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ANALYTICAL DAMAGE ESTIMATES FOR CONCRETE PIPELINES

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ABSTRACT

Analytical fragility relations for a class of concrete pipe subject to seismic wave propagation are developed. Tests were conducted at Rensselaer to determine the load-deformation characteristics and failure criterion for rubber gasketed concrete pipeline joints. This information was implemented in a mathematical model to assess the vulnerability of concrete pipelines to seismic wave propagation damage. For tensile ground strain, the joints accommodate nearly all the imposed ground deformation and hence are vulnerable to joint "pull-out" and leakage. For compressive ground strain, the joints are subject to high compressive forces which might lead to the crushing of these joints. Statistical data on these response parameters of interest are obtained using Monte Carlo simulation. Based on the response parameters and failure criterion, fragility curves for concrete pipelines are established. These relations are benchmarked against observed water transmission pipe damage in Mexico City occasioned by the 1985 Michoacan Earthquake. In this benchmark case history, the analytical fragility relations correctly identify the most predominant observed failure mode. In addition, they provide a reasonable estimate of the expected damage rate in repairs per kilometer.

KEYWORDS

seismic wave propagation damage, seismic vulnerability evaluations, rubber-gasketed concrete pipeline joints, fragility relations, lab tests.

INTRODUCTION

Seismic damage to buried pipelines is typically due to some combination of wave propagation effects and permanent ground deformation (PGD) effects. As used herein, wave propagation refers to the transient strain induced in pipe due to traveling ground waves while PGD refers to landslides, lateral spreading due to liquefaction, and other forms of non-transient ground movements. Estimation of seismic damage to buried pipeline systems has, in the past, been based almost exclusively on empirical relations developed from post-earthquake reconnaissance. O'Rourke and Liu (1996) present a brief summary of most of the existing empirical relations for both wave propagation and PGD effects.

Herein, the response and vulnerability of buried concrete pipelines with rubber gasketed joints to wave propagation effects is considered. Laboratory tests were used to characterize the axial stiffness and leakage potential of the pipeline joints. This information was then used to develop mathematical models for straight portions of a pipeline network subject to simulated pseudostatic tensile and comparative ground strain. The mathematical models provide statistical information (mean and standard deviation) for the response parameters of interest, specifically, the relative displacement and axial force at the pipeline joints. Fragility relations (expected number of pipe repairs as a function of ground strain) are developed by combining pipe response information with damage criterion for the failure modes of interest. For the relatively large diameter concrete pipe considered herein, seismic damage typically occurs at the joints. Hence, the failure modes of interest are joint leakage (axial pullout due to tensile ground strain) and crushing at the joint (telescoping due to compressive ground strain). The usefulness and relative accuracy of this analytical approach is established by a comparison of predicted and observed damage to concrete water transmission pipelines in Mexico City caused by the 1985 Michoacan earthquake.

LABORATORY TEST RESULTS

The overall objective of the test program at Rensselaer was to determine the characteristics of concrete pipe joints thought to most strongly influence seismic behavior. These characteristics are the force-deformation behavior of the joint subject to axial tension or compression, and the level of joint deformation or load leading to failure. For the case of axial extension, failure is quantified by the amount of joint extension which results in leakage. Alternately for axial compression, failure is quantified by the magnitude of the axial compressive force resulting in crushing at the joints.

The reinforced concrete cylinder pipe (RCCP) segments used in the test program were specially manufactured by "Ingenieria Y Construcciones Hidraulicas, S.A." (ICHSA) of Mexico City. The pipe had a nominal inside diameter of 30 inches. The pipewall consisted of an inner concrete lining thickness of about 1 inch, a steel cylinder thickness of 0.25 inches, and an outside concrete thickness of about 3 inches. The test segments were about 46 inches long with one bell end (i.e., female end of the joint) and one spigot end (i.e., male end of the joint). In addition, ICHSA supplied a 16 inch long transition segment with spigots at both ends and two bulkheads (bell type) for closing the ends of the simulated pipeline.

