must be matched to ground conditions to sufficiently strengthen and/or reduce permeability. The water-cement ratio of the *in situ* mix is a key index of strength, initial set time, and durability. Bentonite is usually added where low permeability is critical. Fly ash is added to control excessive bleeding and to improve durability.

4. *Jet System.* Single, double and triple jet systems are available. The single jet system only uses grout jets for both soil erosion and mixing. In the double jet system, the erosive effect is enhanced by shrouding the grout jet with compressed air. The triple jet system uses water jets shrouded by compressed air for soil erosion, and grout jets located lower down the drill stem for grout placement and mixing. The triple system permits greater flexibility in the control of the final properties of treated ground since the flow rate of the grout can be regulated independently of the erosive air-water jets. On the other hand, more waste cuttings are generated with the triple system than with the single system.

5. *Jet Pressure and Injection Rate.* High jet pressures and injection rates can erode soil to great distances. Pressure and nozzle diameter control the grout injection rate and the erosive energy. Typically, jet pressures range between 40 and 60 MPa, and nozzle diameters are 2 to 4 mm in diameter.

6. *Drill Rod Rotation and Withdrawal Rates.* The amount of grout injected and the degree of mixing depend on the rotation and withdrawal rates of the drill rod. Approximate relationships showing the variation of column diameter, withdrawal (or lift) rate, and jet system for granular materials are presented in Fig. 1.

7. *Column Sequencing.* A column of grouted soil without sufficient strength may be influenced by the formation of any adjacent columns. Sodium silicate is sometimes added to the grout mix to accelerate the set time.

The number and spacing of grout holes are also important factors contributing to the overall performance of jet grouted soil. Grout holes spaced too far apart will leave zones of ungrouted soil. Zones of poorly grouted soil are possible even with close spacings.

The cost to mobilize and demobilize jet grouting equipment is over $35,000 per rig. The cost of injection labor and grout materials starts at $320 per cubic meter of improved ground (Welsh 1992, 1995). This cost does not include handling, removal, and disposal of the large quantities of waste slurry that are produced. Depending on the jet system, the amount of waste slurry produced is 60% to 100% of the volume of treated soil.

In Situ Soil Mixing

In situ soil mixing is the mechanical mixing of soil and stabilizer using rotating auger and mixing-bar arrangements. As augers penetrate the ground, the stabilizer is pumped through the auger shaft and out the tip. Mixing bars attached to the auger shaft mix injected stabilizer and soil. Upon reaching the designed depth, a second mixing occurs as augers are withdrawn. The result is high strength or low permeability columns and panels.

The main factors which influence the effectiveness of *in situ* soil mixing (Stroud 1994; Taki and Yang 1991; JSSFME 1995) include:

1. *Soil Being Mixed.* Boulders, logs, and hard strata can make mixing impossible. Soil moisture increases water content of the soil-cement mix, resulting in lower strengths. Relatively high strength can be expected in clean granular soils, whereas low strengths in clayey soils.

2. *Ground Water Conditions.* Stabilizer could be leached out of soil by seepage, or attacked chemically or biologically.

3. *Stabilizer.* Cement is the primary agent for solidification. The water-cement ratio is an important index for strength, initial set time, and durability. Bentonite is added to increase workability and where low permeability is critical. Additives such as silicate, slag, and gypsum have been used for gaining strength in saline and organic soils. Retarding agents which extend set time have been used to make overlapping easier.

4. *Mixing Equipment.* The maximum possible treatment depth depends on auger size, number of augers, and torque capacity. Large augers (up to 4 m in diameter) require more torque, and are generally limited to depths less than about 8 m. For deeper mixing, a single-row of two to four auger shafts about 1 m in diameter is typically used.

5. *Grout Injection Volume.* Large volumes of stabilizer injected into the soil may cause ground to heave.

6. *Auger Rotation, Descent and Withdrawal Rates.* Slow auger rotation, descent and withdrawal rates increase consistency of soil mix.

7. *Mixing Sequence.* It is easier to overlap adjacent columns before the first column hardens.

It is very expensive to mobilize and demobilize a large multi-auger rig since there are just a few available in the United States (Welsh 1995). The approximate cost is $100,000 per rig and grout plant. The cost of grout materials and mixing starts at about $100 per cubic meter of improved ground for shallow mixing (say depths less than 8 m), and $200 per cubic meter for deep mixing (say depths between 8 and 30 m). The waste soil-cement produced during augering is about 30% of the treated volume.

