SEISMIC VULNERABILITY OF A WATER DISTRIBUTION SYSTEM A CASE STUDY

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SYNOPSIS

This case study applies state-of-the-art earthquake engineering techniques plus the results of current research developed during a National Science Foundation sponsored project at Rensselaer Polytechnic Institute to assess the potential vulnerability of the distribution piping system of the Latham Water District, Albany County, New York to earthquake effects. The Latham Water District was considered typical of existing water distribution systems which have a variety of different types of pipe and joint systems reflecting the historical development of the service areas and the technology existing at the times of expansion.

Conservative analysis techniques indicate that a substantial portion of the Water District could experience earthquake related failures based on a 450 year return period earthquake (100 year economic lifetime and a 20% probability of exceedance). The potential failure area is over a deep, loosely consolidated, sand, silt and clay area that has filled in a pre-glacial river valley to a depth of 300-350 feet (90-105 m). In addition, distribution piping in this area is of a relatively non-flexible leadite or lead joint construction resulting in potential leakage under tensile forces.

This case study considers only wave effects and assumes major soil failures such as landslides, faulting, and zones of soil liquefaction do not occur.

Introduction of flexible joints for new portions of the system as well as replacement of damaged older portions tends to continually upgrade the system and decrease vulnerability.

RESUME

On présente la susceptibilité de ruine du réseau d'eau pour le district de Latham, dans l'état de New York. On trouve que pour un tremblement de terre avec une période de retour de 450 années, la région la plus susceptible à subir des dommages se trouve dans une section ayant un dépot d'environ cent mètres d'un mélange lâchement consolidé de gravier, d'argile et de silt. De plus, le système désuet a un grand potentiel de fuite car il consiste de plomberie rigide avec joints en plomb. On tient compte seulement de l'effet des ondes longitudinales et on néglige les possibilités de glissement, de liquéfactions, ou de failles géologiques.

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INTRODUCTION

This case study applies current research and state-of-the-art earthquake engineering to the study of the distribution piping system of the Latham Water District, Albany County, New York, to assess the vulnerability of the system to probable earthquake effects. Information in this case study is based on material developed during the SVBDUPS Project as partially noted in pertinent reports (2,8) and on meetings with Water District personnel, government representatives, and local engineering consultants.

The Latham Water District is considered typical of existing water distribution systems which have different types of pipe and joint systems reflecting the historical development of the service areas and the technology existing at the times of expansion. Latham was chosen for this case study due to its proximity to the research institution, its well documented facilities, and its readiness to cooperate in all phases of the project.

ENGINEERING ASPECTS OF THE WATER DISTRIBUTION SYSTEM (8)

The Latham Water District was formed in 1929. The water distribution system presently covers the major portion of the Town of Colonie, Albany County, New York, with a total area of approximately 50 square miles (130 square kilometers). The system has two major intakes, several well locations and several water storage tanks. One intake, built in 1953, is from the Stony Creek Reservoir in Saratoga County. The water is transported to a treatment plant, adjacent to the Mohawk River in the northern portion of the district, via a 24 inch (61 cm) diameter cast iron pipe with lead joints. A crossing of the Mohawk River is accomplished with submerged twin 16 inch (41 cm) diameter cast iron pipes with mechanical ball and socket connections. The second major intake is directly from the Mohawk River. A second treatment plant constructed in 1969 also exists at this river site. Major piping from this second plant is of prestressed concrete construction of 24 and 30 inch (61-76 cm) diameter. Both plants are located along the northern boundary of the water district approximately 2 miles (3 kilometers) apart. Figure 1 shows the relative location of the treatment plants as well as major distribution piping sized 10 inches (25 cm) diameter and above. Water storage tanks are located in seven general areas in the district with storage capacities ranging from 100,000 to 3,000,000 gallons (378,500 to 11,356,000 liters). Distribution of the storage tanks is noted in Figure 1.

Distribution piping ranges in size from 6 inch (15 cm) diameter to 30 inch (76 cm) diameter and includes a variety of pipe and joint materials. Initial distribution system construction consisted of cast iron pipes with mechanical joints of 6 through 16 inch (15-41 cm) diameters; cast iron pipe with lead joints used for reservoir supply lines and major distribution lines of 12 through 24 inch (30-61 cm) diameters; and cast iron pipes with "leadite" joints of 6 through 12 inch (15-30 cm) diameters. "Leadite" refers to a sulfur cement non-lead aggregate compound used as a lead substitute for water pipe joints from approximately 1929 through 1950 in the Latham Water District. The leadite joints used in the Latham Water District were relatively rigid and brittle materials as compared to the lead system. Indications are that chemical reactions of the joint material with the soil resulted in a corrosive environment near the bell and spigot joints with physical failures noted in many areas. Split sleeve systems are typically used to correct such joint failures. Beginning in 1950, new portions of the distribution system were typically installed with cast iron pipes and traditional lead joints with sizes ranging from 6 through 8 inch (15-20 cm) diameters. Circa 1967 rubber gasketed connections were initiated and since 1973 new and replacement pipe installations have generally been of ductile iron pipe with rubber gasket connections in sizes ranging from 6 to 24 inch (15-61 cm) diameters. In areas of corrosive soil such installations have been wrapped in polyethelene. A section of prestressed concrete pipe with cast in place concrete joints associated with the 1969 water treatment plant construction is located in the northeastern portion of the district and is of 24 and 30 inch (61 and 76 cm) diameter. An additional 24 inch (61 cm) diameter trunk section is located in the western portion of the district.

Generally, the historical development of the system has resulted in "cast iron pipe-leadite" joint systems in the older more densely populated regions of the town with the newer residential and commercial developments serviced by "cast iron-lead" and/or "ductile iron-rubber gasket" pipe-joint systems. All of the above mentioned pipe-joint systems are presently found in the network of major distribution piping.

Of the non mechanical pipe-joint systems, the "ductile iron-rubber gasket" combination provides the greatest flexibility followed by the "cast iron-lead" system. The "cast iron-leadite" and "prestressed concrete-cast in place" systems represent rigid joint conditions, with tensile failure likely in the leadite joint. The "prestressed concrete-cast in place" system is expected to act as a continuous pipe similar in action to the mechanical joint systems with joints capable of transferring tensile forces. Strength and/or flexibility requirements of pipe joint systems under earthquake conditions are noted in the seismic vulnerability portion of the paper.

Due to climatic conditions in the Northeast portion of the United States the majority of the water distribution piping in the Latham Water District is installed with a natural soil cover of 5 feet (1.5 m) from ground surface to the top of the pipe.

GEOLOGICAL AND SOIL CONDITIONS

The Latham Water District is underlain by shale bedrock of the Normanskill, Snake Hill and Indian Ladder formations (8). Overlying the shale bedrock are deposits of till and glacial outwash consisting of sands, silts and clays. Due to the presence of a pre-glacial river channel orientated roughly north-south in the central portion of the water district, depth to bedrock ranges from 300 to 350 feet (90-105 m) in this region as compared to 30 to 50 feet (9-15 m) in the western portion of the district and to occassional shale outcroppings in the eastern portion of the district. Bedrock contour lines shown in Figure 2* note the presence of this pre-glacial steep sided valley. Figure 3 notes a typical east-west cross section through the water district. Figure 4 presents soil isopachs, contours of equal soil depth above bedrock. Blow count results from numerous soil investigations in the area conducted for airport, roadway and sewer construction indicate Standard Penetration Resistance, N, values of 4-12 from the surface layers to depths of 250 feet (76 meters) with sample descriptions typically including sand, silt and traces of clay. The water table generally lies within 10 feet (3 meters) of the surface. Based on information from the soil borings it is concluded that the major portion of the soil is loosely consolidated, saturated, and of fine grained sands and silts.