The pipe segments, transition segment and bulkheads were supported by a specially fabricated steel test frame. Axial loads, both tension and compression, were applied at one end of the pipeline by an actuator powered by a hydraulic pump. A pressure gauge attached to the pump was used to monitor the applied load. Joint displacement (both extension and contraction) were measured by two LVDT's placed at opposite sides of the joint.

FORCE-DEFORMATION BEHAVIOR

For tension loading, there were two distinct regions in the force-deformation relation for the rubber gasketed joints. For small values of the applied load, below a slippage threshold, a non-linear softening behavior was observed. Above the slippage threshold, a constant force resulted in ever increasing joint displacement.

It was found that the force corresponding to slippage in the tension tests, F_s, was an increasing function of the internal water pressure, p, while the joint displacement corresponding to slippage, Δ_s, was not a function of internal water pressure. At zero internal pressure, the non-linear behavior was characterized by

$$F = 5.16 \ \Delta^{.495} \qquad \Delta < \Delta_s \tag{1}$$

where F is the applied tensile joint force in kips and Δ is the joint displacement in inches. Irrespective of internal pressure, slippage began for a joint displacement about 0.16 inches on average. At zero internal pressure the corresponding slippage force was, again on average, about 2 kips. Note that this value of about 2 kips is essentially the same as the force needed to initially close the joints when the segments were being assembled into a simulated pipeline. As mentioned above, the slippage force F_s was an increasing function of internal water pressure. A statistical analysis suggests the following relation for internal pressures up to 30 psi, which was the range of the test data:

$$F_s = 2.08 + 0.136 \ p^{.93} \tag{2}$$

where F_s is in kips and p is in psi.

The increase in F_s with increasing p is explained by the fact that internal water pressure causes additional pressure between the rubber gasket and the bell and spigot surfaces due to Poisson ratio effects. This increased pressure results in increased friction and hence larger slippage forces.

Due to the experimental setup, all of the load-deformation tests in compression were dry (i.e., no internal water pressure). These tests indicate that the joint behaves in a sigmoidal fashion before "lock up." The joint displacement (compressional) at lock-up typically ranged from 0.125 to 0.375 inches with corresponding loads of 3.5 to 4.5 kips.

FAILURE CRITERION

For wave propagation resulting in tensile ground strain, the failure mode of interest is leakage due to joint pull-out. The joint displacement (extension), Δ_u, leading to leakage was found to be a mild function of the internal water pressure. For the 30 inch nominal diameter pipe in the Rensselaer test program, the leakage displacement, Δ_u, was on average, about 1.85 inches for zero internal pressure. The variation (best straight line fit) with internal pressure, again for internal pressures up to 30 psi, was found to be

$$\Delta_u = 1.83 - 0.0066p \tag{3}$$

where Δ_u is in inches.

However, the scatter in observed values of Δ_u for a given internal pressure were comparable to the variation with internal pressure suggested by Eqn (3). As a result, all Δ_u values were used to determine the probability of leakage. In order to project leakage values to other pipe diameters, Δ_u was normalized by the depth of the joint (3.25 inches for the 30 inch nominal diameter pipe). The resulting plot of probability versus normalized joint extension is shown in Figure 1. That is, no leakage is expected if the joint is pulled out less than 48%. Conversely, leakage is certain if the joint is pulled out more than 57%. For wave propagation resulting in compressional ground strain, the failure mode of interest is crushing (i.e., telescoping) at the joint. A strength of materials model, based upon the pipe wall thickness, diameter and concrete strength, was used to establish the joint crushing force. Limited laboratory tests on small portions of pipe joints suggest that the strength of materials approach is appropriate.

