NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
State University of New York at Buffalo

LIQUEFACTION HAZARDS AND THEIR EFFECTS ON BURIED PIPELINES

by

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Technical Report NCEER-89-0007

February 1, 1989

This research was conducted at Cornell University and was partially supported by the National Science Foundation under Grant No. ECE 86-07591.

50272-101

REPORT DOCUMENTATION PAGE	1. REPORT NO. NCEER-89-0007	2.	3. Recipient's Accession No. PB89-218481/AS
4. Title and Subtitle: Liquefaction Hazards and Their Effects on Buried Pipelines			5. Report Date: February 1, 1989
			6.
7. Author(s): T. D. O'Rourke and P. A. Lane			8. Performing Organization Rept. No
9. Performing Organization Name and Address			10. Project/Task/Work Unit No.
			11. Contract(C) or Grant(G) No. (C) 87-5001 and 88-3014 (G) ECE 86-07591
12. Sponsoring Organization Name and Address: National Center for Earthquake Engineering Research, State University of New York at Buffalo, Red Jacket Quadrangle, Buffalo, New York 14261			13. Type of Report & Period Covered: Technical Report
			14.

15. Supplementary Notes

This research was conducted at Cornell University and was partially supported by the National Science Foundation under Grant No. ECE 86-07591

16. Abstract (Limit: 200 words)

The research described in this work involves the evaluation of liquefaction-induced ground movements and their effects on buried pipelines. The work is divided into three components: review of liquefaction phenomena and associated ground displacements; characterization of liquefaction-induced lateral spreading through observations and measurements of lateral spread deformations during past earthquakes; parametric study to evaluate buried pipeline response as a function of soil properties and geometric characteristics of lateral spreads. Case studies of four earthquakes were reviewed in which occurrences of lateral spreading have been reported. The results of the case studies indicate a close relationship between geologic and morphologic conditions and the occurrence and pattern of lateral spreading. The parametric study demonstrates the strong influence of geometric characteristics of the soil displacements on buried pipeline response. The delineation of zones of potentially large ground movement and the estimation of displacement patterns can be useful in the design of future pipeline systems, and the modification of existing ones, to limit earthquake damage.

17. Document Analysis a. Descriptors

b. Identifiers/Open-Ended Terms

Earthquake engineering	Soil properties	San Francisco, CA. earthquake, 1906
Liquefaction	Lateral spreading	San Fernando, CA. earthquake, 1971
Buried pipelines	Computer codes	Alaska earthquake, 1964
Lifelines	UNIPIP	Nihonkai-Chubu, Japan earthquake, 1983
Ground displacement	Soil pipeline interaction	

c. COSATI Field/Group

18. Availability Statement	19. Security Class (This Report)	21. No. of Pages
Release unlimited	Unclassified	197
	20. Security Class (This Page): Unclassified	22. Price: A09

(See ANSI-Z39.18) See Instructions on Reverse OPTIONAL FORM 272 (4-77) (Formerly NTIS-35) Department of Commerce

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NSF Master Contract Number ECE 86-07591

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PREFACE

The National Center for Earthquake Engineering Research (NCEER) is devoted to the expansion and dissemination of knowledge about earthquakes, the improvement of earthquake-resistant design, and the implementation of seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis is on structures and lifelines that are found in zones of moderate to high seismicity throughout the United States.

NCEER's research is being carried out in an integrated and coordinated manner following a structured program. The current research program comprises four main areas:

- Existing and New Structures
- Secondary and Protective Systems
- Lifeline Systems
- Disaster Research and Planning

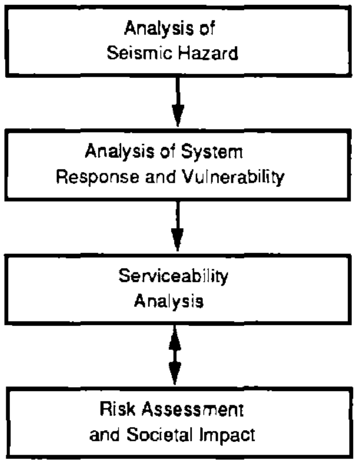
This technical report pertains to Program 3, Lifeline Systems, and more specifically to water delivery systems.

The safe and serviceable operation of lifeline systems such as gas, electricity, oil, water, communication and transportation networks, immediately after a severe earthquake, is of crucial importance to the welfare of the general public, and to the mitigation of seismic hazards upon society at large. The long-term goals of the lifeline study are to evaluate the seismic performance of lifeline systems in general, and to recommend measures for mitigating the societal risk arising from their failures.

From this point of view, Center researchers are concentrating on the study of specific existing lifeline systems, such as water delivery and crude oil transmission systems. The water delivery system study consists of two parts. The first studies the seismic performance of water delivery systems on the west coast, while the second addresses itself to the seismic performance of the water delivery system in Memphis, Tennessee. For both systems, post-earthquake fire fighting capabilities will be considered as a measure of seismic performance.

The components of the water delivery system study are shown in the accompanying figure.

Program Elements:



Tasks:

Wave Propagation, Fault Crossing
Liquefaction and Large Deformation
Above- and Under-ground Structure Interaction
Spatial Variability of Ground Motion

Soil-Structure Interaction, Pipe Response Analysis
Statistics of Repair/Damage
Post-Earthquake Data Gathering Procedure
Leakage Tests, Centrifuge Tests for Pipes

Post-Earthquake Firefighting Capability
System Reliability
Computer Code Development and Upgrading
Verification of Analytical Results

Mathematical Modeling
Socio-Economic Impact

The research described in this work consists of three components: 1) review of liquefaction phenomena and associated ground displacements; 2) characterization of liquefaction-induced lateral spreading through observations and measurements of lateral spread deformations during past earthquakes; and 3) parametric study to evaluate buried pipeline response as a function of soil properties and geometric characteristics of lateral spreads. Throughout, the result of case studies of four earthquakes (1906 San Francisco, 1964 Alaska, 1971 San Fernando, and 1983 Nihonkai-Chubu) is extensively utilized.

ABSTRACT

The research described in this work involves the evaluation of liquefaction-induced ground movements and their effects on buried pipelines. The work is divided into three components: 1) review of liquefaction phenomena and associated ground displacements, 2) characterization of liquefaction-induced lateral spreading through observations and measurements of lateral spread deformations during past earthquakes, and 3) parametric study to evaluate buried pipeline response as a function of soil properties and geometric characteristics of lateral spreads.

Case studies of four earthquakes were reviewed in which occurrences of lateral spreading have been reported. These include the 1906 San Francisco, 1964 Alaska, 1971 San Fernando, and 1983 Nihonkai-Chubu earthquakes. From these case studies, the geologic and morphologic features which control the displacement patterns of lateral spreads are identified. The damage caused by lateral spreading to lifeline systems, and pipeline networks in particular, are examined.

A detailed investigation was performed of ground movements associated with the 1906 San Francisco earthquake. The direction and magnitude of soil deformations are plotted on the city street system, using as references both historical accounts and photographs of damaged parts of the city after the earthquake. The pattern of soil movements are related to former topographical features of the region, the location and nature of filled areas, and the location of breaks in the pipeline system.

Displacement patterns typical of those observed during the 1906 San Francisco earthquake were used as a basis for a parametric study of buried pipeline response to lateral spreading. The soil/pipeline interaction was analyzed by means of a special computer code, UNIPIP, which is capable of evaluating the elasto-plastic behavior of both the soil and the pipeline material well into the post-yield range. The strains and deformation of a 610-mm-diameter continuous steel pipeline were evaluated as a function of the maximum displacement of a lateral spread, the width over which the

maximum displacement develops, and the shear strength and density of the surrounding soil.

The results of the case studies indicate a close relationship between geologic and morphologic conditions and the occurrence and pattern of lateral spreading. The case study of the 1906 earthquake shows that pipeline breaks caused by liquefaction-induced movements can be related explicitly to features of the backfilled topography. For example, 80 percent of all pipeline failures south of Market St. fall on or within the 12-m contour lines of the original Mission Creek and marsh areas. Additionally, the largest soil displacements occurred in areas where the contour lines of the buried topography converged, indicating a narrowing of the underlying valley or ravine which restricted the soil movements to relatively confined zones. This work clearly demonstrates that careful study and mapping of previous terrain and morphologic features can be used to identify locations of pipeline deformation, and even define the pattern and approximate magnitude of potential distortion.

The parametric study demonstrates the strong influence of geometric characteristics of the soil displacements on buried pipeline response. The maximum displacement of 1.5 m imposed on the pipeline was distributed over widths ranging from 10 to 50 m. In changing the width of the lateral spread from 10 to 50 m, holding all other parameters constant, the strains induced in the pipeline decreased by a factor of 6.5. In contrast, the tensile strains decreased by a factor of only 1.3 as a result of varying the soil density from 18.8 to 20.4 kN/m^3 and the soil friction angle from 35 to 45 degrees. Accordingly, pipelines should be sited to avoid potentially narrow zones of soil displacement, which are controlled by underlying morphologic and structural features. The delineation of zones of potentially large ground movement and the estimation of displacement patterns can, therefore, be useful in the design of future pipeline systems, and the modification of existing ones, to limit earthquake damage.

ACKNOWLEDGMENTS

The authors thank Robert Darragh, of Dames and Moore, and John Hovland, of the Pacific Gas and Electric Company, who have been very helpful in providing information on the subsurface conditions in downtown San Francisco. The authors also thank Maged Tawfik for his assistance and guidance with the parametric study.

Very special thanks are given to Ali Avcisoy, who prepared the majority of the figures in this work. Not only is his talent appreciated, but also his patience and encouragement. The authors also thank Kristin J. Stewart for her assistance in preparing the manuscript.

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SECTION 1
INTRODUCTION

1.1 BACKGROUND

In regions of high seismic activity, soil liquefaction has been identified as a major hazard to lifeline structures [Committee on Gas and Liquid Fuel Lifelines, 1984]. Liquefaction has been defined [Youd, 1973] as "the transformation of a cohesionless material from a solid state into a liquefied state as a consequence of increased pore pressure and reduced effective stress." Liquefaction of a soil deposit does not necessarily mean that ground failure occurs, but when liquefaction is combined with certain geologic conditions, it can lead to large permanent ground movements and soil failure. Conditions most conducive to liquefaction involve loose cohesionless granular deposits combined with a high water table. Such conditions are most likely to be found in recently deposited deltaic, river channel, flood plain, and eolian deposits, as well as loose or partially compacted fill.

Lateral spreading is one of the most common forms of ground deformation associated with liquefaction during earthquakes. This deformation mode involves lateral extension of intact surficial material caused by liquefaction of a subjacent layer. The displacements commonly range from one to several meters. The slides are generally accompanied by ground cracking. Lateral spreads normally develop on very gentle slopes, ranging between 0.3 and 3 degrees.

Lateral spreads pose special problems for pipelines in areas subject to earthquakes. Because movements are associated with gentle ground slopes, areas of potential lateral spreading commonly are thought of as stable. Because it remains in a more competent state, the surficial material being carried along with the underlying liquefied mass will impose a significant load on the pipeline.