EARTHQUAKE RISK

Reference 2 has developed the seismic risk for the Latham Water District area and presented the results in terms of annual risk, average return periods and probabilities of exceedance. The annual risk results are based on a study of historic earthquakes in the Northeast United States. Specifically, all historic earthquakes with epicenters within a circle of 160 kilometer radius and center at Latham, were used to establish an average earthquake occurrence rate of 0.204×10^{-4} earthquakes per year per square kilometer for Richter magnitudes of 2 or greater. The historical data was also used to develop a magnitude-frequency relationship for lower and upper Richter magnitudes of 2.0 to 6.3 for the Latham area. Finally, an attenuation relationship was incorporated into the analysis to account for the decrease in acceleration magnitude with distance between site and epicenter. The attenuation function selected used a set of conservative parameters with a probabilistic error term. The report notes an order of magnitude difference in attenuation effects is possible depending on the rela-

*Based on a map entitled "Bedrock Topography of the Albany-Lake George Area, New York", D. H. Bruehl, New York State Dept. of Trans., Albany, New York, April, 1969.

tionship used. Use of the conservative approach appears justified until additional attenuation data is developed for the Northeast portion of the country.

Using a Poisson process to model the random occurrence of a natural event, the annual risk (the probability that the ground acceleration will exceed a given value in one year) was developed and plotted as a graph. The reciprocal of the annual risk, the return period, was presented in a table giving maximum ground acceleration for return periods of 10 to 200 years. An additional table was presented listing recommended maximum ground accelerations for specific exceedance probabilities and economic lifetimes. This latter table provides the engineer with a design earthquake acceleration once the economic lifetime and the acceptable criteria for exceedance of the design parameter is established. The determination of the economic lifetime is seldom a problem but the philosophy of allowing for the possibility of exceeding a design parameter, especially when the loss of life may be involved, is a more difficult criterion to establish.

If q_a represents the annual risk (reciprocal of return period), T represents the economic lifetime in years, and p represents the probability of exceeding the event of annual risk q_a, the relationship is:

$$p = 1 - (1 - q_a)^T \qquad (1)$$

Hence, for a given economic lifetime the probability of exceedance can be developed for events of different return periods. In other civil engineering fields it is common to speak in terms of the return period event such as the "100 year storm". To illustrate the equation above, if a water distribution system with a 100 year economic lifetime is designed/analyzed for the 200 year earthquake the probability that the corresponding acceleration will be exceeded at least once in the systems lifetime may be calculated as: $p = 1 - (1 - 1/200)^{100} = 0.39$, and for the 100 year event the probability of exceedence increases to 0.63.

Since many civil engineers think more easily in terms of the design year event Equation 1 can be solved for q_a, the annual risk, in terms of a known economic lifetime T, and acceptable probability of exceedance, p, as follows:

$$q_a = 1 - (1 - p)^{1/T} \qquad (2)$$

then $1/q_a$ is equal to the return period or design year event. For example, a water distribution system with a 100 year economic lifetime and an acceptable probability, p, of exceeding the design earthquake acceleration of 0.20 results in designing/analyzing for the 450 year earthquake. The 450 year earthquake has a ground acceleration of 205 cm/sec^2 as determined from O'Rourke and Solla's report (2). Table 1 has been developed showing design year earthquakes as well as maximum ground accelerations for specific exceedance probabilities and economic lifetimes and is an adaption of Table 3 from Reference 2.

Discussions with the Superintendent of the Latham Water District have established a 100 year economic lifetime for the distribution system and an acceptable probability of exceedance of 0.20, corresponding to a 450 year design earthquake. The determination of the seismic vulnerability of the distribution system is based on this design/analysis guideline (i.e., the acceptable probability of exceedance), and hence such a decision should not be made hastily. There are few if any guidelines available for aiding the public administrator in making this decision.

SEISMIC VULNERABILITY OF THE DISTRIBUTION SYSTEM

Strain and Displacement Criteria

Using information regarding the engineering aspects of the distribution system, the local soil and geologic conditions, and the results of the seismic risk study, an analysis of the distribution system for earthquake effects can be accomplished using state-of-the-art techniques for the previously established design/analysis guideline of a 100 year economic lifetime and an acceptable probability of exceedence of 20% (i.e., the 450 year event).

Numerous studies have indicated that, with the exception of fault crossings, the soil strain and pipe strain during an earthquake are very much the same unless "slippage" occurs between the pipe and the soil. In this case "slippage" refers to a shear failure in the soil in a circumferential ring around the pipe when the pipe is subjected to axial forces. For the typical distribution system configuration with relatively long pipe runs and/or frequent junctions it is expected that the pipe strain and soil strain will be very nearly equal due to the availability of a large shear force transfer area in the long piping runs (analogous to a development length in reinforced concrete theory) and the mechanical "locking" effect of cross connections and valves.

Assuming that the shape of the seismic wave remains relatively constant and the wave velocity in soil is a lower bound for the wave velocity with respect to the pipeline, it can be shown that the axial soil strain, ε_a is equal to V_{max}/C_p, where V_{max} represents the maximum ground velocity during an earthquake and C_p is the soil pressure wave velocity. Assuming that the pipe axial strain, ε_a is also the pipe strain due to axial ground motion for continuous pipes.

The soil curvature, $1/R$ can be represented as:

$$\frac{1}{R} = \frac{A_{max}}{C_s^2} \qquad (3)$$

where A_{max} represents the maximum ground acceleration expected at the site and C_s is the shear wave velocity in the controlling medium.

Assuming the pipe curvature is the same as the soil curvature, the pipe flexural strain, ε_f, can be obtained by multiplying the curvature by the pipe radius, r. Thus,

$$\varepsilon_f = \frac{r}{R} = \frac{r\,A_{max}}{C_s^2} \tag{4}$$

and the combined pipe strain, ε_t, is conservatively:

$$\varepsilon_t = \varepsilon_a + \varepsilon_f \tag{5}$$

This combined strain is deemed to be conservative since the maximum values of acceleration and velocity will rarely occur simultaneously.

The product of the combined strain and modulus of elasticity of the pipe material gives the longitudinal stress due to earthquake effects.

For a continuous piping system such as the "cast iron-mechanically jointed" and the "prestressed concrete-cast in-place" pipe-joint combinations, the longitudinal stress represents the required capacity of the pipe and joints since no joint movement is available to relieve the strains. For a jointed system, acting in compression, with pipe segments in contact, a compressive longitudinal stress capacity of the above magnitude is required to prevent failure due to crushing of bells or local buckling of thin walled pipes.

For a flexible jointed system subject to tensile strains, pullout of the pipes at a joint will tend to relieve the axial strain and joint rotation will relieve the flexural strain. If the joint system can allow the necessary axial movement without losing its ability to maintain a watertight seal it will survive the earthquake ground displacements without leakage.