Low Vibration Gravel Drain Pile

Ono et al. (1991) described a low vibration system for constructing gravel drain piles using a large casing auger. The casing is screwed downward into the ground, while simultaneously pouring water into the casing to prevent hydrostatic imbalance and sediment flow into the casing. Gravel is discharged into the casing upon reaching the final depth. As the casing is unscrewed, gravel is pushed out the end of the casing and compacted by a rod. One study showed that standard penetration resistances measured at the midpoint between piles after installation were about 5 blow counts higher than before installation. The most important factors affecting densification (Oishi and Tanaka 1992) are: the shape of the impact surface of compaction rod, the number of compactive strokes, and the stroke length. When drains are installed without the compaction rod, little densification occurs. It is important to note that with gravel drain systems liquefaction-induced settlement will be greater than with densification systems.

There are many factors which influence the effectiveness of drain pile systems (Barksdale 1987; Onque et al. 1987; JSSFME 1995):

1. *Soil Being Drained.* Soil permeability is the single most useful index. Other important parameters include fines content, type of fines, coefficient of volume compressibility, grain size, gradation, and density. It is unlikely that a soil with 30% fines (say D_{10} less than about 0.02 mm) can allow a permeability greater than about 10^{-3} cm/s.

2. *Ground Water Conditions.* Careful consideration of seepage conditions is required. Drain piles may create serious problems if applied in dams and in areas of artesian pressure.

3. *Drain Material.* Drain permeability and grain-size distribution are the most useful indexes. Gravel drains are constructed of poorly graded, coarse gravel. There is no easy way to install filters around gravel drain piles, and drains may clog when liquefaction occurs.

4. *Equipment and Installation.* Installation procedures may result in drain with more fines, and smearing of interbedded cohesive soil.

5. *Drain Diameter, Length, and Spacing.* Excess pore water pressures will dissipate quicker when drain spacings are small and drain diameters are large. The diameter of gravel drains is typically 0.4 to 0.5 m. Drain spacings of 0.8 to 1.5 m have been used.

No reliable cost information is available for the low vibration drain pile technique, since it has been used primarily in Japan.

CASE HISTORIES

Reported case studies of liquefaction remediation near existing lifeline structures are not common. The following eight case histories are presented to illustrate the application of ground improvement techniques to existing lifeline structures.

Settled Pipes at Waste Water Treatment Plant

A concrete effluent channel and three buried concrete pipelines connected to the channel at a waste water treatment plant had settled as much as 190 mm within two years after their construction (Scherer and Weiner 1993). Joints in the pipelines had opened as a result of the settlement. The diameters of the three pipes were 1.22 m, 1.52 m, and 2.13 m. It was concluded that settlement was caused by consolidation of a thick lens of very soft organic silt and clay supporting the channel and pipelines. To avoid costly excavation, dewatering, and problems posed by other utilities within the area, the concrete effluent channel was raised and supported with hydraulically driven steel mini piles located on the interior of the channel. The buried pipes were raised and supported with compaction grout piles. The compaction grout piles were installed on each side of the concrete pipe at joint locations or intervals not exceeding 3 m. The grout piles were designed to have a diameter of about 0.6 m and extend from the shale bedrock to the bottom of the concrete pipe, as illustrated in Fig. 2. The cutoff criteria for grout injection was set at a maximum pump pressure of 4 MPa, or when unwanted pipe lift or ground heave occurred. Grout injection volumes for the initial piles were only 0.023 m^3 per linear meter within a dense sand layer overlying bedrock. Thus, the tips of subsequent grout piles were located in the dense sand. Following the construction of the vertical grout piles, grout was injected beneath the center of the concrete pipe to lift the pipe, as depicted in Fig. 2. Finally, the interface between the vertical grout columns and concrete pipe was filled with additional grout to establish positive support. A total of fifty-two vertical and angle grout columns were installed.