PSEUDOSTATIC ANALYTICAL MODEL

The mathematical model used to determine buried pipeline response to pseudostatic ground strain was composed of three parts, the pipe segments, the surrounding soil, and the joints. In the analytical model the pipe segments were discretized into a finite number of beam and truss (i.e., rod) elements. Elastic properties of the equivalent composite section were used to characterize the axial stiffness of the segments. The surrounding soil was modeled by elasto-plastic elements (i.e., spring-sliders) based upon information in the technical literature. The axial load-deformation characteristics of the joint elements were based upon the laboratory tests described above. Special care was taken with the joints at each end of the 10 segment pipeline model. This was needed to ensure that all joints in a uniform pipeline model (identical pipe segment, soil and joint properties along the total 10 segment length) would have the same relative displacement when subject to uniform soil strain. Soil strain was simulated by displacing the bases of the soil spring-sliders.

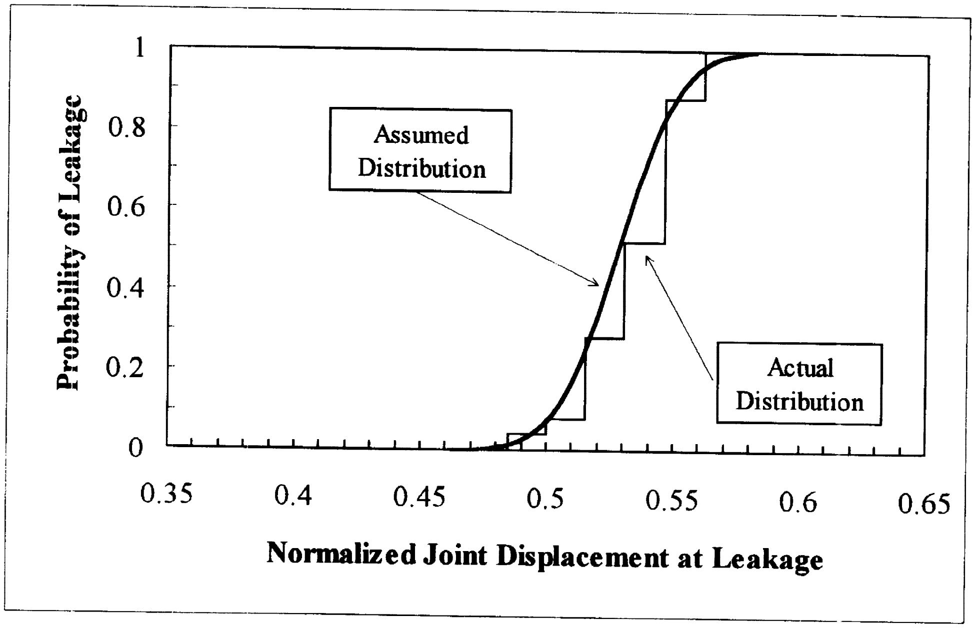

Fig. 1 Probability of Joint Leakage as a Function of Normalized Joint Displacement for 30 in. RCC Pipe ($0 \leq p \leq 30$ psi)

In order to approximate actual field conditions, the system properties for the mathematical model were established by Monte Carlo simulation. For example, the variation in stiffness and slippage displacement from joint to joint was based upon the experimental program results.

Figure 2 is a histogram of joint displacement for a 30 inch RCCP pipe (pipe segment length = 20 ft.) subject to a uniform tensile ground strain, ε_g, of 0.2% (0.002). The variation in joint displacement for this constant value of ground strain results from the random variation in system properties (pipe segment, joint and soil properties) along the model.