For the siting and design of pipelines in seismic regions, it is important to identify areas susceptible to liquefaction. Analytical models may be helpful in evaluating the response of large or critical pipelines to liquefaction-induced ground movements. Ground deformation patterns, however, are extremely difficult to predict. Much of our knowledge of soil failures caused by liquefaction has been obtained from case studies. Quantitative information about soil displacement patterns, which is derived from case studies, is important for understanding the mechanics of large ground deformations, and for verifying analytical models of soil/structure interaction.

1.2 HISTORIC PERSPECTIVE

Lateral spreads have been reported in many of the earthquakes of this century under various terminology, such as "earth lurches" [Richter, 1958], "land spreading" [McCulloch and Bonilla, 1970], and "lateral spreading" [Oldham, 1899; Youd, 1974]. Varnes [1958] defined the lateral spread as one type of landslide. Youd [1978] adopted this name in an attempt to standardize terminology.

Lateral spreading can be severely disruptive to lifeline systems. Because pipeline networks traverse large areas, it is highly probable that sections of a pipeline system will cross faults and potentially liquefiable zones. Often, there are no alternatives to rerouting a pipeline around sensitive areas. Therefore, in the design of pipelines in seismic regions, consideration must be given to the potential for large soil displacements. Several studies have analyzed the response of buried pipelines to abrupt fault displacements [Newmark and Hall, 1975; Kennedy, et al., 1977; Wang and Yeh, 1985; O'Rourke and Trautmann, 1981] and distributed soil displacements [Tawfik and O'Rourke, 1986].

Several researchers have investigated lateral spread ground deformations during recent earthquakes, and have analyzed their effects on buried pipelines. These studies include the 1964 Alaska earthquake [Eckel, 1967; McCulloch and Bonilla, 1970; Kachadoorian, 1968], the 1971 San Fernando earthquake [Youd, 1971; Smith and Fallgren, 1975; O'Rourke and Tawfik, 1983], and the 1983 Nihonkai-Chubu earthquake [Hamada, et al., 1985; Kawashima, et al., 1985]. The information gained from these studies provides a basis for calibrating the predictive models used in the design of pipelines to evaluate their response to potential ground movements.

The 1906 San Francisco earthquake is of particular interest, since the permanent ground movements resulting from the earthquake were primarily responsible for disruption of the city water supply system. The loss of water to sections of downtown San Francisco contributed to the destruction of over 490 city blocks by the fire which followed the earthquake. Several investigators reported on the ground movements associated with the earthquake [Derleth, 1906; Hall, 1906; Himmlewright, 1906; Hyde, 1906; Jordan, 1907; Leonard, 1906; Schussler, 1906; Duryea, et al., 1907; Gilbert, et al., 1907; Lawson, et al., 1908; Hovland, 1980]. In particular, Youd and Hoose [1978] cataloged many of the reported ground movements. The response of the pipeline system also was examined by several researchers [Schussler, 1906; Derleth, 1906; Hyde, 1906; Manson, 1908].

1.3 OBJECTIVES

This work focuses on liquefaction-induced ground movements and their effects on buried pipelines. Special attention is directed to lateral spreading. The soil movements associated with lateral spreads are controlled by certain geologic conditions. By identifying these conditions and by describing the pattern of soil displacements, rational decisions can be made about pipeline design and siting in seismic regions.

The primary objectives of this study are two-fold:

1. To describe the lateral spreading phenomenon both qualitatively and quantitatively. Case studies of several earthquakes by other researchers are reviewed for information on magnitude and patterns of soil displacements, as well as the response of pipeline systems to these displacements. A case study of the 1906 San Francisco earthquake also is performed in this work. This involves cataloging and mapping ground displacements, as described by literature or estimated from photographs, relative to the geologic and morphologic conditions of the region. The extent of pipeline damage also is related to the morphologic conditions of the area to further define boundaries of permanent ground deformation.

2. To evaluate how certain characteristics of lateral spreading affect the response of a buried pipeline. A parameter study is conducted

using the computer code UNIPIP developed at Cornell [Tawfik and O'Rourke, 1986], which predicts pipeline response to permanent soil displacements. The pipeline response is examined as a function of the magnitude of horizontal displacements associated with lateral spreading, the width of the slide, and the unit weight and strength of the displaced soil mass.

1.4 SCOPE

This work is composed of five sections, of which the first presents background information and introductory comments. The second section discusses liquefaction and associated ground deformations. Several case studies are reviewed to delineate general deformation patterns caused by lateral spreading and their effects on buried pipelines. The third section presents a case study of the liquefaction-induced ground movements during the San Francisco earthquake of April 18, 1906. The geologic conditions in the zones of high intensity of earthquake damage are examined in relation to the magnitude and pattern of permanent ground displacements. Damage to the pipeline network resulting from the 1906 earthquake is used to improve previous work identifying zones of potential ground deformation in San Francisco. Information from the case studies in Sections 2 and 3 is used as a basis for the parameter study presented in Section 4. The response of buried pipelines to horizontal soil displacements is investigated by varying certain characteristics of the lateral spread patterns. The fifth section provides conclusions and recommendations for improved practice.

SECTION 2

LIQUEFACTION-INDUCED GROUND DEFORMATION

2.1 INTRODUCTION

Liquefaction leads to diverse patterns of soil movement. Excess pore pressures generated during liquefaction can result in the expulsion of water and soil to form relatively small-scale features, such as sand boils, as well as large-scale features, such as sinkholes and deep surface depressions. On level ground, loss of shear strength can cause bearing capacity failure. In slopes, reduction in shear strength can result in flow failures and lateral spreads. Deep-seated landslides also may be caused by the liquefaction of one or more sand layers within a stratified soil deposit. Buoyancy of buried structures within liquefied soil may cause serious structural deformation.

Given the protean nature of ground movements arising from liquefaction, it is useful to review the principal characteristics of this phenomenon and its influence on ground deformation patterns and pipeline performance. In this section, factors contributing to soil liquefaction are reviewed. A general description of ground deformation patterns is presented, followed by a review of case histories in which lateral spreads caused by liquefaction were responsible for damage to buried pipeline facilities.

2.2 LIQUEFACTION

During an earthquake, soil particles are subjected to alternating cycles of shear stress of randomly varying magnitude [Seed, 1968]. In a cohesionless deposit, the soil structure tends to decrease in volume in response to the application of cyclic strains. No decrease in volume can occur if the sand deposit is undrained, and thus there is a resulting increase in pore water pressure and a decrease in effective stress [Committee on Earthquake Engineering, 1985]. The amount to which the soil structure must rebound such that the volume remains constant determines the magnitude of the build-up of pore water pressure. Figure 2-1 shows the mechanism involved [Seed, 1979; Committee on Earthquake Engineering, 1985]. The volume change, Δe, from A to B, which would result from drained cyclic loading, cannot occur because of the undrained conditions. The pore pressure, therefore, increases an

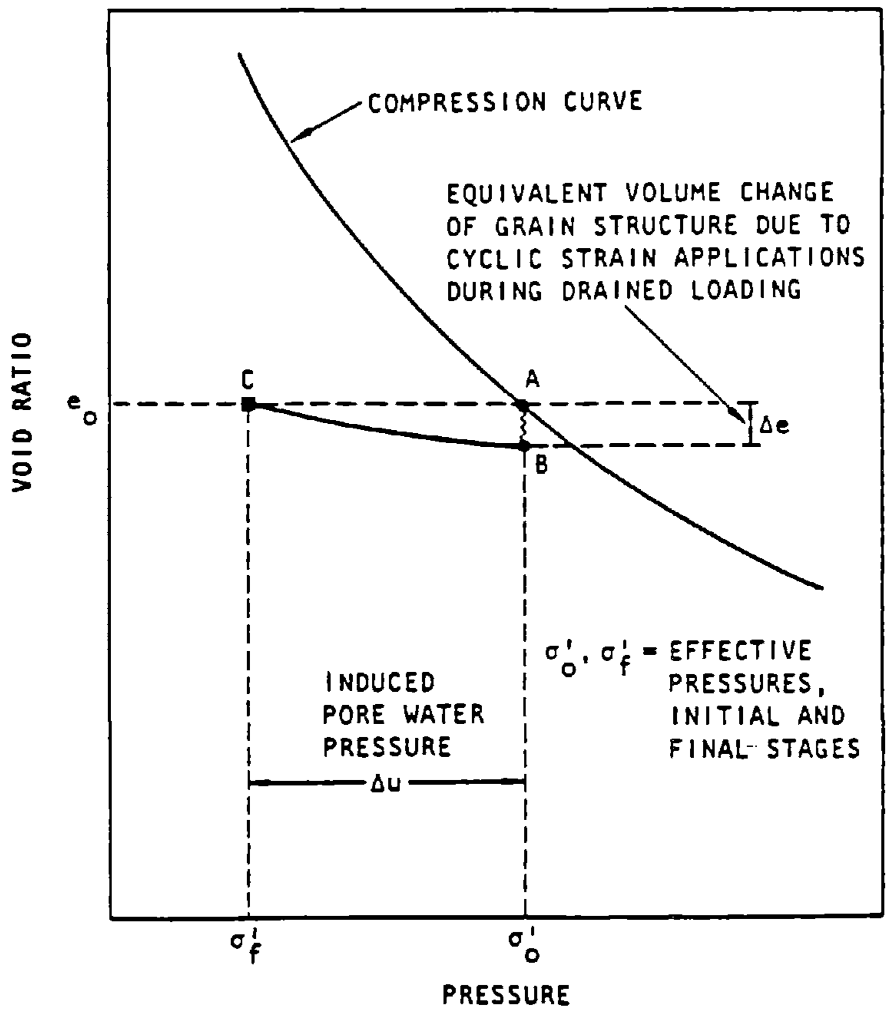


FIGURE 2-1. Schematic Illustration of Pore Pressure Generation During Cyclic Loading [after Seed, 1979]

amount equivalent to the decrease in effective stress associated with the rebound from B to C. If the earthquake is of sufficiently long duration, the pore water pressure approaches the overburden stress.

In a loose sand, the pore water pressure build-up can be rapid. Liquefaction is said to develop if unlimited deformation occurs without the sand mobilizing significant resistance [Seed, 1979]. Dense sand, however, will tend to dilate, and the pore water pressure will decrease if the sand is undrained. The soil will eventually develop enough resistance to withstand the applied stress. Some deformation is required to develop resistance, and this amount of deformation may increase with the continuation of cyclic loading. Castro [1975] and Seed [1979] have referred to this type of behavior as "cyclic mobility."

At the end of cyclic loading, there still will exist a residual pore pressure which will lead to an upward flow of water. Upward flow of water from a lower layer may cause liquefaction of an overlying layer, sand boils, and water seepage at the ground surface.