Settled Railroad Embankment, Georgia

A section of rail line in northern Georgia passed through a sinkhole prone area (Brill and Hussin 1992). Compaction grouting was used to remediate conditions beneath the rail line. Grout holes were drilled at an angle from the eastern edge of the embankment 1.5 m into bedrock. Primary grout holes were spaced on 6 m centers, with injection volumes set at 7.5 m^3 per linear meter of casing for the first 0.9 m above bedrock, and 5 m^3 per linear meter in the soft/loose soil. These volumes were generally achieved. Secondary grout holes split the primary holes, with injection volumes set at 7.5 m^3 per linear meter for first 0.3 m above bedrock, 2.5 m^3 per linear meter for next 0.6 m, and 1.2 m^3 per linear meter in soft/loose soil. However, ground heave at the surface was typically observed before these target volumes were reached. Tertiary grouting was performed between the secondary holes when secondary injections seemed insufficient. A total of 1326 m^3 of grout was injected into 88 holes.

Riverside Avenue Bridge, California

From the report by Mitchell and Wentz (1991), the Riverside Avenue Bridge over the San Lorenzo River in Santa Cruz, California, is supported by reinforced concrete nose piers. The river was eroding away the soil beneath the south nose pier. Some settlement had occurred, causing damage to the bridge decking above. The river channel beneath the bridge and nose piers is lined by a concrete slab-apron. The upper 5 m of soil beneath the slab-apron consisted of saturated, loose to dense sandy gravel with a maximum size of 25 mm. Permeation grouting was considered the technique best suited for remedial work beneath the nose pier and slab-apron. Holes were drilled through the concrete nose pier and slab-apron for grout injection beneath and around the pier. Steel sleeve port grout pipes were passed through each drilled hole and vibrated or jetted into the granular soil. Grout consisting of sodium silicate (N grade) and micro-fine cement (MC 500) was injected through the sleeve port pipes and into the surrounding granular soil. The set time was controlled by adding to the grout mix less than 0.1% by volume of phosphoric acid. (No mention was made in the review about the environmental impact of using phosphoric acid or special handling procedures.) A total of 160 m^3 of grout was injected into 77 locations within the 15 day limit. During the 1989 Loma Prieta earthquake, the area experienced a peak ground surface acceleration of about 0.45 g. No settlement or detrimental ground movement was observed around the concrete slab-apron after the earthquake.

Containment Wall at Utility Crossings, Michigan

Jet grouting was used to construct sections of a vertical containment wall, up to 7.3 m deep, where underground pipes and other utilities crossed the barrier (Gazaway and Jasperse 1992). A typical section is shown in Fig. 3. Based on the results of a pilot study conducted at the site, the center-to-center spacing of the jet grouted columns was conservatively specified at 0.6 m for most of the work. Grout pressures were set at about 40 MPa. Drill rod rotation and withdrawal rates were set at about 1.3 r.p.m. and 0.4 m/min, respectively. To ensure closure beneath the larger diameter (up to 1.2 m) pipes, much slower rotation and withdrawal rates were used. Near the smaller and more fragile conduits, column spacings were tightened, and rotation and withdrawal rates were increased. Jet pressures of about 35 MPa were used for a few short periods in the immediate vicinity of particularly sensitive conduits. Approximately 530 square meters of containment barrier was installed by jet grouting. The jetting action caused no detectable damage to any of the underground utilities.

Tunnel Construction Beneath Rail Line, Switzerland

A new underpass was to be constructed beneath a busy rail line that separates the town of Fluelen from Lake Uri, Switzerland (Steiner et al. 1992). Two cut-off walls were needed to make dewatering effective and prevent excessive settlement beneath the tracks. Jet grouting was used to construct the two cut-off walls. It was determined from a pilot study that columns with diameters of 1.5 m and 1.2 m could be constructed with the double jet system and single jet system, respectively. The double jet system, i.e. grout jet shrouded with air, was used to construct columns with dip greater than 20°. The single jet system, which uses no air, was used for the flatter columns. Each cut-off wall consisted of three rows of columns. The outer row was constructed first, and the central row was constructed last with the axes of columns shifted so that they were positioned between the outer and inner columns. Cores taken from two borings drilled through the final wall revealed no evidence of joints between columns. Core specimens after 28 days exhibited an unconfined compressive strength between 6 and 10 MPa. During the two months of jet grouting work, the tracks underwent 4 mm of settlement, about the same rate observed before the work started. Measured settlement during excavation of the underpass was about 3 mm.