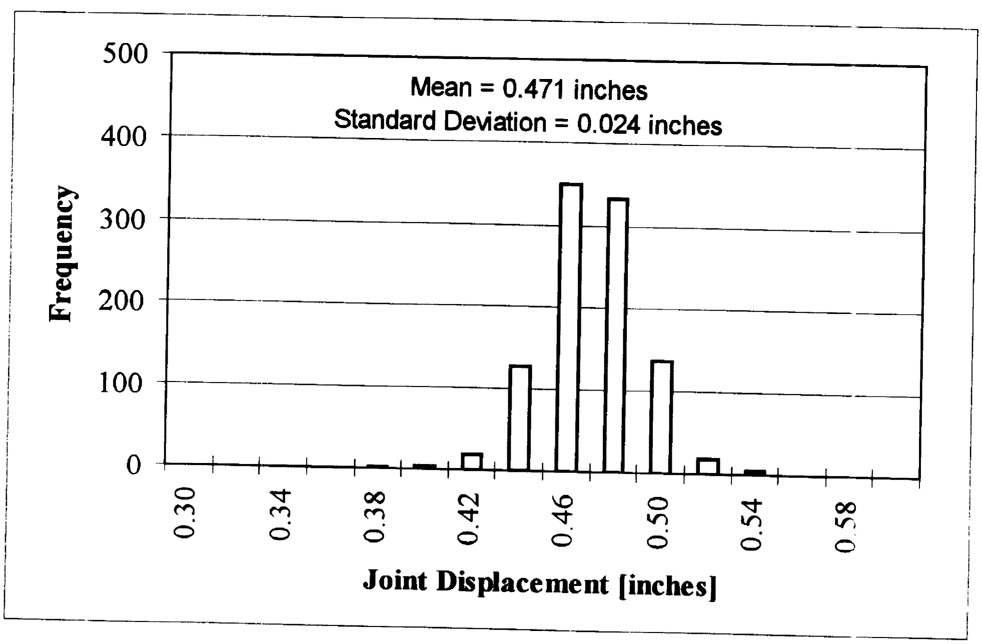

Fig. 2 Histogram of Joint Displacement for 30 in. RCCP (Tensile Ground Strain = 0.2%)

A statistical analysis of this information (e.g., Figure 2) yields joint displacement probability of exceedence for various levels of tensile ground strain. Figure 3 shows 0.1% and 50% probability of exceedence values for the 30 inch RCCP model. The median value (50% probability of exceedence) for the joint displacement is simply the ground strain times the pipe segment length. That is, for ε_g = 0.5%, the median joint displacement is 1.2 inches. However, there is a 1 in 1000 chance that the joint displacement will be 1.45 inches or larger. Information in this tail region is considered extremely important since, for even relatively heavy amounts of system damage, only 1 in 500 to 1 in 5000 joints are damaged.

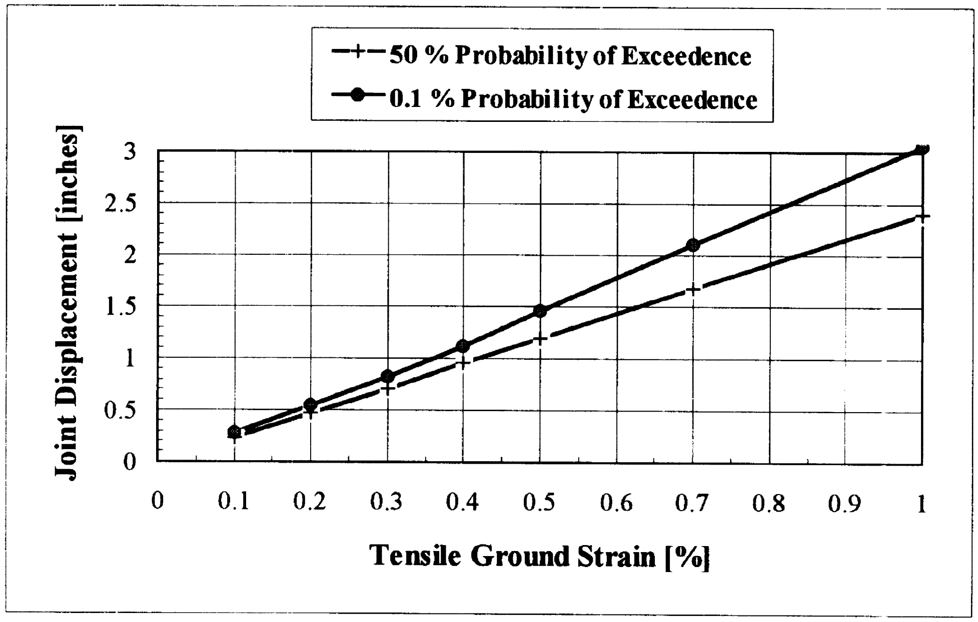

Fig. 3 Exceedence Probabilities for 30 in. RCCP Joint Displacement versus Tensile Ground Strain