Liquefaction can be caused by both static (monotonic) and dynamic (cyclic) loading. In a study of static liquefaction, Kramer and Seed [1988] present stress-strain and pore pressure data for four isotropically consolidated tests on triaxial samples of Sacramento River Fine Sand, as illustrated in Figure 2-2. For the samples at relative densities of 32 and 37%, both the excess pore water pressure and rate of straining increased rapidly after the peak deviator stress was reached. These samples liquefied. After reaching maximum deviatoric stress, the sample at 44% relative density deformed rapidly to about 10% axial strain, and then began to dilate. Because of the potential damage which may ensue from strains of this magnitude, this sample also was considered to have liquefied.

The sample tested at a relative density of 47% showed dilative behavior with decreasing pore water pressure and increasing deviatoric stress at high strains. This sample did not liquefy. Sands exhibiting this type of dilative behavior will continue to deform when subjected to cyclic loads, but the accumulation of deformation will be significantly less than that associated with liquefaction. The deformational response under cyclic loading of relatively dense, or dilative, sands is referred to as cyclic mobility [Castro, 1975].

Youd [1973] defined liquefaction as the transformation of a granular material from a solid state to a liquefied state as a consequence of increased pore water pressure. This liquefied state includes both the full liquefaction represented in Figure 2-2 by the samples at 32 and 37% relative density, and the limited liquefaction represented by the sample at 44% relative density. Because Youd's definition is simple and consistent with the

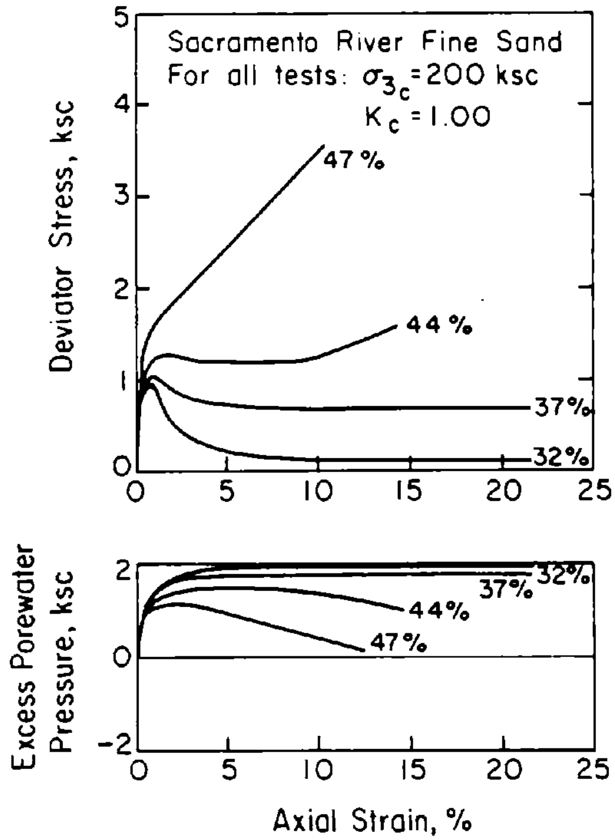


FIGURE 2-2. Effect of Relative Density on Stress-Strain and Pore Water Pressure Response of Sacramento River Sand [after Kramer and Seed, 1988] (© ASCE. Used by Permission.)

definitions offered by many others [Youd, 1973], it is adopted in this report.

2.3 CONSEQUENCES OF LIQUEFACTION

Site exploration and laboratory test procedures for evaluating liquefaction potential have been the subjects of intense study for many years, and procedures have been developed for establishing the susceptibility of a given deposit to liquefaction. This, in itself, may not be of substantial value if the effects of the soil liquefaction cannot be forecast and quantified in, at least, an approximate way. The types and magnitudes of ground deformation resulting from liquefaction are of critical importance, especially for buried structures like pipelines, which are constrained to move as the ground moves, or resist such displacement by means of flexural and axial stiffness.

There is general agreement that, at the present time, our procedures for predicting and characterizing ground deformations induced by liquefaction are imprecise, and lacking the same definition and reliability which pertain to procedures for identifying which soils are potentially liquefiable [Committee on Earthquake Engineering, 1985; Seed, 1987]. Seed [1987] has drawn attention to the fact that, once liquefaction occurs, the current ability of the geotechnical engineering profession to predict its consequences deteriorates significantly.

An understanding of soil behavior under cyclic loads can provide a framework for distinguishing general types of ground response and the associated mechanisms of deformation. Castro [1987] has suggested a classification scheme, based primarily on the presence of driving shear stress, for explaining the different kinds of ground performance which have been observed after an earthquake. This general classification scheme is summarized in Table 2-I.

On level ground, where there is no permanent driving shear stress, soil deformations may occur because of volume changes and associated settlement. If excess pore pressures are generated by cyclic loading in potentially liquefiable soils, then the pore pressures will tend to vent themselves by water migration to the ground surface. Depending on the near-surface soil layering and permeability, dissipation of excess pore pressure may occur in the form of sand boils and fissure ejections of fine sands. Although sand boils may be highly visible markers of excess pore water pressure, they do not contribute so much to damage as they participate in a process of volume change. The settlements associated with this volume change may be highly irregular, and therefore impart significant deformations to buried structures.

In the presence of driving shear stress, which requires a slope, bearing load, or buoyancy force, the consequences of liquefaction change dramatically. With reference to Table 2-I, deformation in the field may occur catastrophically as a flow or bearing capacity failure, when the in-situ driving shear exceeds the residual or steady state shear capacity of the soil. Alternatively, lateral spreading and slumping of slopes will occur when stresses induced by the earthquake temporarily exceed the soil shear

TABLE 2-I. Summary of Stress Conditions, Soil Behavior, and Consequences of Liquefaction [adapted from Castro, 1987]

In-Situ Stress Condition[1]	Soil Behavior	Typical Field Observation
No driving shear stresses	Volume decrease Pore pressure increases	Ground settlement Sand boils and ejections from surface fissures
Presence of driving shear stresses	Loss of stability	Flow failures Sinking of heavy buildings Floating of light structures
	Limited shear distortion (soil mass remains stable)	Slumping of slopes Settlement of buildings Lateral spreading

[1]Refers to in-situ shear stresses before cyclic loading

strength. Under these conditions, the stresses after cyclic loading will return to equilibrium, in which driving shears are less than the soil strength.

The distinction between flow failures and lateral spreading in Castro's classification [1987] depends on a limiting condition of undrained soil strength, referred to as the steady state of deformation. As defined by Poulos [1981], the steady state is that "in which the mass is continuously deforming at constant volume, constant normal effective stress, constant shear stress, and constant velocity." A full discussion of steady state concepts is beyond the scope of this report, and the reader is referred elsewhere for more elaboration on these principles [Poulos, 1981; Poulos, et al., 1985; Castro, 1987]. It should be recognized that the steady state is a unique limiting condition of the soil, and is a function only of the initial void ratio, and not of the initial state of stress, undrained stress path, nor initial soil structure.

Figure 2-3 shows the stress-strain behavior under cyclic load in comparison with that under monotonic loading. In Case 1, the undrained steady state shear strength, S_{us}, is lower than the initial driving shear stress, τ_d. If

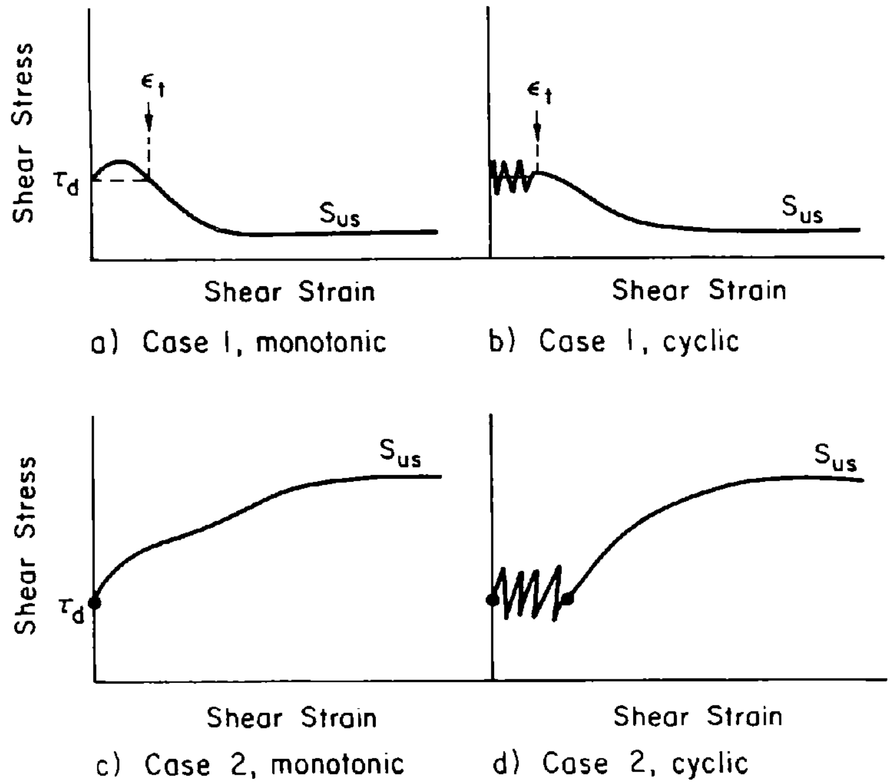


FIGURE 2-3. Monotonic and Cyclic Undrained Loading of Contractive (Case 1) and Dilative (Case 2) Sand [after Castro, 1987] (© Elsevier Science Publishing Co. Used by Permission.)

cyclic loading leads to a condition in which the strain, ϵ_t, is exceeded, then flow failure will result. Castro [1975; 1987] explicitly defines this case as liquefaction. As such, it is covered by, but not entirely consistent with, the definition of liquefaction adopted in this report. In Case 2, cyclic loading leads to limited deformation, without changing the stable configuration of the soil mass. The stress-strain relationship for the soil under this condition need not be strain hardening, as shown, but may include strain softening similar in shape to that in Figure 2-3a, as long as the final undrained strength exceeds the driving stress after cyclic loading. Ground deformation can accumulate only during cyclic loading, when the combined static driving and cyclic stresses exceed the shear strength of the soil. Continuing deformations after shaking, however, may still occur because of water migration and associated volume loss associated with water and soil venting through sand boils and fissures, as well as the gradual dissipation of water pressures.

It should be recognized that the mechanisms proposed for flow failure are not limited to the condition described by Castro [1987]. The Committee on Earthquake Engineering [1985] has distinguished at least two other mechanisms: 1) loss of static shear resistance within a portion (usually the upper portion) of a mass of cohesionless soil, owing to a redistribution of soil density, and 2) loss of static shear resistance within ground adjacent to cohesionless soil in which high pore water pressures develop, owing to reduction in effective stress and possibly cracking, as pore pressures push outward from the zone of high pore pressure. Seed [1987], in particular, has emphasized the importance of density redistribution. Shaking table tests [Liu and Qiao, 1984] have shown that water may accumulate under an impervious zone such that the void ratio in sand beneath the zone may actually increase as a consequence of the earthquake. This increase in void ratio would tend to decrease the steady state strength, and thus promote a more unstable condition than projected from the pre-earthquake void ratio.