In a jointed system a conservative approximation for the required axial joint movement, Δ, can be made by multiplying the peak axial soil strain, ε_a, by the length of the pipe segment, L, thus,

$$\Delta = \varepsilon_a L \tag{6}$$

This approximation assumes uniform peak soil axial strain over the pipe segment length, assumes that the wave length of the seismic ground displacement is much larger than the pipe segment and further assumes the pipe is relatively rigid with respect to the soil. Under such assumptions, the displacement thus lumped at the joints will approximate closely the overall displacement configuration of the soil. The "apparent" strain in the pipe over a length, L, will then be $\Delta/L = \varepsilon_a$, even though the actual pipe segment has experienced little physical axial strain. An ideal joint, with no axial force resistance, would then have a requirement on joint movement equal to Δ.

Similarly, assuming the displacement wave shape is long relative to the pipe length, the required rotation capacity, θ, of a flexible joint/rigid pipe combination can be approximated as the product of the curvature and the length of the pipe segment

$$\theta = L/R \text{ radians} \tag{7}$$

For the case of actual "flexible" joints it is expected that both force transfer and joint movement will occur. For the "leadite" and lead joints it is expected that an initial tensile capacity will be available until the joint compound is pulled free of the bell and spigot connection. After this joint has been "cracked" no additional tensile force will be available during further earthquake strain cycling. The rubber gasketed joint will also have a limited load capability represented by straining of the gasket material until it is pulled free or "rolled" out of position. In this case study the tensile capacity as well as the rotational moment restraint of the non-rigid joints will be ignored as a conservative approximation. Continuous mechanically jointed and prestressed concrete pipe and joint systems will thus be analyzed for tensile and compressive strains/stresses due to a probable earthquake. Discontinuous, non rigid "leadite", lead and rubber gasketed systems will be analyzed for compressive strains/ stresses and tensile joint movement and rotation.

Wave Velocities

In order to evaluate the strains, displacements and rotations in the Latham Water District, it is necessary to establish the shear wave velocity, C_s, and the pressure wave velocity with respect to the pipe, C_p, that will govern the soil behavior at various locations within the distribution system. For a realistic range of soil parameters (1), the relationship between the wave velocities can be approximated as:

$$C_p = (3)^{1/2} C_s \tag{8}$$

Thus, the problem reduces to determining the shear wave velocity at various locations within the system. The shear wave velocity can be related to the shear modulus, G, and the mass density of the medium, ρ (1), by:

$$C_s = (G/\rho)^{1/2} \tag{9}$$

For rock, the values of shear wave velocity have been measured. For cohesionless soils, the shear modulus, G, for various levels of shear strain, is related to the soil relative density and overburden pressure.

Seed and Idriss (4) have presented the relation

$$G = 1000\ K_2\ (\sigma_m')^{1/2} \text{ psf} \tag{10}$$

where σ_m' is the effective confining pressure and K_2 is an empirical factor. Figure 5 in this paper is a representation of their curves to evaluate K_2.

Seed and Idriss (3) have additionally presented a relationship between Standard Penetration Resistance, N, and soil relative densities for various overburden pressures. This relationship is presented as Figure 6 and is credited to the U.S. Bureau of Reclamation.

By reference to Figure 3, the calculation of the wave velocity in the water district may be simplified by dividing the case study area into relatively "deep" and "shallow" zones of soil overburden. It is assumed that the wave velocity over the preglacial valley areas will be controlled by the deep layers of the sand, silt and clay mixture. In the "shallow" area, the wave velocity will be assumed to be generally controlled by the underlying shale bedrock. Several hills noted in the surface topography of the "shallow" area are localized and represent former sand dunes. These high points may be noted as the 100 foot isopachs of limited area in Figure 4. For purposes of this study, the wave velocity at these localized high points has been assumed to be controlled by the "shallow" zone bedrock. The 100 foot isopach, bordering the preglacial valley, has been selected to generally delineate the two major zones.

Coupling the technical information on wave velocities with the soil conditions in the Latham Water District, an approximation can be made for the shear wave velocity in areas controlled by the deep soil layers. Blow count information at various locations within the Latham Water District indicated Standard Penetration Resistance, N, values of 4-12 to depths of 250 feet (76 m) in the sand, silt and silty clay glacial lake deposits. The water table was generally found to be within 10 feet (3 meters) of the surface. Depending on overburden pressure, Figure 6, gives a range of relative densities from 30-70% with higher relative densities nearer the surface. Assuming an average Standard Penetration of 8 and a soil depth of 10 feet to represent the zone of pipe burial and a soil weight of 120 lbs/ft^3, (18.85 kN/m^3), one can obtain a relative density of approximately 60% from Figure 6.

Previous calculations have indicated that a shear strain in the order of 10^{-1}% is reasonable. Entering Figure 5 with a relative density, D_r, of 60% and a shear strain of 10^{-1}% one finds the value of K_2 is relatively insensitive to relative density. Using K_2 value of 14, and σ'_m, the effective mean principal stress, of 1200 psf (57.5 kPa), the shear modulus, G, is 0.485 x 10^6 psf (23.2 MPa). Thus, the shear wave velocity, C_s, may be calculated as $(G/\rho)^{1/2}$=361 ft/s (110 m/s).

Using an alternate approach and assuming the shear wave velocity will be controlled by the middepth of the average soil layer thickness in the pre-glacial valley results in a depth of 100 feet (30.5 m), an effective pressure of 6380 psf (305.5 kPa), a relative density of approximately 35%, a value of K_2 of approximately 12 and a value of G equal to 0.96 x 10^6 psf (46.0 MPa), giving a shear wave velocity of 508 ft/s (155 m/s).

Based on shear wave velocities observed in other locations, this latter value appears more reasonable and still represents a relatively loose material such as the sediments noted in the boring logs.

For simplicity, a shear wave velocity of 500 ft/s (152 m/s) has been used in further portions of this study to represent the approximate magnitude of the shear wave velocity in the deep cohesionless layers within the pre-glacial valley. For the shallow soil depths adjacent to the valley, the shear wave velocity has been assumed to be controlled by the underlying shale bedrock. A shear wave velocity of 2500 ft/s (762 m/s) has been assumed for this value.

To delineate the Water District into "shallow" zones controlled by the shale bedrock shear wave velocity and "deep" zones controlled by the loosely consolidated sands and silts a soil depth of 100 feet (30 m) has been selected to represent the edge of the pre-glacial valley. A rationale for using the two shear wave zones is obtained by assuming the wave velocity will be controlled by the medium depth at approximately 1/2 wavelength. For a velocity of 500 ft/s (152 m/s) and an extreme period of 2 seconds this would indicate an approximate 125 foot (38 m) depth. Hence, the shallow zones would be governed by the underlying shale in this extreme case.