Analytical fragility relations for tensile ground strain were developed by combining the probabilistic joint displacement data in Figure 3 with joint leakage relations in Figure 1. Figure 4 shows such an analytical tensile ground strain fragility relation for 30 inch RCCP, 48 inch prestressed lined cylinder pipe (LCP) and 60 inch prestressed embedded cylinder pipe (ECP). In this figure, the pipe segment length is assumed to be 20 ft. for all three pipe types. Less damage is expected for the larger diameter pipe due primarily to their greater joint depth (less leakage potential for same level of non-normalized joint displacement). Note that the joint properties for the larger diameter were extrapolated from the lab test results for the 30 inch pipe. For example, as described in more detail by Bouabid (1995), the joint slippage force is proportional to the rubber gasket's elastic modulus, and diameter, as well as the pipe diameter (specifically the outside diameter of the male spigot).

Figure 5 shows a similarly developed fragility relation for three pipes subject to axial compression. In this case the variation in joint crushing thresholds was based upon the cross-sectional area near the joint and an assumed normal distribution of concrete strength (mean strength of 5 ksi and 7% coefficient of variation).

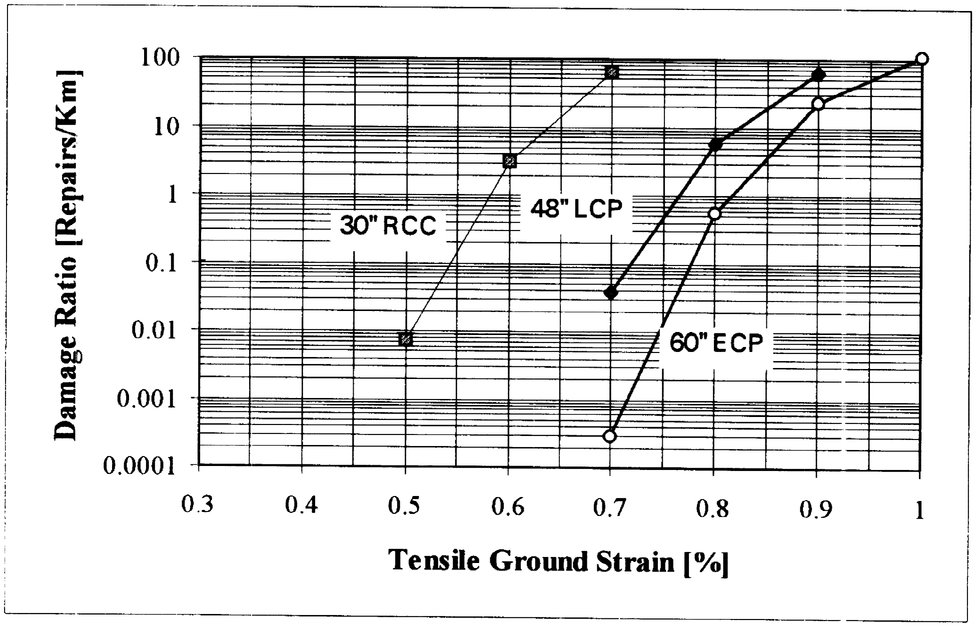

Fig. 4 Analytical Fragility Curves for Concrete Pipelines Subject to Tensile Ground Strain (pipe segment length = 20 feet)