2.4 GEOLOGIC CONTROLS ON LIQUEFACTION

The susceptibility of soils to liquefaction depends on composition and available water. Accordingly, the depth of the groundwater table, grain size characteristics, density, and age of the deposit play important roles in determining the degree of vulnerability to soil liquefaction. Because these features are strongly correlated with landforms, geomorphology plays an important role in assessing the potential for and consequences of liquefaction. The geomorphology helps not only to identify the most likely physical characteristics of a deposit, but also provides clues regarding groundwater depth, surface slope, as well as inclination, thickness, and areal extent of subsurface strata. These latter aspects establish the in-situ driving shears and geometric boundaries of the potentially liquefiable soils, and thus influence the magnitude and distribution of ground deformation.

One of the most comprehensive field surveys of earthquake-induced ground deformations was performed by McCulloch and Bonilla [1970] along the Alaska Railroad after the 1964 earthquake. The landforms which showed the largest lateral and vertical deformations were those associated with active flood plains and alluvial fans and deltas. The most extreme damage was caused on active flood plains adjacent to modern stream channels. Large ground deformations on fans and deltas were most prominent in the toe areas, where groundwater tables were closest to the surface. The data collected by McCulloch and Bonilla [1970] on the tensile and compressive displacements at bridges crossing stream and river channels provides one of the most comprehensive data bases on the magnitudes of lateral spreading.

Youd and Perkins [1978] summarized various types of soil deposits and assigned a qualitative rating of liquefaction susceptibility to each. As shown in Table 2-II, liquefaction susceptibility is summarized for continental, coastal, and artificial deposits. As is evident from the table, age strongly influences the potential for liquefaction, with recent (less than 500 years) and Holocene deposits exhibiting the greatest vulnerability.

Studies by Japanese researchers [Midorikawa and Wakamatsu, 1988], based on an evaluation of previous earthquake effects, have shown that the occurrence of soil liquefaciton is correlated better with peak ground velocity than peak ground acceleration. Various geomorphological conditions were investigated for 250 sites during 19 Japanese earthquakes, and the occurrence and absence of liquefaction was correlated with the peak ground velocity appropriate for the sites [Kotodo, et al., 1988]. Table 2-III lists various geomorphological units and threshold velocities associated with the occurrence of liquefaction. In this context, the velocities serve as a severity index of the liquefaction potential.

On the basis of historical evidence of liquefaction-induced ground deformation, Youd and Perkins [1987] have developed a technique for compiling liquefaction hazard maps by mapping a parameter called the liquefaction severity index (LSI). The LSI represents the general maximum differential ground movement (in inches) associated with lateral spreading that can be anticipated in active flood plains, deltas, or other areas of gently sloping Holocene fluvial deposits. Table 2-IV summarizes the abundance and characteristics of liquefaction effects as a function of LSI for areas with widespread liquefiable deposits. By means of statistical correlations, Youd and Perkins [1987] developed an equation relating LSI, earthquake magnitude, and distance from seismic energy source for data pertaining to western U.S.

TABLE 2-II. Estimated Susceptibility of Sedimentary Deposits to Liquefaction During Strong Ground Shaking [after Youd and Perkins, 1978]

Type of deposit (1)	General distribution of cohesionless sediments in deposits (2)	Likelihood that Cohesionless Sediments, When Saturated, Would Be Susceptible to Liquefaction (by Age of Deposit) <500 yr (3)	Holocene (4)	Pleistocene (5)	Pre-pleistocene (6)
(a) Continental Deposits					
River channel	Locally variable	Very high	High	Low	Very low
Flood plain	Locally variable	High	Moderate	Low	Very low
Alluvial fan and plain	Widespread	Moderate	Low	Low	Very low
Marine terraces and plains	Widespread	—	Low	Very low	Very low
Delta and fan-delta	Widespread	High	Moderate	Low	Very low
Lacustrine and playa	Variable	High	Moderate	Low	Very low
Colluvium	Variable	High	Moderate	Low	Very low
Talus	Widespread	Low	Low	Very low	Very low
Dunes	Widespread	High	Moderate	Low	Very low
Loess	Variable	High	High	High	Unknown
Glacial till	Variable	Low	Low	Very low	Very low
Tuff	Rare	Low	Low	Very low	Very low
Tephra	Widespread	High	High	?	?
Residual soils	Rare	Low	Low	Very low	Very low
Sebka	Locally variable	High	Moderate	Low	Very low
(b) Coastal Zone					
Delta	Widespread	Very high	High	Low	Very low
Esturine	Locally variable	High	Moderate	Low	Very low
Beach					
High wave energy	Widespread	Moderate	Low	Very low	Very low
Low wave energy	Widespread	High	Moderate	Low	Very low
Lagoonal	Locally variable	High	Moderate	Low	Very low
Fore shore	Locally variable	High	Moderate	Low	Very low
(c) Artificial					
Uncompacted fill	Variable	Very high	—	—	—
Compacted fill	Variable	Low	—	—	—

earthquakes. The equation, a model of seismic sources, and a published seismic risk algorithm were used to compile probabilistic LSI maps for southern California.

2.5 RECURRENCE OF LIQUEFACTION

Liquefaction of a soil layer does not necessarily render the deposit more

TABLE 2-III. Summary of Peak Ground Velocities Associated with Onset of Soil Liquefaction for Various Geomorphological Units [after Kotoda, et al., 1988]

Geomorphological Unit	Critical Peak Ground Velocity kine (cm/s)
Reclaimed land Drained land Natural levee[1] River channel Sand dune[2] Lowland between sand dunes	15
Back marsh Valley plain Delta	25
Sand bar Alluvial fan	35

[1] Including outer margin of levee
[2] Including outer margin of dune

stable. Although densification will occur in the lower layer from which the water is expelled, the upward flow of water may leave the overlying layer in a looser state, and therefore less stable with regard to liquefaction. Lawson, et al. [1908] report evidence of liquefaction in the same area of San Francisco, near the foot of Market Street, from both the 1868 and the 1906 earthquakes. Similarly, lateral spreading during the 1964 Alaska earthquake was shown through field investigations to coincide with lateral spread locations of past earthquakes [Hansen, 1965].

Both U.S. and Japanese observations indicate that liquefaction can recur at sites known to have shown evidence of liquefaction during previous earthquakes [Kuribayaski and Tatsuoka, 1975; Youd, 1984; Yasuda and Tohno, 1988]. Most of the evidence for liquefaction is based on observations of sand boils, although Youd [1984] cites the recurrence of spreading and slumping along the banks of Coyote and Alameda Creeks during both the 1868 Hayward and 1906 San Francisco earthquakes. In several cases, reliquefaction occurred in places where accelerations produced by the latter earthquake were less than those caused by the former earthquake. Youd describes the

TABLE 2-IV. Summary of Abundance and General Character of Liquefaction Effects as a Function of LSI for Areas with Widespread Liquefiable Deposits [after Youd and Perkins, 1987] (© ASCE. Used by Permission.)

LSI (1)	Abundance and general character of liquefaction effects (2)
5	Very sparsely distributed minor ground effects include sand boils with sand aprons up to 0.5 m (1.5 ft) in diameter, minor ground fissures with openings up to 0.1 m wide, ground settlements of up to 25 mm (1 in.). Effects lie primarily in areas of recent deposition and shallow ground water table such as exposed stream beds, active flood plains, mud flats, shore lines, etc.
10	Sparsely distributed ground effects include sand boils with aprons up to 1 m (3 ft) in diameter, ground fissures with openings up to 0.3 m (1 ft) wide, ground settlements of a few inches over loose deposits such as trenches or channels filled with loose sand. Slumps with up to a few tenths of a meter displacement along steep banks. Effects lie primarily in areas of recent deposition with a ground water table less than 3 m (10 ft) deep.
30	Generally sparse but locally abundant ground effects include sand boils with aprons up to 2 m (6 ft) diameter, ground fissures up to several tenths of a meter wide, some fences and roadways noticeably offset, sporadic ground settlements of as much 0.3 m (1 ft), slumps with 0.3 m (1 ft) of displacements common along steep stream banks. Larger effects lie primarily in areas of recent deposition with a ground water table less than 3 m (10 ft) deep.
50	Abundant effects include sand boils with aprons up to 3 m (10 ft) in diameter that commonly coalesce into bands along fissures, fissures with widths up to 1.5 m (4.5 ft), fissures generally parallel or curve toward streams or depressions and commonly break in multiple strands, fences and roadways are offset or pulled apart as much as 1.5 m (4.5) in some places, ground settlements of more than 1 ft (0.3 m) occur locally, slumps with a meter of displacement are common in steep stream banks.
70	Abundant effects include many large sand boils [some with aprons exceeding 6 m (20 ft) in diameter that commonly coalesce along fissures], long fissures parallel to rivers or shorelines usually in multiple strands with many openings as wide as 2 m (6 ft), many large slumps along streams and other steep banks, some intact masses of ground between fissures displaced 1–2 m down gentle slopes, frequent ground settlements of more than 0.3 m (1 ft).
90	Very abundant ground effects include numerous sand boils with large aprons, 30% or more of some areas covered with freshly deposited sand, many long fissures with multiple strands parallel streams and shore lines with openings as wide as two or more meters, some intact masses of ground between fissures are horizontally displaced a couple of meters down gentle slopes, large slumps are common in stream and other steep banks, ground settlements of more than 0.3 m (1 ft) are common.

reoccurrence of 150-mm-diameter sand boils during the 1981 Westmoreland earthquake (M_S = 6.0) at the same location of 2-m-diameter sand boils during the 1979 Imperial Valley earthquake (M_S = 6.6). Similarly, Yasuda and Tohno [1988] report that relatively small sand volcanoes formed during the main aftershock of the 1983 Nihonkai-Chubu earthquake (M_S = 7.1) at the locations of 7 to 8-m-diameter sand boils which occurred during the main shock (M_S = 7.7).

Youd [1984] attributes the recurrence of liquefaction to the creation of loose zones near the top of liquefiable deposits as consolidation occurs at lower elevations, and to loosening as a result of shear-induced dilation. Yasuda and Tohno [1988] suggest that the upward migration of water under high pressure loosens soils near the upper levels of a liquefied deposit. Youd describes a layer of silty sand at a site in Brawley, CA where upper drainage was impeded by surficial clay. Cone penetration tests disclosed a very loose condition in the material from which hundreds of sand boils and soil slumping developed during the 1979 Imperial Valley earthquake.