For the two zones the pertinent wave velocities are then:

	C_s	C_p
Shallow	2500 ft/s (762 m/s)	4330 ft/s (1320 m/s)
Deep	500 ft/s (152 m/s)	866 ft/s (264 m/s)

Ground Acceleration and Velocity

O'Rourke and Solla (2) have presented recommended ground accelerations for the Latham area. Based on a 100 year economic lifetime and 20% probability of exceedance the probable maximum ground acceleration in rock at the Latham Water District site is 0.21 g. To develop the acceleration and velocities that will control strains in the piping system it will be necessary to determine the ground accelerations in the region of pipe burial. Seed, et al (5) have presented a relationship between "maximum acceleration" and "maximum acceleration in rock" for four basic soil conditions. Figure 7 is a representation of this relationship. For the "shallow" zone in the Latham case study the maximum ground acceleration has been assumed to be represented by the "stiff soil"curve and has been kept at 0.21 g. For the "deep" zone, the "deep choesionless soils" curve indicates an approximate value of 0.17 g for the effective ground acceleration.

Relationships between maximum velocity and maximum acceleration have been presented by Newmark and Seed and are summarized in Reference 5 and in Table 2. For this study a value of 50 in/s/g (127 cm/s/g) has been selected to represent the glacial soil deposits. Thus, maximum ground velocities are 10.5 in/s (26.7 cm/s) for shallow deposits and 8.5 in/s (21.6 cm/s) for deep deposits.

Maximum Strains, Stresses and Displacements

Synthesizing the above information results in Table 3 indicating the maximum strains, stresses and displacements for the idealized "shallow" and "deep" soil conditions. Calculations are based on a pipe segment length of 20 feet (6.1 m) and a diameter for flexural strain calculations of 30 inches (76 cm). Even with the use of the largest pipe diameter, the effect of curvature related flexural strain is an order of magnitude less than axial induced strain.

"Shallow" areas, controlled by the shale shear wave velocities, would develop stress and/or displacement requirements of a tolerable magnitude for all pipe joint combinations of initial sound condition. Pipes and joints severely weakened by corrosion and non-earthquake loadings could be potentially damaged by these additional strength requirements but such conditions are not considered in this paper.

"Deep" areas controlled by the 100-350 ft (30-107 m) layer of loosely consolidated sands and silts would develop appreciable stresses in the rigid pipe modes (rigid joints & joints in contact in compression) and would require moderate values of tensile joint movement in the range of 1/4 inch (0.6 cm). Sound pipe and joint systems would normally have the ability to resist the axial stresses developed during the probable earthquake since they are normally lightly stressed in the axial direction with the exception of flexural stresses due to concentrated surface loadings or non-uniform embedment. Wang and Fung (7) indicate a considerable reserve stress capacity in the axial direction for normally designed pipes. Since the flexural strain is so low, its stress effect on a locked joint would appear tolerable. The noted relative rotation between pipe segments of 0.03° is minor and well within the leakage range presented by Untrauer, et al (6) for cast iron pipe with lead caulked joints.

The required axial motion of approximately 1/4 inch (0.6 cm) appears to be more critical. It is expected that such a movement would open both the "leadite" and lead caulked joints and result in numerous leaks in the distribution system. For the ductile iron-rubber gasket joint, manufacturers literature indicates axial motion up to 3/8 in. (1 cm) is tolerable without leakage or twisting of the gasket*. Assuming tensile joint movement does not undergo a "growth" phenomena under repeated cycling, a tendency to continually open under tensile forces but not to completely close under compression, the rubber gasketed joint systems appear to be capable of surviving the probable earthquake.

Damage Assessment

Based on the above capabilities of pipe-joint systems, probable joint failure would be expected to occur in several areas where lead and leadite joint systems coincide with the deep cohesionless soil deposits.

Major distribution lines matching these failure criteria have been cross hatched on Figure 8. From this figure it is noted that potential pipe failures would divide the district into two relatively undamaged zones separated by failures of main distribution lines in the central region. In the north portion the reservoir supply line of "cast iron-

*Manufacturers literature Form 371, "Dresser Couplings", Dresser Manufacturing Division, Dresser Industries, Inc., PA lead" configuration could experience leakage. In the south portion of the central region a relatively densely populated, older residential section serviced by 6 inch (15 cm) and 8 inch (20 cm) diameter "cast iron-leadite" distribution lines would be expected to suffer a loss of water due to the probable earthquake event. This area is noted by denser grid hatching on Figure 8. This area represents the greatest potential for damage and loss of life and property from diminished fire fighting capabilities due to a loss of water supply. Disaster control planning should anticipate a major water distribution failure in this populated area. Contingency plans should be prepared for zoning off the east and west portions of the district from the probable central failure region. In addition, techniques for rapidly restoring the reservoir supply should be developed.

Additional investigations should be made of the treatment plants and water tanks to determine their behavior during the design earthquake. A loss of either treatment plant could result in serious long term water shortages. Loss of water tanks would lead to immediate loss of fire fighting capability.

This report has been limited to a vulnerability study based on assesing the impact of an earthquake on a water distribution network assuming major soil failures, such as land-slides and zones of soil liequfaction, do not occur. Since the pre-glacial valley area contains a relatively deep deposit of loosely consolidated sands, silts, and clays, future investigations should analyze this region for potential liquefaction and consolidation during a probable earthquake event. Such an overall soil failure would tend to overshadow the effects analyzed above.

In addition, a more detailed analysis should investigate wave reflection and refraction at the "shallow"/"deep" interface and these effects on piping crossing through this region. Amplification could result in greater magnitudes of strain and joint movement in these areas.

SUMMARY

State-of-the-art earthquake engineering and recent research have been applied to assess the potential vulnerability of a typical Water District to a probable 450 year return period earthquake. This study was limited to the effects of a probable earthquake on the distribution line piping only and assumed soil failures did not occur. The local soil and geological conditions were such that the district could be separated into two zones representing a "shallow" area controlled by shale bedrock behavior and a "deep" area controlled by the behavior of loosely consolidated layers of silt, sand and clay.

Conservative analysis techniques indicated the "shallow" zone would experience little if any damage from a probable earthquake within a 100 year time period assuming that a 20% probability of exceedance is acceptable. The same conservative techniques indicate the potential for damage within the "deep" area with the possibility of distribution line failures dividing the district and resulting in a loss of water

supply for a relatively densely populated residential area.

Actual numerical values presented in this paper are assumed to be conservative. Further research into attenuation coefficients for the Northeast and a more detailed study of shear wave velocities in the Water District could significantly alter these results.

It has been recommended that local disaster planning consider the potential for distribution line failure in this Water District and develop appropriate contingency plans.

Joint failures in the Water District, due to non-earthquake effects, are currently repaired with relatively flexible split sleeve couplings and new distribution lines are typically of the "ductile iron-rubber gasket" configuration. It has been recommended that this practice continue since such repair/replacement joint systems tend to reduce the overall vulnerability of the system.

Since the Latham Water District is considered typical of existing water distribution systems, it is expected that such conservative analysis procedures could indicate potentially vulnerable areas in water systems in other communities. Knowledge of such critical areas could result in local disaster planning and a reduction in loss of life and property if a major earthquake event were to occur.

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8. Wang, L.R.L. and Lavery, W.T., "Engineering Profile of Latham Water District, Albany, New York", Dept. of Civil Engineering, Rensselaer Polytechnic Institute, Troy, N.Y., Technical Memorandum (SVBDUPS Project) No. 3, April, 1978.