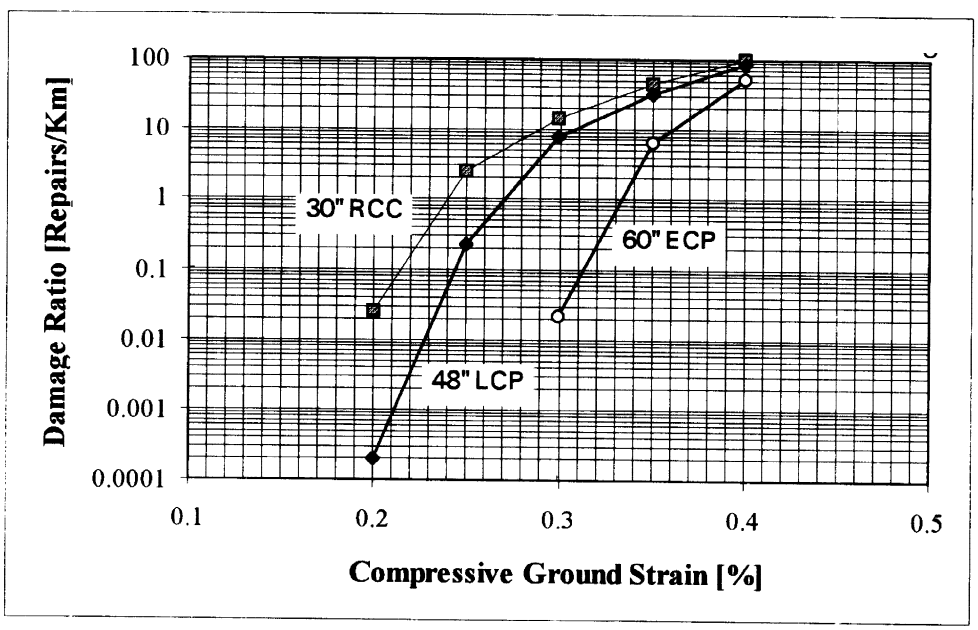

Fig. 5 Analytical Fragility Curves for Concrete Pipelines Subject to Compressive Ground Strain (pipe segment length = 20 feet)

MEXICO CITY BENCHMARK

A case study of damage to a concrete pipeline system in Mexico City, occasioned by the 1985 Michoacan Earthquake, is presented in order to benchmark the analytical results presented previously. The system considered consists of 32 inch, 36 inch and 48 inch diameter concrete pipelines which are part of the District Federal (D.F.) system. The pipelines in this system are predominantly prestressed concrete, 48 inches in diameter. The operating pressure at the time of the earthquake was 25 psi.

The epicenter for the September 19, 1985 Michoacan Earthquake was located about 400 Km southwest of Mexico City. Figure 6 shows the three accelerograph sites nearest the pipelines. Station CF recorded peak ground velocities, V_{max}, of about 38 cm/sec in the east-west direction and 25 cm/sec in the north-south direction. Station CO recorded peak ground velocities of about 42 cm/sec in the east-west direction and 35 cm/sec in the north-south direction. Station S1, on average, recorded somewhat larger peak ground velocities. Note that site S1 is located in a very soft clay zone, and as shown in Figure 6, most of the pipe and pipe damage is in a soft clay zone. For this reason, stations CF and CO are used herein to characterize the ground motion.