2.6 LARGE GROUND DEFORMATION ASSOCIATED WITH LIQUEFACTION

Several types of large ground deformations may be triggered by soil liquefaction. In this report, the following conditions of deformation are identified: flow failures, lateral spreads, loss of bearing capacity, subsidence, buoyancy, and ground oscillation. On the basis of investigations performed by Seed of ground deformations during the 1964 Alaska earthquake [Seed, 1968; 1973], it also is possible to add a category pertaining to landslides caused by liquefaction of sand layers and seams. Because landslides of the type which occurred in Anchorage during the 1964 earthquake share many characteristics with lateral spreads, they are discussed in this report under the heading of lateral spreads. It should be recognized, however, that the Anchorage landslides represent a special case, and that recent investigations have shown that sensitive clays played a key, if not predominant, role in these ground deformations.

2.6.1 Flow Failure

Flow failures are perhaps the most catastrophic of the ground deformations associated with liquefaction. They generally occur in saturated loose sands with slopes ranging between 10 and 20 degrees [Youd, 1978]. Several cases, however, also have occurred in loess deposits. Keefer [1984] describes this failure mode as "rapid soil flows," in which soil grains flow in a fluid-like fashion.

During flow failures, large amounts of material may flow many tens of meters at relatively high speeds of tens of km/h. Only when the driving forces are reduced to values less than the viscous shear resistance of the flowing material, such as by a flattening of the slope, will the movement cease. The soil may move as a completely liquefied mass, or as intact blocks floating in the fluid material. The material is generally deposited in a mass, rather than fanning onto relatively level ground. The final slope grade usually is less than 7% [Youd, 1975].

Several cases of flow failures occurred along the coastline during the 1964 Alaskan earthquake at Seward, Whittier, and Valdez. These flows, which occurred as submarine landslides, undermined large sections of the port facilities in these towns. In Valdez, the flow slide involved an estimated 75 million cubic meters of deltaic sediments. Ground fractures broke water and sewage lines throughout the town [Eckel, 1967]. Additional damage and loss of lives were sustained from the sea waves generated by these slides [Youd, 1978].

Another example of a flow failure occurred during the 1920 Kansu earthquake in China [Close and McCormick, 1922] in a collapsible partially saturated loess deposit. It has been postulated that liquefaction was generated in this case by the buildup of pore air pressure, not pore water [Seed, 1968]. Some of the flow failures were up to 1-1/2 km in length and width, and flowed for several km. The slides originated in terraced hills and flowed into an adjacent valley. In some cases, the flows carried with them intact soil blocks. The earthquake caused a loss of nearly 200,000 lives, and completely destroyed 10 large cities and hundreds of small villages, with much of this damage as a result of the landslides.

Flow failures also occurred in the 1906 and 1957 San Francisco erthquakes. For example, a slide occurred during the 1906 earthquake at Mount Olivet

Cemetery on the south side of San Bruno Mountains [Youd and Hoose, 1978]. At the time of the earthquake, water and sand gushed at a point near the upper end of the cemetery. The fluid material mixed with the loam of the slope and flowed several hundred meters downslope. The slide involved over 100,000 cubic meters of material, and lasted roughly 3 minutes [Lawson, et al., 1908]. Flow slides also took place in the 1957 San Francisco earthquake, a dozen of which occurred around Lake Merced [Bonilla, 1959]. Most of these slides involved artificial fills of loose, uniform, saturated sands. The material flowed and spread out over the bottom of the lake.

Flow failures may impart viscous drag forces on buried pipelines, which are comparable to or larger than those associated with the reaction pressures of intact soil caused by lateral spreads. Assuming a viscosity of 10^{10} N-S/m^2, suggested by Vyalov [1986] for rapid landslides, and a velocity of one to several m/sec, the forces conveyed to a 300-mm-diameter pipeline may range from 10 to 100 kN/m. Given the catastrophic nature of the deformation and the fact that intact soil and objects may be carried in the debris, rapid flow failures are among the worst geotechnical hazards for buried structures.

2.6.2 Lateral Spreads

Lateral spreads are one of the more common forms of permanent ground movement associated with liquefaction during earthquakes. This failure mode involves lateral extension and fracturing of intact surficial material caused by liquefaction of a subjacent layer. Youd [1978] has reported displacements associated with lateral spreads ranging from one meter to tens of meters. Displacements commonly range, however, from one to two meters. The spreads often are accompanied by ground cracking. Lateral spreads develop normally on very gentle slopes (between 0.3 and 3 degrees) [Youd, 1978].

Figure 2-4 shows an idealized cross-section through a lateral spread ground failure [Committee on Gas and Liquid Fuel Lifelines, 1984]. Many of the deformation patterns which occur in lateral spreads resemble those associated with faulting. Differential displacements typically are concentrated along the margins of the spread. The head of the lateral spread generally resembles a normal fault, where abrupt offsets of 1 to 2 m have been

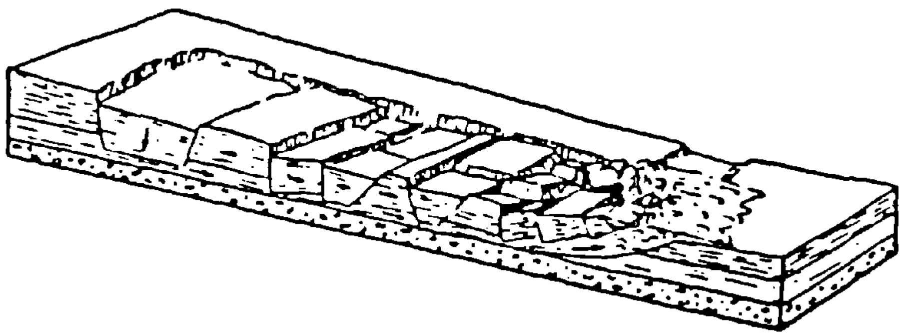


FIGURE 2-4. Lateral Spread Ground Failure [after Varnes, 1958; Committee on Gas and Liquid Fuel Lifelines, 1984]

observed. Displacements along the side and toe of the slide replicate strike-slip faulting and reverse faulting, respectively. Large compressive strains generally develop at the base of the slide [Committee on Gas and Liquid Fuel Lifelines, 1984]. The horizontal displacements are distributed over the width of the slide, with the maximum displacement generally occurring towards the center of slide.

On the basis of observations in Anchorage after the 1964 earthquake, Seed [1968; 1973] proposed additional mechanisms for large lateral slides associated with the liquefaction of sand layers and sand seams underlying an otherwise stable soil mass. Subsequent investigations of soil deposits at the Fourth and L Street sites of the 1964 landslides indicate that failure of sensitive layers of the Bootlegger Cove clay was the most likely cause of landslide activity [e.g., Idriss, 1985; Moriwaki, et al., 1989] at these locations.

The 1964 Anchorage landslides represent a mode of ground deformation, observed after other earthquakes, in which large lateral movements of several meters developed concurrently with the formation of grabens, at distances of several to scores of meters behind the spreading front. The morphology of these landslides differs from that conventionally associated with lateral spreads, for which ground slopes are very gentle, with perhaps some local steepening at stream embankments and shallow cuts. The Anchorage landslides all developed in soil masses with relatively steep embankments at the spreading front of 20 to 30 m in height. Despite this difference in morphology, the mechanisms of ground deformation for lateral spreads and the Anchorage slides are similar. The model proposed by Idriss for analyzing the Fourth Street slide [1985] is based on Newmark's approach [1965], and essentially is the same as the approach adopted by Castro for analyzing the lateral spread at Heber Road during the 1979 Imperial Valley earthquake [1987].

Five major landslides developed in the downtown and residential areas of Anchorage at Fourth Street, L Street, Government Hill, First Avenue, and Turnagain Heights [Seed, 1973]. The locations of these slides are shown in Figure 2-5. With the exception of the First Avenue slide, which was probably the result of oversteepening of the slope by excavation at the toe, liquefaction of sand layers and lenses was considered to be a factor in initiating these large block slides. As the soil moved out laterally toward the free face of the slope, large grabens developed behind the slides. The slides ranged from several hundred to over a thousand meters long, and moved laterally 4 to 8 m. The grabens which developed were generally 2 to 6 m deep [Wilson, 1967].

Figure 2-6 presents a schematic representation of the failure sequence in the Turnagain Heights Slide [Seed, 1973]. It is postulated that failure was initiated in this case by the liquefaction of sand lenses within the Bootlegger formation. Liquefaction of the sand lenses was accompanied by the loss of strength in very sensitive layers of the silty clay deposit. The soil mass moved out laterally on a horizontal failure plane. A complex system of ridges and depressions developed as blocks of soil moved on the liquefied mass, some rotating slightly. The slide was 2.6 km wide, and regressed about 370 m from the location of the original bluff. Lateral displacements of the soil mass were as much as 600 m into the bay [Seed, 1973].

Eckel [1967] conducted a study on the earthquake damage to utilities in the south-central portion of Alaska, in which he described the disruption to