Tunnel Construction Beneath Airport Runway, Japan

A 70-m-wide underpass for vehicles was planned beneath a functioning airport runway in Japan (Ichihashi et al. 1992). The excavation would require dewatering, which could also cause settlement. It was determined that settlement and heave to the runway could not exceed 50 mm. Jet grouting was used to form soil-cement piles that extended to the bearing layer, and cut-off walls to prevent lowering of the water level outside the excavation. Since the soil could be improved by jet grouting through drill holes less than 220 mm in diameter, minimal damage occurred to the runway. To prevent settlement, a steel guide casing was first installed down to the top of the zone to be grouted. The grout pipe was then lowered down through the guide casing and advanced to the final depth, 2 m into the bearing layer. A tank containing a sand pump was attached to the casing guide at the ground surface to prevent waste slurry from flowing onto the runway. A triple jet system was used. Grout injection pressures varied between 30 and 40 MPa. Air injection pressures varied between 0.6 and 0.7 MPa. The drill rod was withdrawn at a rate between 50 and 100 mm/min. During the excavation of the tunnel, measured settlement and heave of the runway surface was less than 3 mm.

Quay Walls at Kushiro Port, Japan

Kushiro Port is located on the eastern shore of a northern island of Japan. As reported by Iai et al. (1994a), the port city experienced a peak horizontal ground surface acceleration of about 0.47 g during a magnitude 7.8 earthquake in 1993. Many quay walls were damaged when liquefaction occurred in the fill materials behind the wall. However, quay walls with treated backfill survived the earthquake without damage. Soil treatment behind the undamaged walls included sand compaction piles and gravel drain piles. The sand compaction piles had been formed to densify soils to within 13 m of the quay wall. The gravel drain piles had been installed by a low vibration procedure to within 5.5 m of the wall. They were 0.4 m in diameter and spaced 1.5 m on centers. Studies following the earthquake (Iai et al. 1994b) showed no evidence of sand migration into the gravel drains.

Highway Viaduct, San Diego

The soil beneath the I-805 viaduct crossing the San Diego River, California, is susceptible to liquefaction (Jackura and Abghari 1994). Estimates of possible liquefaction-induced horizontal ground displacement ranged from 1.4 m to 4.5 m, well above the maximum tolerable value of 0.8 m. A 15-m-wide underground buttress composed of stone columns was constructed between two bents at the toe of steepest ground slope to prevent ground displacement. Right-of-way restrictions limited the length of the buttress to roughly 85 m. While stone column (vibro-replacement) in not one of the five low vibration techniques, this case illustrates the application of other techniques when soil needing improvement is not obstructed by the lifeline and when work vibration will not cause damage.

One approach to reducing near-surface vibration has been pre-auger to the problem soil, and then lower the vibratory probe down the augered hole before applying the vibro-replacement technique. According to Baez (1995), the pre-auger approach has permitted ground improvement by vibro-replacement to within 3 m of many near-surface lifelines.

CONCLUSIONS

This paper presents five low vibration techniques that have been used for ground improvement near existing structures. These five techniques are: compaction grouting, permeation grouting, jet grouting, *in situ* soil mixing, and gravel drain pile. The factors which influence the effectiveness of each technique are reviewed. Of these five techniques, only jet grouting and *in situ* soil mixing can treat all liquefiable soil types. Compaction grouting may be marginally effective in treating silts. Chemical grouts cannot permeate soils with more than about 25% fines (silt and clay). It seems that gravel drain piles would be ineffective in ground with low permeability.

Upon reviewing the available cases studies, one quickly becomes aware that very little has been reported on ground improvement near existing lifeline structures. With great care and depending on their nature and condition, permeation and jet grouting could improve soil conditions immediately adjacent to lifelines. Compaction grouting could be applied beneath lifelines, but may not sufficiently compact soils immediately adjacent to them. The *in situ* soil mixing and gravel drain pile techniques could possibly be effectively employed a short distance away (say 1 to 3 m). Other less expensive ground improvement techniques, such as vibro-replacement through pre-augered holes, could be used to within about 1 m of many lifelines. A combination of techniques may provide the most cost-effective ground improvement solution.

The following recommendations are provided to identify areas that need further study.

1. Compile additional case studies of ground improvement near lifelines. These case studies should include detailed information about the condition of the lifeline, ground improvement procedures, verification techniques, and cost.