Table 1 - Design Year Event and Recommended Maximum Ground Acceleration (A_{max}) For Specific Exceedance Probabilities and Economic Lifetimes

Economic Lifetime T (years)	P = probability of design year event being exceeded at least once in T years		
	P = 0.05	P = 0.10	P = 0.20
25	488 yr (0.23 g)	238 yr (0.18 g)	112.5 yr (0.155 g)
50	975 yr (0.275 g)	475 yr (0.23 g)	225 yr (0.18 g)
100	1950 yr (0.34 g)	950 yr (0.27 g)	450 yr (0.21 g)

Table 2 - Relationship Between Maximum Velocity and Maximum Acceleration for Various Soils

Geologic Conditions	V_{max}/A_{max}	Ref.
Rock	$26 \frac{in/sec}{g}$	Seed (5)
Stiff Soil	$45 \frac{in/sec}{g}$	Seed (5)
Deep Cohesionless Soil	$55 \frac{in/sec}{g}$	Seed (5)
Rock	$24 \frac{in/sec}{g}$	Newmark (1)
Alluvium	$48 \frac{in/sec}{g}$	Newmark (1)

TABLE 3 - Maximum Strains, Stresses and Displacements

Relative Depth to Bedrock	a_{max}	v_{max}	C_s	C_p	Strains			Stresses*		Displacements	
					ε_a	ε_f	ε_t	σ_{ci}	σ_{pc}	Δ	θ
	g's	in/s (cm/s)	ft/s (m/s)	ft/s (m/s)	(10^{-3})			ksi (MP_a)		in (cm)	degrees
Shallow<100 ft (shale controls)	0.21	10.5 (26.7)	2500 (762)	4330 (1320)	0.24	-	0.24	3.36 (23.2)	0.72 (5.0)	0.06 (0.15)	-
Deep > 100 ft	0.17	8.5 (21.6)	500 (152)	866 (264)	0.82	0.03	0.85	11.9 (82.0)	2.55 (17.6)	0.20 (0.51)	0.03

*σ_{ci} = cast iron stress, $E = 14 \times 10^3$ ksi (96.5 GPa)

σ_{pc} = prestressed concrete stress, $E = 3 \times 10^3$ ksi (20.7 GPa)