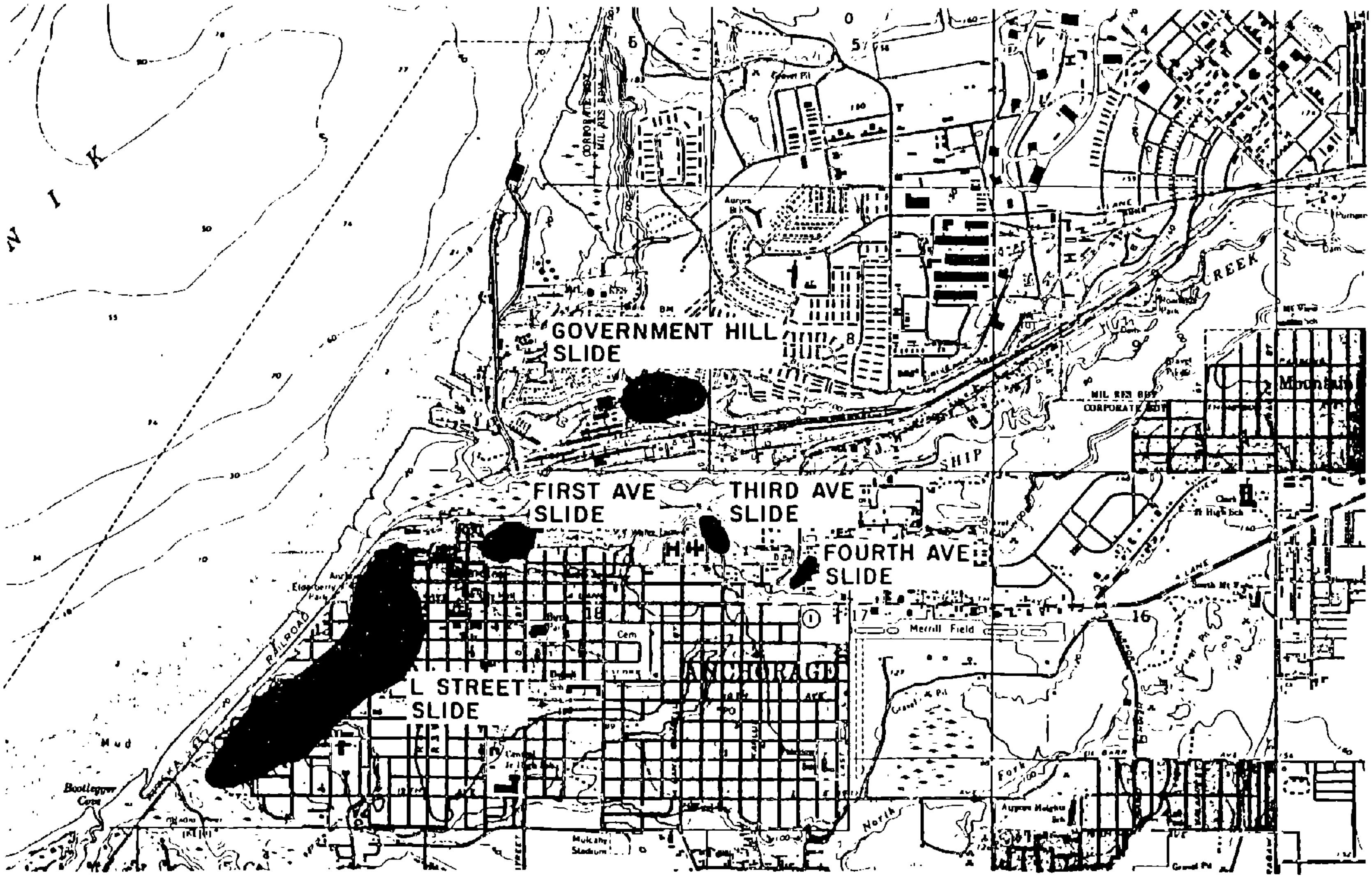


FIGURE 2-5. Location of Major Landslides in Anchorage - 1964 Alaska Earthquake [after Wilson, 1967]

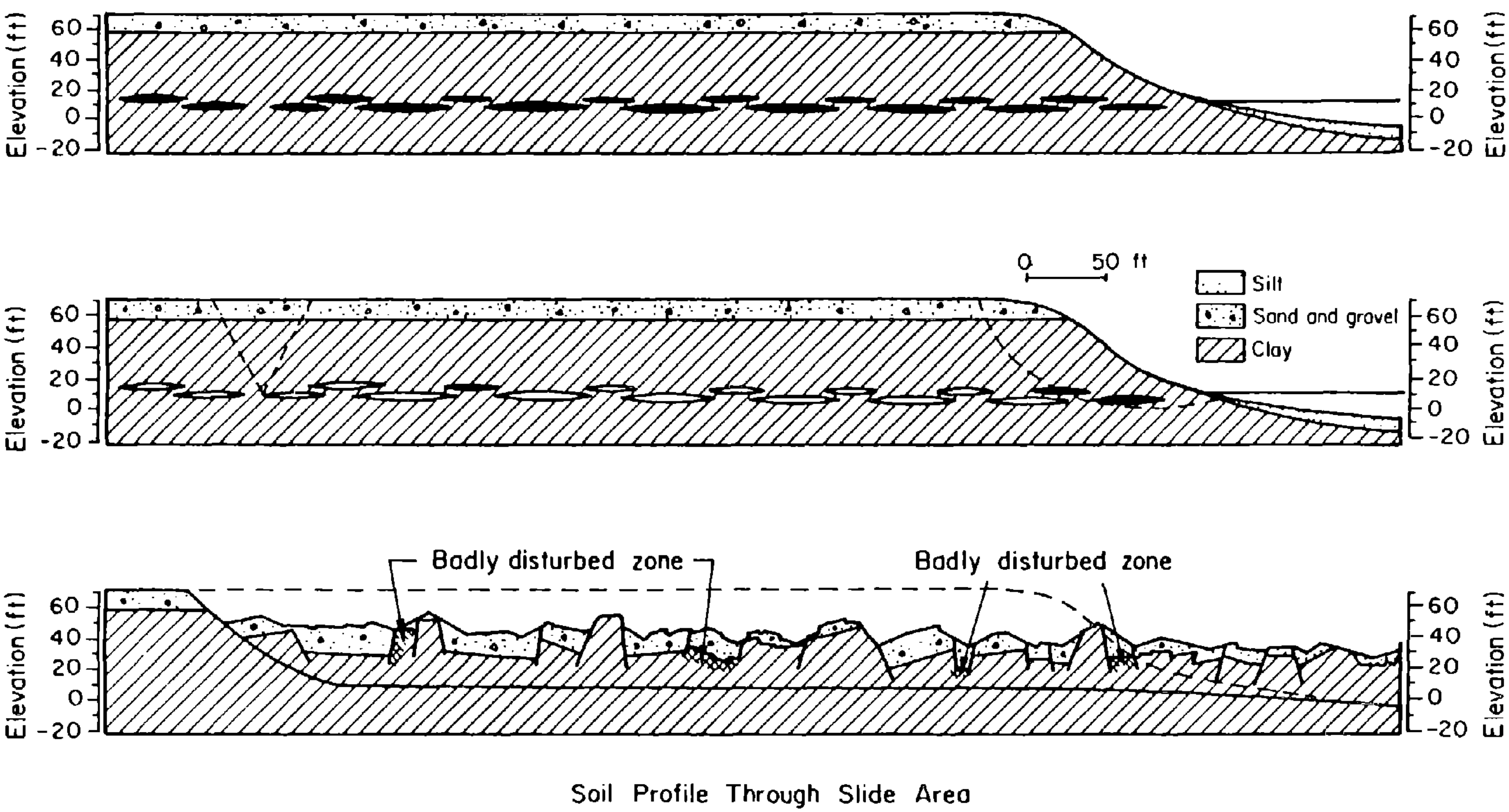


FIGURE 2-6. Conceptual Development of Turnagain Heights Landslide, Anchorage, During 1964 Alaska Earthquake [after Seed, 1968]

utility networks in Anchorage. Damage to the gas distribution system within Anchorage was estimated at $1 million. Some 200 breaks occurred in the gas distribution system, most of which developed at the grabens of the landslides. The pipes were found to have failed in tension, compression, shear, and repeated flexing. The water distribution and sewer systems also were severely damaged. Landslides and ground fractures broke water pipes in nearly 100 locations.

Slides similar to the Turnagain Heights slide developed during the 1811-12 New Madrid earthquakes along 56 km of the Chickasaw Bluffs, which face the Mississippi bottoms from Hickman, Kentucky to the mouth of the Obion River in Tennessee. Fuller [1912] described the development of "compound fissures" from the earthquake which have the appearance of narrow down-faulted blocks between two parallel cracks. A complex series of troughs and ridges developed behind the slides, which were described by Fuller [1912] as "sharp ridges of earth (which) alternate with deep gashes, the whole surface locally being broken into a jumble of irregular ridges, mounds, and hummocks, interspersed with trench or basin-like hollows and other more irregular depressions." The "canal-like" depressions were reported as "being as much as 100 ft (30.5 m) wide...100 ft deep and varying from a few feet to 100 feet wide" [Fuller, 1912]. The soil profile of the bluffs consists of a thick layer of shaley clays which extend below the base of the cliff, overlain by deposits of silts, gravel, and then by loess. Water-bearing layers containing sand occur at the contacts of clay strata, which are postulated as the surface of ground slippage [Fuller, 1912].

2.6.3 Bearing Capacity Failures

Loss of bearing capacity has received much attention as a result of failures such as those during the 1964 Niigata earthquake. Buildings tipped over and settled when the soil supporting them liquefied. The most severe failures occurred at an apartment complex in Kwangishicho, Niigata, one building of which rotated as much as 60 degrees [Youd, 1978]. The apartment complex buildings were underlain by a 15-m layer of loose sand overlying a dense sand deposit. The groundwater table was about 1 m beneath the surface at the time. Liquefaction is believed to have initiated in the middle-to-lower part of the loose sand, and then to have propagated upward.

Failures of this kind generally occur in deposits of saturated cohesionless soil that extend from near the ground surface to a depth of at least half of the building width. If the deposit which liquefies is shallower, then differential settlement, but not overturning of the structure, can result [Youd, 1978].

2.6.4 Subsidence

During an earthquake, when liquefaction occurs, the elevated pore water pressures often are relieved by the expulsion of water and sediment through a vent or crack, which results in the formation of sand boils. Volume loss from these ejections will cause differential surface settlement, even though no appreciable horizontal displacements occur [Committee on Gas and Liquid Fuel Lifelines, 1984]. During the 1959 Hebgen Lake earthquake, large sinkholes were formed, the largest of which was 13.7 m (45 ft) long by 4.6 m (15 ft) wide by 3.7 m (12 ft) deep [Swenson, 1964]. Ground settlements caused by densification of cohesionless soil deposits also occur commonly during earthquakes. This densification process often is associated with, and enhanced by, liquefaction [Committee on Earthquake Engineering, 1985].

2.6.5 Buoyancy

Buried structures, such as pipelines, tanks, and timber piles tend to rise buoyantly when the surrounding soil mass liquefies [Committee on Earthquake Engineering, 1985]. For example, during the 1964 Niigata earthquake, a sewage treatment tank, originally buried below ground, rose buoyantly such that 3.0 m of the tank were exposed above the ground surface after the earthquake [Seed and Idriss, 1967].

Buoyancy-related failures are most likely to occur in areas where the ground slope is virtually flat, and where the groundwater table is at or near the surface, such as in flood plains and estuaries [Committee on Gas and Liquid Fuel Lifelines, 1984]. Recommendations to limit the buoyancy-related displacements of pipelines have been given by Kennedy, et al. [1977].

2.6.6 Ground Oscillation

The Committee on Earthquake Engineering [1985] has identified a transient form of deformation which occurs when liquefaction develops at slopes too gentle to allow permanent lateral displacement. Under these conditions, liquefaction at depth may decouple overlying intact soil blocks so that they move back and forth on the liquefied layer during the earthquake. The ground oscillation is accompanied frequently by opening and closing of fissures and surface settlement.

2.7 CASE HISTORIES OF LATERAL SPREADING

Lateral spreads are regarded by some researchers as the most significant liquefaction hazard for buried lifeline facilities [Kennedy, et al., 1977]. Their troublesome nature can be linked with at least three characteristics. Because a lateral spread involves the movement of competent soil, full passive soil pressure can be mobilized against a buried pipeline. Lateral spreads are a relatively common occurrence. They have been classified by Keefer [1984] as abundant, on the basis of their relative frequency during 40 historical earthquakes. Finally, the locations of lateral spreads may be difficult to identify in advance, given that spreads have developed on slopes as gentle as 0.3 degrees.

Lateral spreads have long been perceived as an expression of ground failure, but their relationship with soil liquefaction and importance for buried pipelines have been recognized mainly within the last 10 to 15 years. Keefer [1984] has identified instances of lateral spreading in 26 of 40 historical earthquakes. Evidence of lateral spreads can be found for most major U.S. earthquakes, including the 1811-1812 New Madrid [Fuller, 1912], 1886 Charleston [Dutton, 1889], 1906 San Francisco [Lawson, et al., 1908], 1933 Long Beach [O'Rourke and Tawfik, 1983], 1964 Alaska [McCulloch and Bonilla, 1970], 1971 San Fernando [Youd, 1973], 1979 Imperial Valley [Youd and Bennett, 1983], and the 1983 Coalinga [Keefer, et al., 1984].