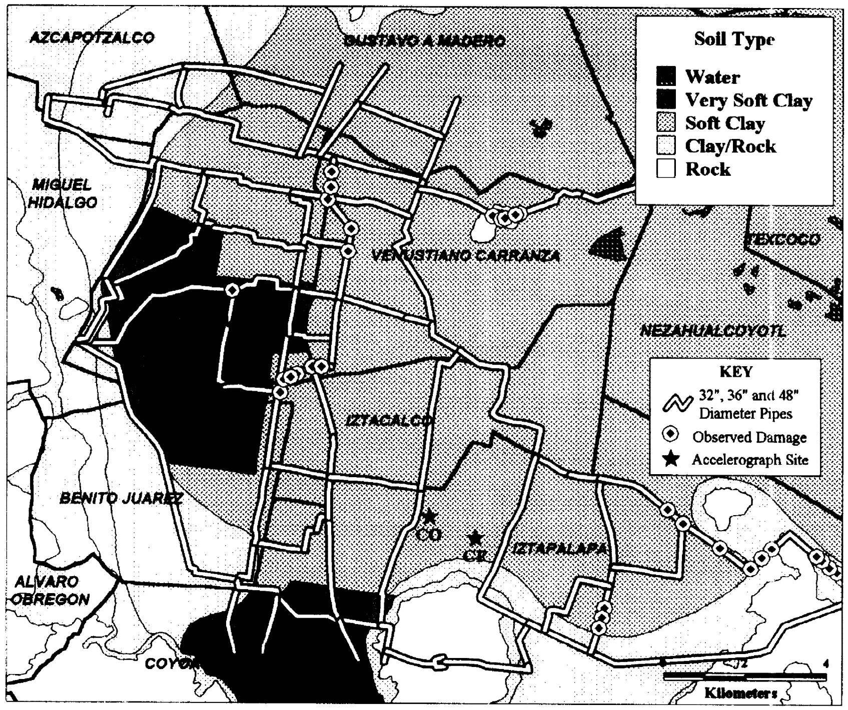

Fig. 6 Case Study Pipelines Overlaying the Soil Map

The system under consideration contains about 175 kilometers of pipelines. A total of 34 joint failures, most telescopic (i.e., compression), resulted from the earthquake and are also shown in Figure 6. Twenty-six of these occurred in 48 inch diameter pipes. The observed damage for the case study region corresponds to a damage ratio of about 0.20 repairs/km. Ayala and O'Rourke (1989) report an overall damage ratio of 0.34 repairs/km in the federal district (D.F.) for all pipe materials in the 20 to 48 inch diameter range.

The ground strain affecting these pipelines is estimated using a procedure developed by O'Rourke and Elhmadi (1988). This procedure is based on the assumption that the soil strain is due to Rayleigh wave propagation, which seems to be the case based on the recorded ground motion at the stations nearest to the pipelines considered here. The recorded ground velocities at these sites have a period of about 3.5 seconds (frequency of about 0.28 Hertz). The ground strain, ε_g, is estimated by

$$\varepsilon_g = V_{max}/C \tag{4}$$

where C is the phase velocity of the seismic wave. Based upon the R-wave dispersion curve for the area in question developed by Ayala and O'Rourke (1989), C ranges from 150 to 180 m/sec for a predominant frequency of 0.28 Hz. To account for the observed variability in ground motion, eight separate estimates of ground strain, based upon the four values of V_{max} and the two values of C, were considered. The ground strain so calculated ranged from 0.14 to 0.28%.

Since the pipeline system considered herein consists of 32 inch, 36 inch and 48 inch diameter concrete pipes, fragility curves for the 30 inch and 48 inch concrete pipelines presented in Figures 4 and 5 are used to predict expected damage ratios.

The analytical methods suggest that these pipelines are more likely to be damaged in a compressive telescopic fashion than by a tensile pull-out failure. Also, the fragility curves for the 48 inch pipes predict an average damage ration of 0.27 repairs per kilometer, while the fragility curves for 30 inch pipes predict an average damage ratio of 1.30 repairs per kilometer. Given that most of these pipelines are 48 inches in diameter and all of them are of sizes greater than 30 inches, the damage ratio of 0.27 repairs per kilometer seems to be the more appropriate number to consider. This damage ratio agrees reasonably well with the observed damage ratio of 0.2 repairs per kilometer.

CONCLUSIONS

Procedures used to develop analytical fragility relations for a class of concrete pipe are outlined. The model consists of a straight run of segmented pipe without bend, elbows, thrust blocks, T-junctions or junction boxes. The procedures utilize information on the load-deformation behavior of the pipe joints which typically would be based in some fashion on laboratory tests. The procedures also require information on pipe joint failure criterion. Herein the tensile pull-out failure criterion was based upon laboratory leakage tests, while the compressive crushing failure criterion was based primarily upon strength of materials concepts.

A benchmark case history from the 1985 Michoacan event suggests that the analytical procedures were able to correctly identify the most predominant failure mode for these concrete pipes, and provided a reasonable estimate of the expected damage rate in repairs per kilometer.

The authors feel that these analytical procedures can be used in seismic vulnerability evaluations to supplement empirical fragility relations, particularly for pipe types and/or diameters for which a large data base of actual earthquake experience is unavailable. The analytical procedures presented herein could also be used to identify and evaluate suggested improvements in joint details.

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