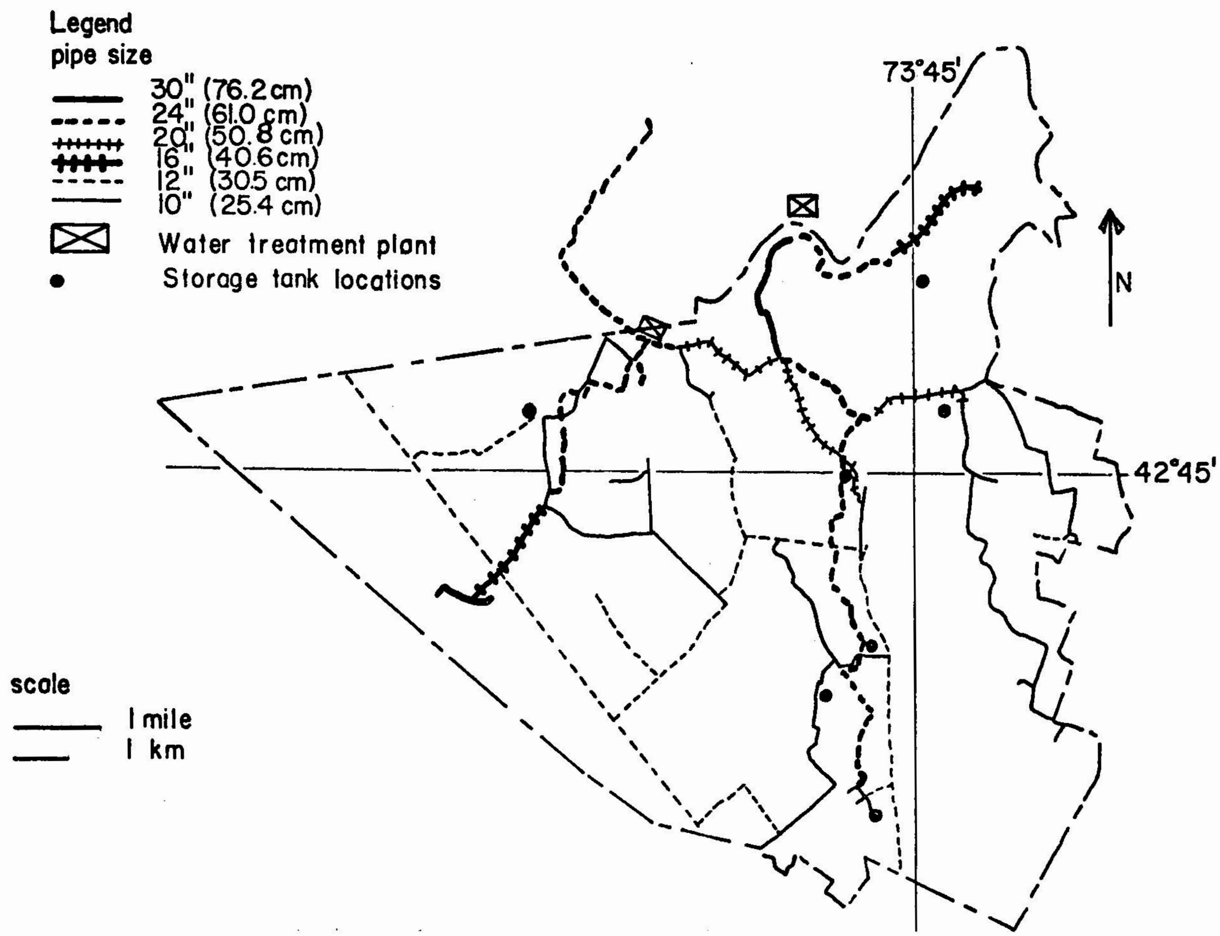

FIGURE 1 - LATHAM WATER DISTRICT DISTRIBUTION MAP

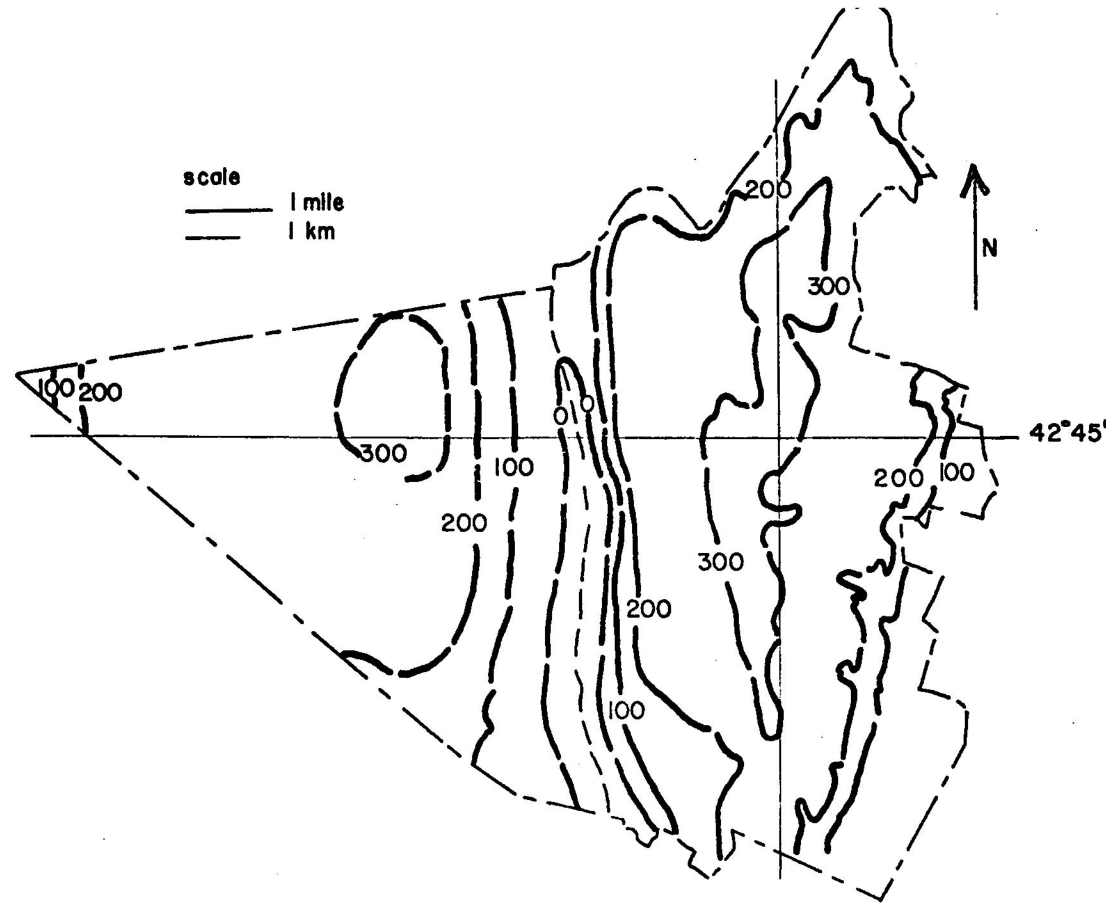

FIGURE 2 - BEDROCK CONTOURS (USGS FEET)

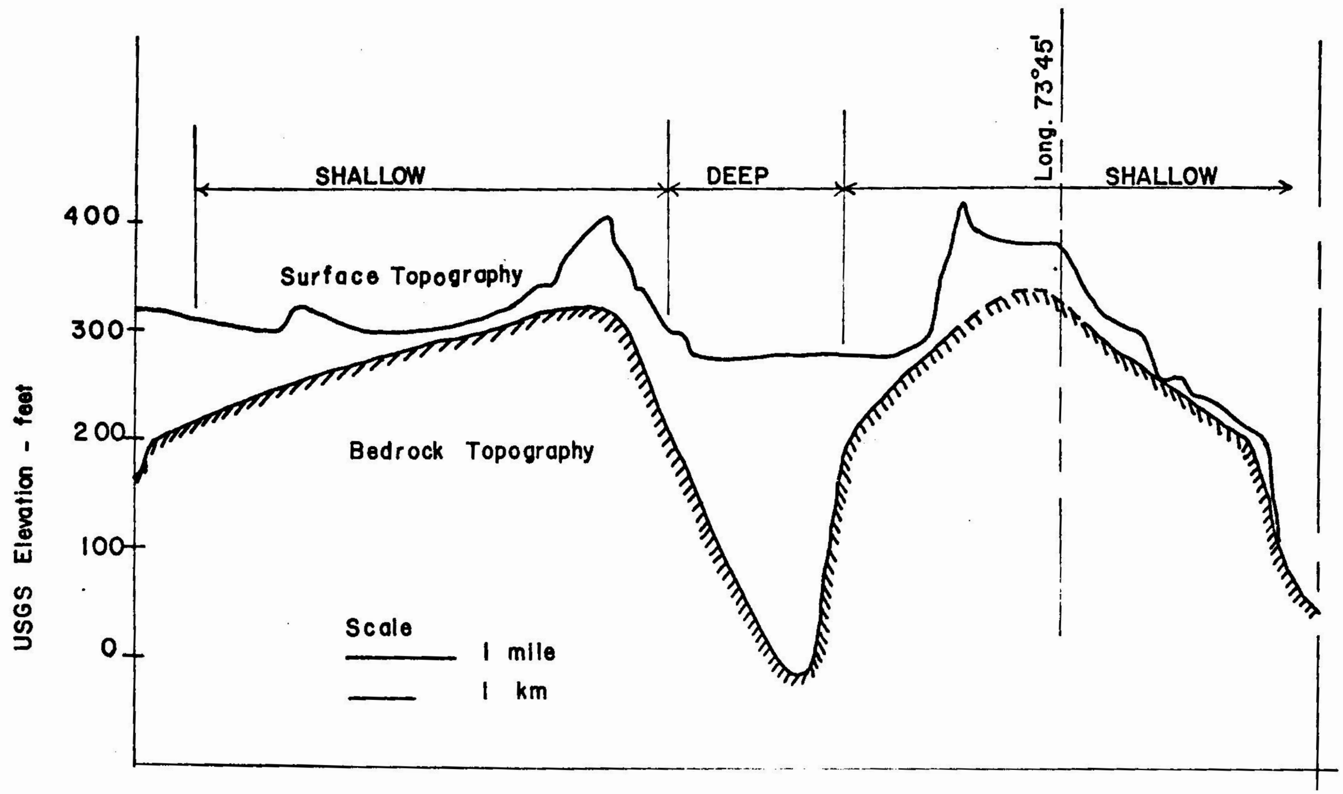

FIGURE 3 - SOIL-BEDROCK PROFILE LATITUDE 42°-45'