Four earthquakes were selected as part of this study for special review. They include the 1906 San Francisco, the 1964 Alaska, the 1971 San Fernando, and the 1983 Nihonkai-Chubu earthquakes. These examples of lateral spreading have been chosen because descriptions of permanent ground movements are available, as well as information about the effects of lateral displacements on lifeline systems.

2.7.1 1906 San Francisco Earthquake

The 1906 San Francisco earthquake, which has been assigned a Richter magnitude of 8.3 and a maximum Modified Mercalli intensity of XI, is the largest historical seismic event to have struck northern California. A large region, extending for over 600 km along the Coast Range province from Eureka in the north to the southern end of Monterey County, was affected by earthquake-triggered ground failures [Youd and Hoose, 1978].

Youd and Hoose [1978] summarized cases of historic ground failures associated with earthquakes in northern California in which they categorized the failures into seven types, the principal ones being hillside landslides, lateral spreads and ground settlement, and ground cracks. Of the liquefaction-induced ground failures, it was found that lateral spreads were the most common and also the most disruptive. Damage to bridges, roadways, structures, and pipelines caused by lateral spreads was extensive. Lateral spreading during the 1906 earthquake was most common in flood plains, tidal marshes, and in the filled areas, or "made land," of San Francisco.

Lawson, et al. [1908] reported lateral spreads, or "earth lurches," which occurred in the flood plains of several rivers: the Eel River, west and north of Ferndale; the Russian River; Alameda Creek, near Alvarado; the Coyote River near Milpitas; the Pajaro River; and the Salinas River. He described the ground displacements seen at these locations as, "...cracks were formed in the alluvium, generally parallel to the stream trench, and the ground between the cracks was caused to lurch horizontally toward the stream, usually with a rotation of the moved mass, which gave to it the profile of a Basin Range fault-block in miniature, the portion of the moved strip farther from the stream collapsing into the vacuity caused by the lurching."

Outside the vicinity of San Francisco, several noteworthy lateral spreads were reported [Youd and Hoose, 1978]. The most extreme lateral spreading occurred in the mud flats of Tomales Bay near Inverness. Horizontal displacements up to 7.6 m were reported. At Moss Landing on the Salinas River, lateral spreads of 2.7 and 3.7 m damaged a railroad bridge and several buildings. Near the Russian River, a roadway and fence were displaced 3 m.

Within the city of San Francisco, lateral spreading occurred in filled areas of former streams, bays, coves, and marshes. Maximum horizontal displacements of 1.8 m were reported at two backfilled marshes. Although these displacements were not as severe as those seen in regions outside of San Francisco, they caused great damage since they occurred in densely populated regions. In particular, lateral spreading was responsible for extensive damage to the water distribution network. Without an adequate supply of water, the fire fighting efforts of the city were severely hampered, and over 500 city blocks ultimately were damaged or destroyed by fire. Because of the special importance of the pipeline system response, this case history was chosen for an in-depth study presented in Section 3.

2.7.2 1964 Alaska Earthquake

On March 27, 1964, at 5:36 p.m., an earthquake of surface-wave magnitude 8.4 hit Alaska. The unusual feature of this earthquake was its duration; strong ground motion was estimated to have lasted three to four minutes, with perceptible motion lasting as much as 7 to 15 minutes [McCulloch and Bonilla, 1970].

Ground cracking, most of which was attributed to "land-spreading" [McCulloch and Bonilla, 1970], affected an area approximately the size of California along the coastal region from the southwest end of Kodiak Island, along the Kenai Peninsula, to about 240 km north of Valdez. Most horizontal soil displacements occurred in nearly flat areas in the direction of a local depression, such as a stream channel. Along rivers, the area affected by lateral spreading commonly extended for 150 m to as much as 300 m behind stream banks [McCulloch and Bonilla, 1970].

Virtually all utilities and communications in the south-central section of Alaska were damaged [Eckel, 1967]. Initial estimates of the damage to the highway and railroad systems were on the order of $46 and $30 million, respectively. One hundred and fourteen of the 204 highway bridges in this region were damaged by densification or lateral displacement of the foundation materials [Kachadoorian, 1968].

The worst highway damage occurred on the Seward-Anchorage Highway at the

Snow River Crossing, the Richardson Highway, and the Copper River Highway. In these areas, the soils underlying the roadways are nonplastic silts and sands, with some gravels. The water table is within 1 to 2 m of the ground surface. The most severe damage occurred in areas where the foundation material consisted of fine grade cohesionless soils (silts and sands), with the water table within the top 0.3 m. Where liquefaction occurred, the liquefied material moved from the road fill, causing the roadway to fracture, subside and displace laterally. A maximum of 4.1-m horizontal movement and 3.4-m subsidence occurred at the Snow River Crossing, where the groundwater table is within 0.3 m of the surface [Kachadoorian, 1968].

Railroad bridges in south-central Alaska also sustained heavy damage from earthquake-induced soil displacements. Soil movements at the damaged bridges started at the edges of stream channels and other topographic depressions. It has been postulated [McCulloch and Bonilla, 1970] that the affected zone widened as the earthquake motion continued. Sediments converged on streams from both sides, creating zones of compression in the centers of the streams, and zones of extension beside the stream banks. Tension cracks formed parallel to the stream banks, and in the zones of compression, upward displacements of sediments occurred, often carrying bridge piles upward. A schematic drawing of typical sediment and bridge pile displacements is presented in Figure 2-7. Soil deformations extended to depths of at least 10.7 m generally, and over 38.1 m in several cases, as evidenced by the fact that piles, which were driven to these depths, moved horizontally without rotation. Lateral spreading decreased the width of streams a maximum of 2.0 m. An investigation of bridges failed by displacement of foundation materials showed that the underlying deposits consisted of waterlaid, noncohesive sediments, all geologically young, with the groundwater table within a meter of the surface. All severely damaged bridges were founded in alluvial deposits over 30 m thick. The maximum horizontal displacement across a stream was measured as 3.0 m [McCulloch and Bonilla, 1970].

McCulloch and Bonilla [1970] recognized the close relationship between the damage to the railroad system and the surficial geologic and physiographic characteristics of the region. The identification of only a few easily

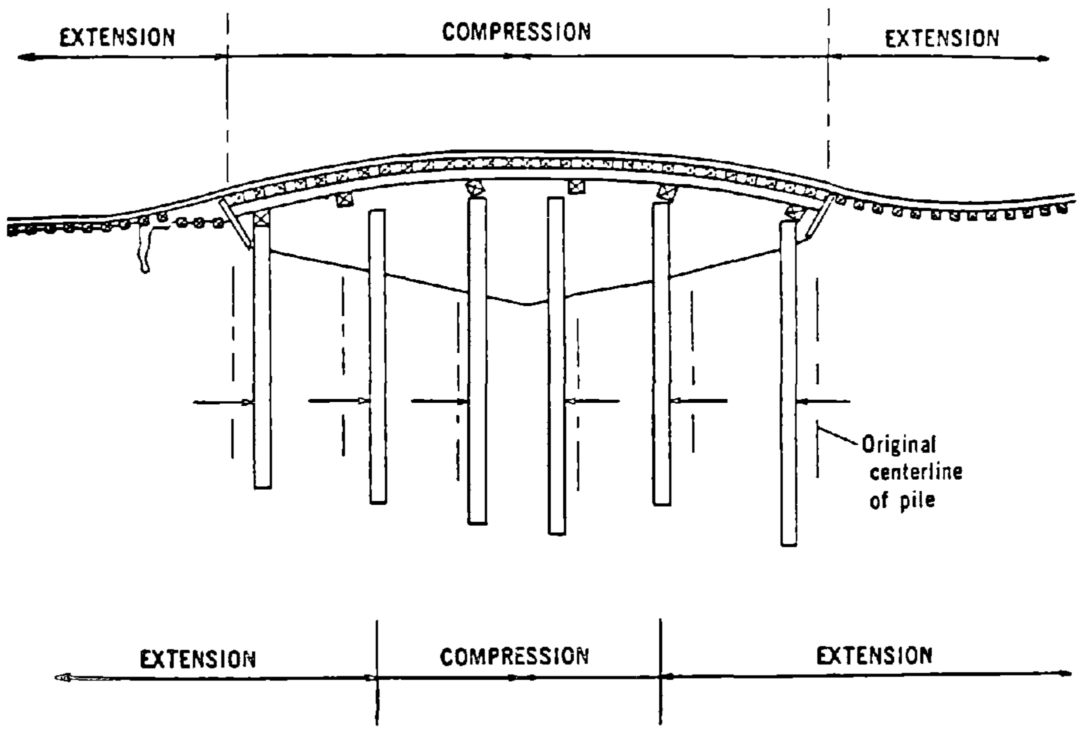


FIGURE 2-7. Typical Displacement of Bridge Piles [after McCulloch and Bonilla, 1970]

distinguishable geologic units was required to bound the zones of extensive damage. These included tidal flats, active flood plains, alluvial fans, deltas, and, to a lesser degree, inactive flood plains. It was suggested that, by mapping these deposits, a basis could be established for predicting the type of ground displacements and severity of resulting damage to be anticipated in large earthquakes under similar conditions.

2.7.3 San Fernando Earthquake - California

An earthquake, with a Richter surface-wave magnitude, M_S, of approximately 6.4, struck southern California on February 9, 1971. The San Fernando and Sylmar regions of Los Angeles County, located near the earthquake's epicenter, were severely damaged [Grantz, 1971]. Earthquake-induced landsliding and lateral spreading in the vicinity of the Upper Van Norman Reservoir, in particular, caused much damage.

The most damaging slide in this region, referred to as the Juvenile Hall slide, developed northeast of the Upper Van Norman Reservoir [Youd, 1971]. The slide, located in Figure 2-8 [Smith and Fallgren, 1975], extended from the reservoir a distance of 1.2 km and crossed several structures. Damage from this slide to the San Fernando Juvenile Hall, Sylmar Electrical Converter Station, Southern Pacific railroad tracks, Interstate 5, and several pipelines and canals was estimated to exceed $30 million [Youd, 1971].

The lateral spread was situated in a lowland created by the coalescence of three alluvial fans. Figure 2-9a shows the location of the lateral spread relative to the surrounding rock hills [O'Rourke, 1986]. The fan deposit, originating from Weldon, Sombrero, and Grapevine canyons, consists of sands, silty sands, and silts. The slide zone was underlain by a saturated sandy silt and fine sand layer at a depth of 6 to 9 m [Smith and Fallgren, 1975]. The groundwater table, as shown by the contours of equal water table depth in Figure 2-9a, was within 3 to 6 m of the ground surface. Evidence of liquefaction was provided by the occurrence of sand boils, shown in Figure 2-9b. The original slopes in the vicinity of the failure were mild, ranging from 0.9 to 3 degrees [Youd, 1971].