2. Compile additional case studies documenting the performance of improved ground during strong earthquake shaking.

3. Perform laboratory and field investigations to determine how much ground improvement is needed to protect lifeline structures.

4. Develop less expensive ground improvement techniques, since all the low vibration techniques reviewed are expensive to employ.

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Table 1 - Advantages and Constraints for Five Ground Improvement Techniques.

Advantage or Constraint	Compaction Grouting	Permeation Grouting	Jet Grouting	*In Situ* Soil Mixing	Gravel Drain Pile
Produces low levels of work vibration and noise	yes	yes	yes	yes	yes
Soil types not treatable	saturated clayey soils	soils with fines content of over about 25%	irregular geometries in cobblely soils and open gravel	boulders, logs, and hard strata can be a problem	soils with significant fines content and very low permeability
Treatment beneath existing structures possible	yes	yes	yes	earth structures yes ; others no	earth structures yes; others no
Small diameter drilling	yes	yes	yes	no	no
Low headroom work possible	yes	yes	yes	no	no
Selective treatment possible	yes	yes	yes	no	no
Intimate contact with structure possible	limited	yes	yes	no	no
Treatment at very low confinement possible	marginal	yes	yes	yes	yes
Without care, likely disturbance	significant ground movement; damaged pipes	significant ground movement; damaged pipes	significant ground movement; damaged pipes	significant ground movement; damaged pipes	damaged pipes
Quantity of waste produced	little	little	large	some	little
Prevents seismic-induced subsidence	yes	yes	depends on design	depends on design	no
Well-defined specifications required	yes	yes	yes	yes	yes
Engineered/observational approach required	yes	yes	yes	yes	yes
Quality control during installation required	yes	yes	yes	yes	yes
Other evaluations required	site pilot study	site pilot study; durability; creep; health and safety	site pilot study; durability	site pilot study; durability	site pilot study; seepage; clogging
Approximate cost, $\$/m^3$	20-30	200-300	300-400	100-200	not available

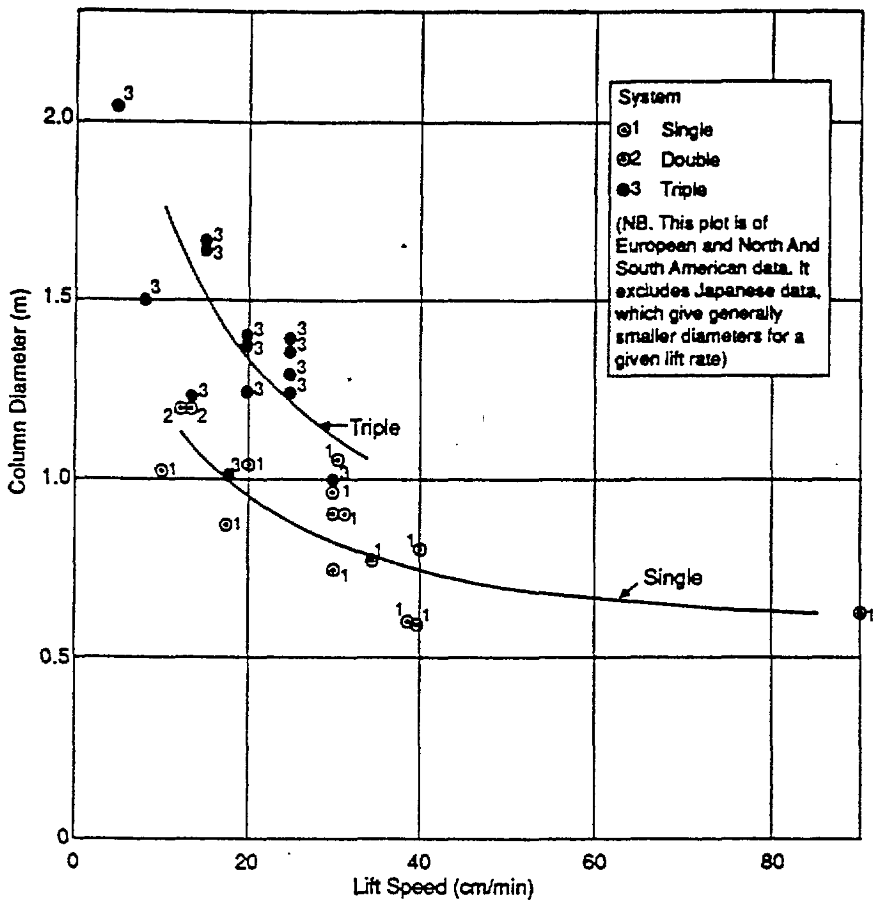

Fig. 1 - Variation in Diameter of Jet Grouted Column with Lift Rate in Sands (Stroud 1994).