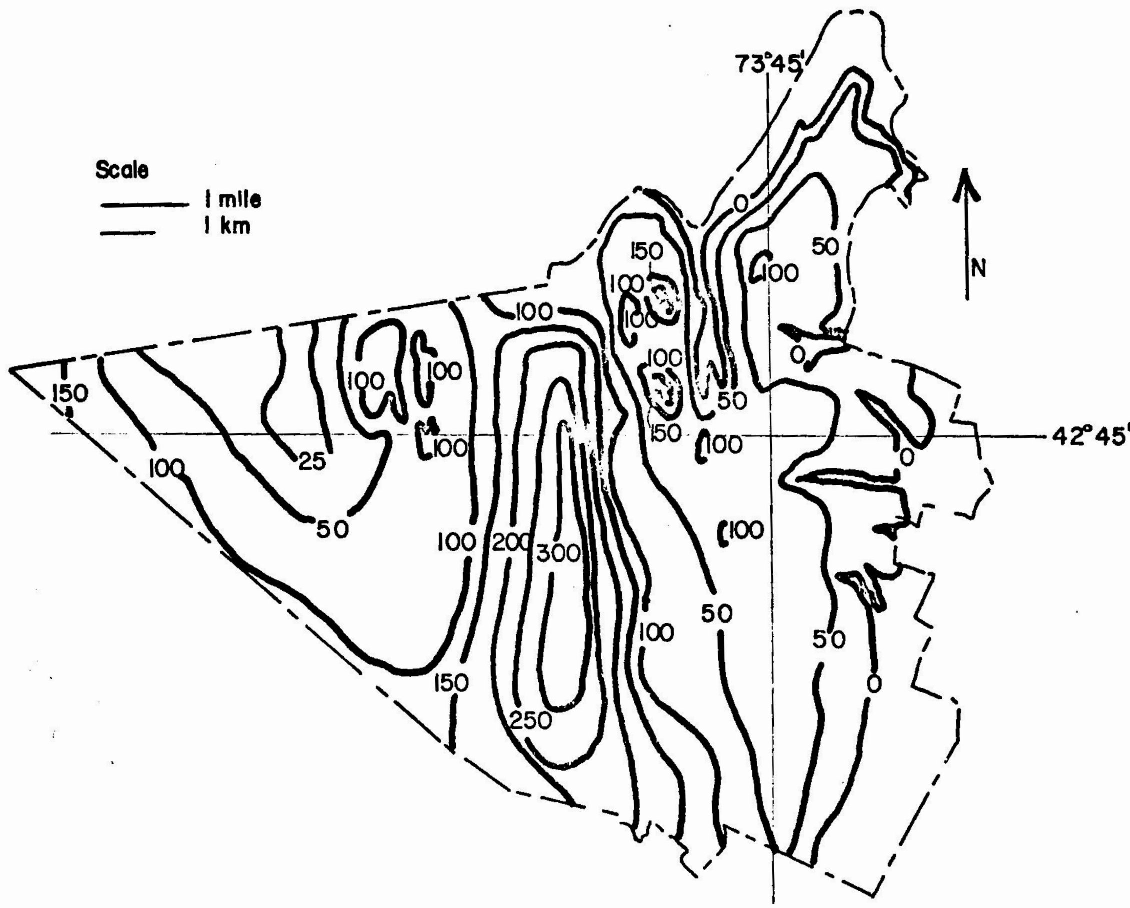

FIGURE 4 - LINES OF CONSTANT SOIL DEPTH - SOIL ISOPACHS (FEET)

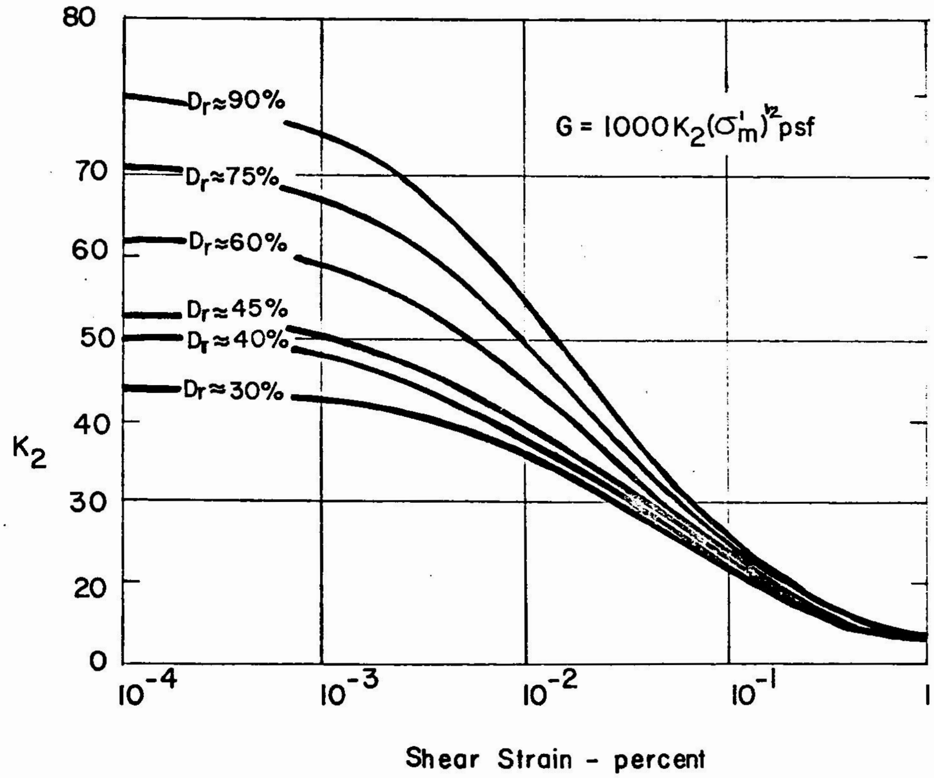

FIGURE 5 - SHEAR MODULI OF SANDS AT DIFFERENT RELATIVE DENSITIES (After Seed & Idriss (4))

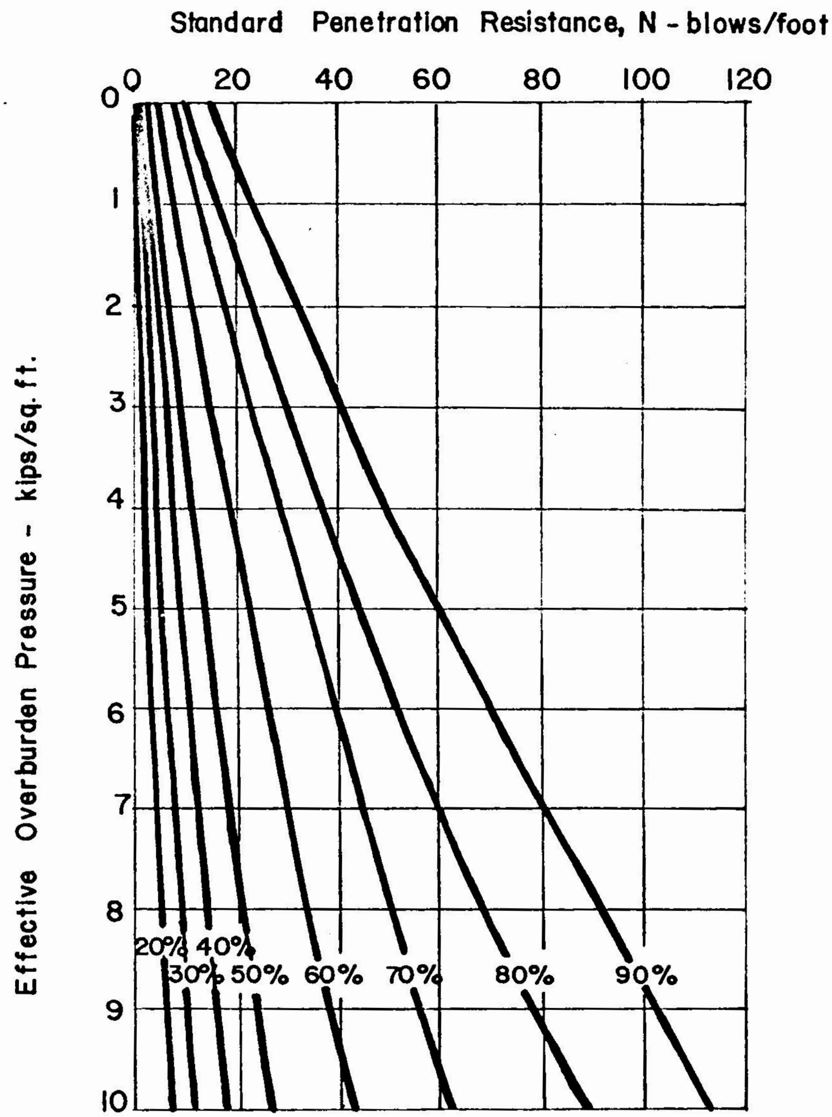

FIGURE 6 - RELATIONSHIP BETWEEN STANDARD PENETRATION RESISTANCE, RELATIVE DENSITY AND EFFECTIVE OVERBURDEN PRESSURE.

(After Seed and Idriss (3))

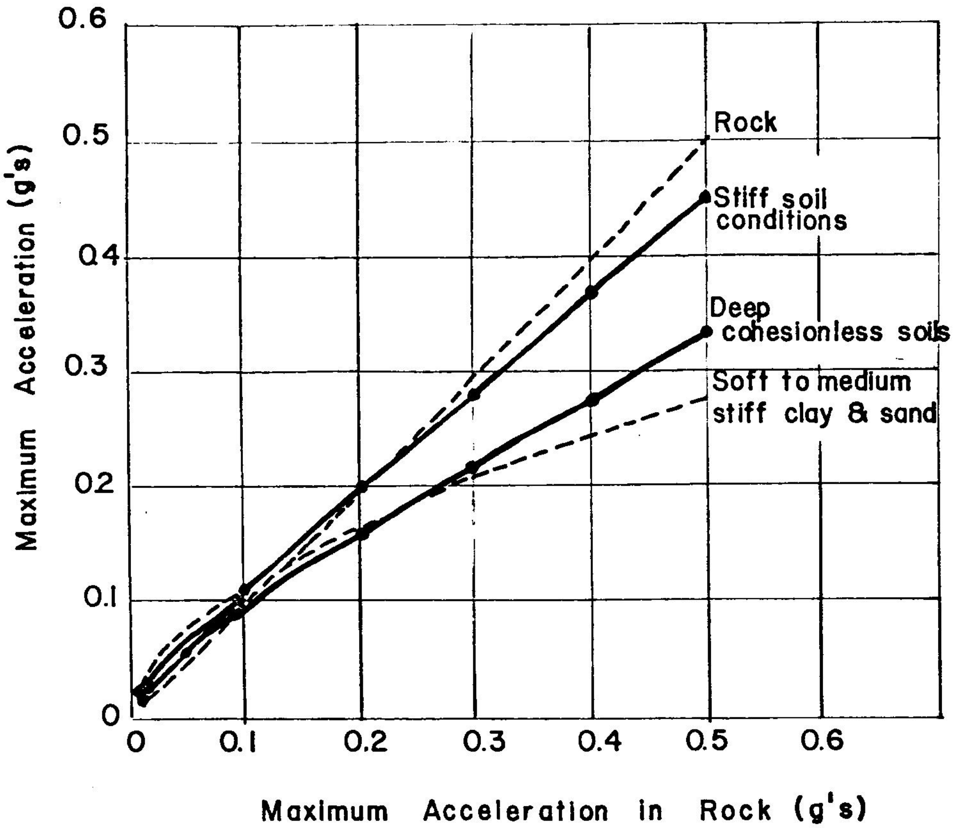

FIGURE 7 - EFFECT OF SOIL ON MAXIMUM ACCELERATION

(After Seed, Murarka, Lysmer & Idriss (5))

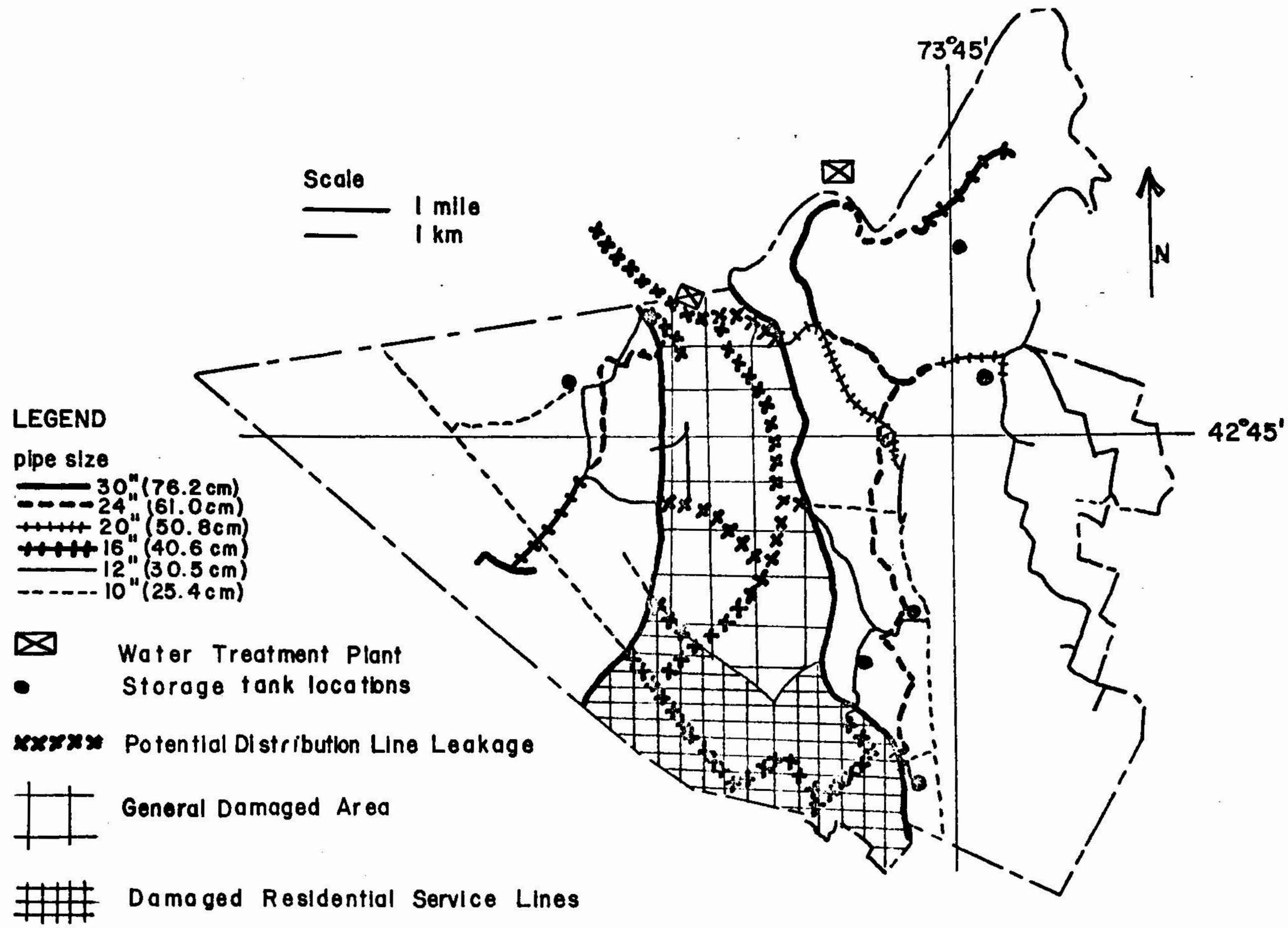

FIGURE 8 - POTENTIAL PIPELINE DAMAGE (100 FT ISOPACH)