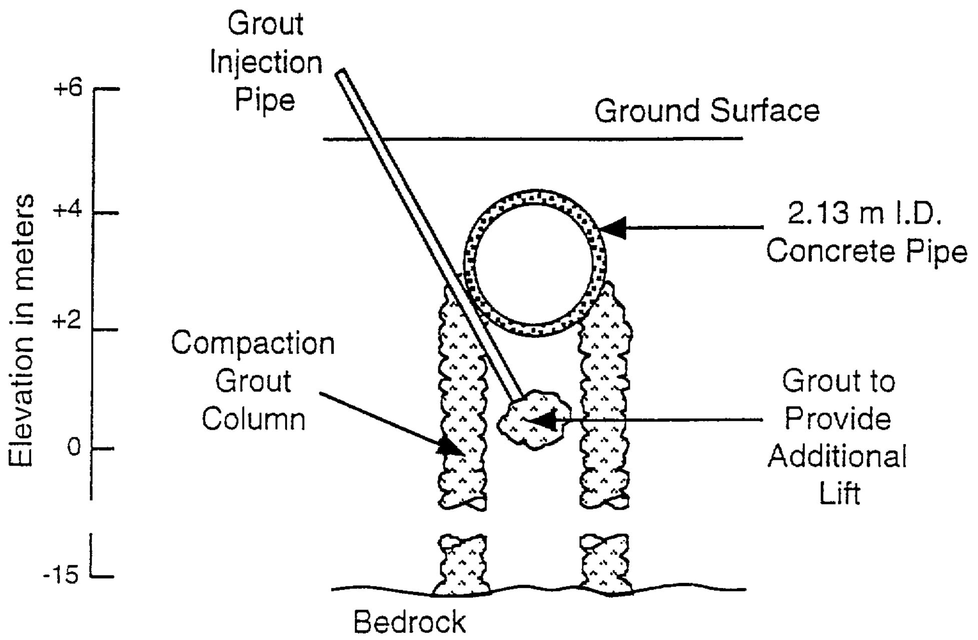

Fig. 2 - Underpinning and Leveling Settled Pipe by Compaction Grouting (after Scherer and Weiner, 1993).

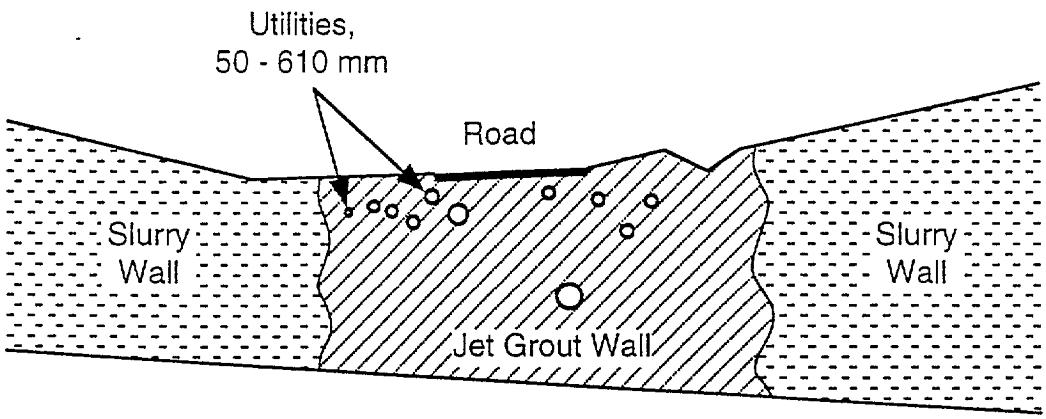

Fig. 3 - Construction of Cutoff Wall at Utility Crossing by Jet Grouting (after Gazaway and Jasperse, 1992).