The pattern of permanent ground displacements of the lateral spread is shown in Figure 2-9b. The displacement vectors are shown relative to the location of buried pipelines which crossed the slide. The soil movements exhibited a distributed pattern that was punctuated at various locations by abrupt movements at ground ruptures. For example, at the center of the slide, an abrupt lateral offset of 0.58 m was observed. The maximum horizontal displacement of the lateral spread was approximately 1.75 m, which developed over a distance of 340 m.

The influence of the topography on the pattern of displacement is demonstrated in Figure 2-9c. The soil displacement vectors, represented by arrows in the figure, show that the direction of the flow changed from almost due south near the Juvenile Hall, to a southwest direction near the Golden State Freeway [O'Rourke and Tawfik, 1983]. The displacement vectors follow the slope of the lower valley, remaining perpendicular to contour lines.

A study was conducted of the damage to transmission pipelines in the Upper

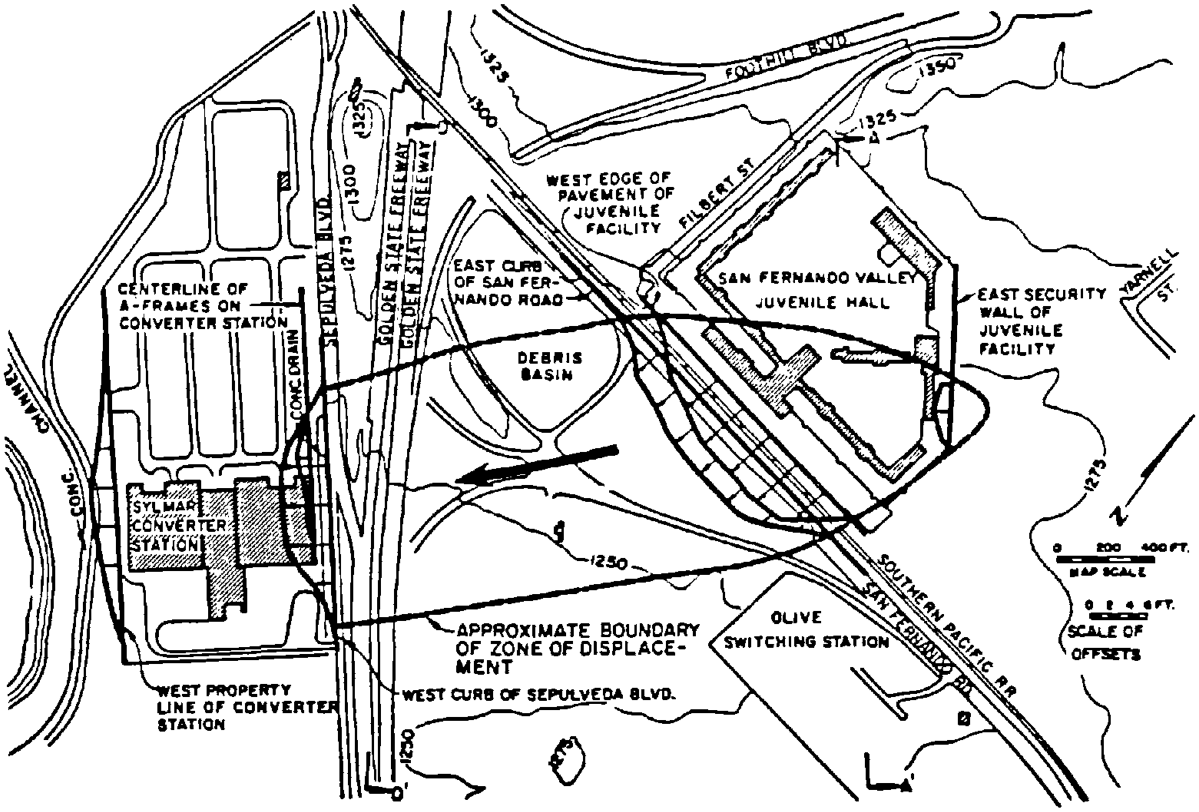


FIGURE 2-8. San Fernando Juvenile Hall Slide - 1971 San Fernando Earthquake [after Smith and Fallgren, 1975]

Van Norman Reservoir area as a result of liquefaction-induced soil displacements [O'Rourke and Tawfik, 1983]. A total of 11 pipelines crossed zones of landsliding and lateral spreading in the vicinity of the reservoir, four of which were intersected by the Juvenile Hall slide.

The locations of two high pressure, natural gas transmission pipelines are shown in Figure 2-9b. Both were continuous girth-welded steel pipelines, installed betweem 1926 and 1930. The two pipelines, designated Nos. 4 and 5 in the figure, ran parallel to San Fernando Road and crossed the path of soil displacements almost perpendicularly. Both of these pipes were severely damaged by the soil displacements. Pipeline No. 4 was repaired in seven locations, while Pipeline No. 5 was damaged at so many locations that it was abandoned.

In Figure 2-9b, the displacement vectors along San Fernando Road show the pattern of the ground deformation imposed on the pipes. In Pipeline No. 5,

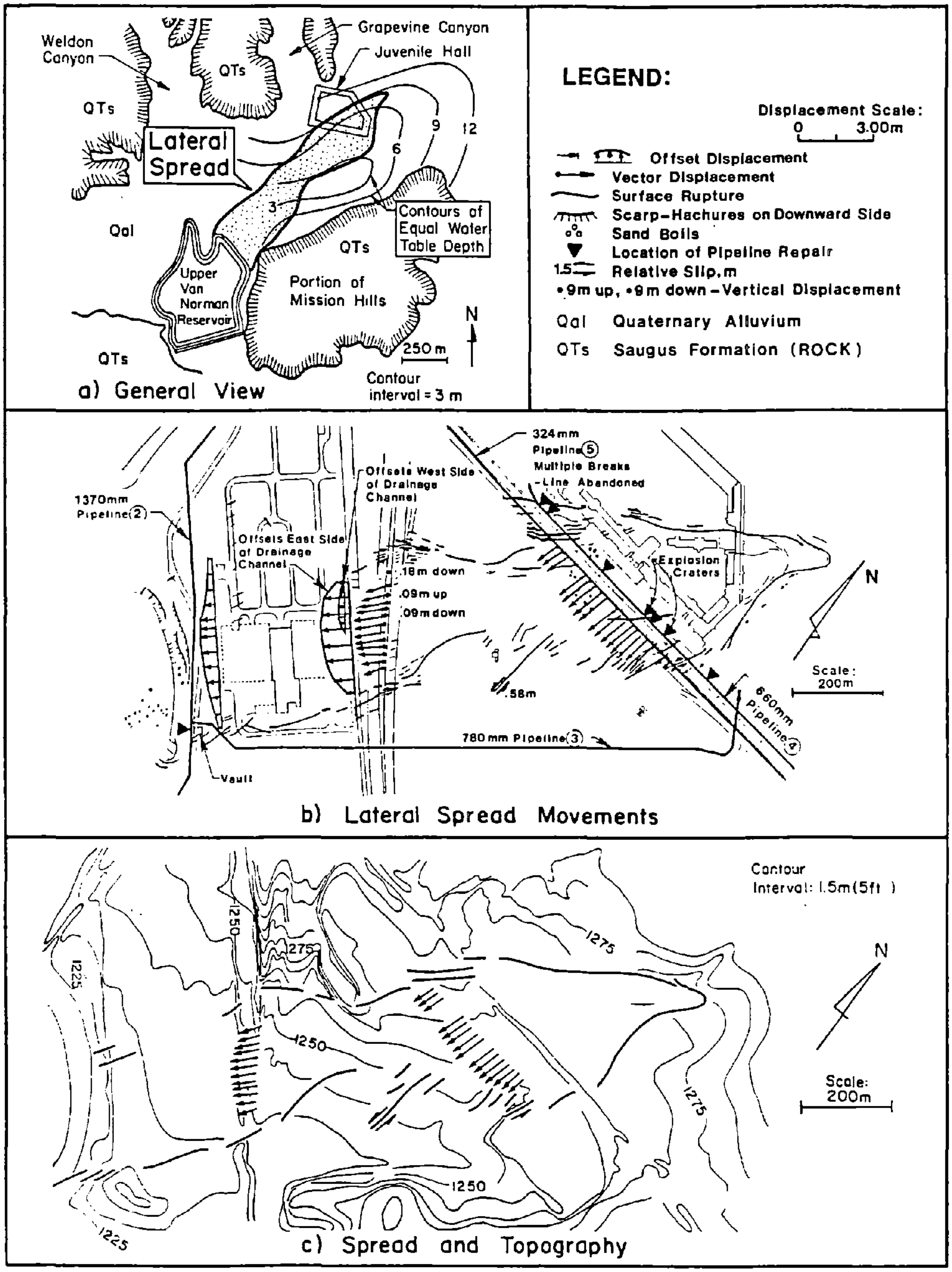


FIGURE 2-9. Plan Views of Lateral Spread on the East Side of Upper Van Norman Reservoir [after O'Rourke, 1986]

the initial ruptures probably occurred at the location of the explosion craters near the southeastern edge of the lateral spread. In this zone, left lateral soil displacements caused substantial tension in the pipe, which is consistent with the reported repairs of tensile failures.

Two steel water transmission lines, shown in Figure 2-9b as Pipeline Nos. 2 and 3, passed within or near the toe of the lateral spread. Only Pipeline No. 2 sustained damage. A mechanical joint, located at a ball valve where the pipe was connected to a reinforced concrete vault, was compressed and showed signs of repeated battering. Although the pipeline was subjected to a maximum horizontal displacement of approximately 0.7 m, it showed no other signs of damage.

2.7.4 **Nihonkai-Chubu Earthquake**

The Nihonkai-Chubu earthquake, which occurred off the northern coast of Honshu Island, Japan, registered a surface-wave magnitude, M_S, of approximately 7.7. Aikita and Aomori prefectures, along the northwest corner of the island, suffered extensive damage, especially to lifeline facilities, harbor structures, embankments of reclaimed land, and residences [Hamada, et al., 1985].

The city of Noshiro, in particular, experienced extensive damage to buried gas, water, and sewage pipelines. Most of this damage has been attributed to ground displacements associated with liquefaction. A study of the horizontal ground displacements resulting from the earthquake was performed by Hamada, et al. [1985], in which soil movements were related to damage sustained by pipeline networks.

Maximum horizontal displacements occurred along two sandy hilltops, Maeyama and Garyu, where the groundwater table was within a few meters of the surface. The two zones of large displacements are located in Figure 2-10. The magnitude and direction of the horizontal displacements were determined by measurements from pre- and post-earthquake aerial photographs. In Zone A, the larger displacements occurred along the lower slopes of Garyu Hill, the gradient of which was less than 3 degrees. The maximum displacement was greater than 3.0 m. In contrast, displacements were small in the eastern

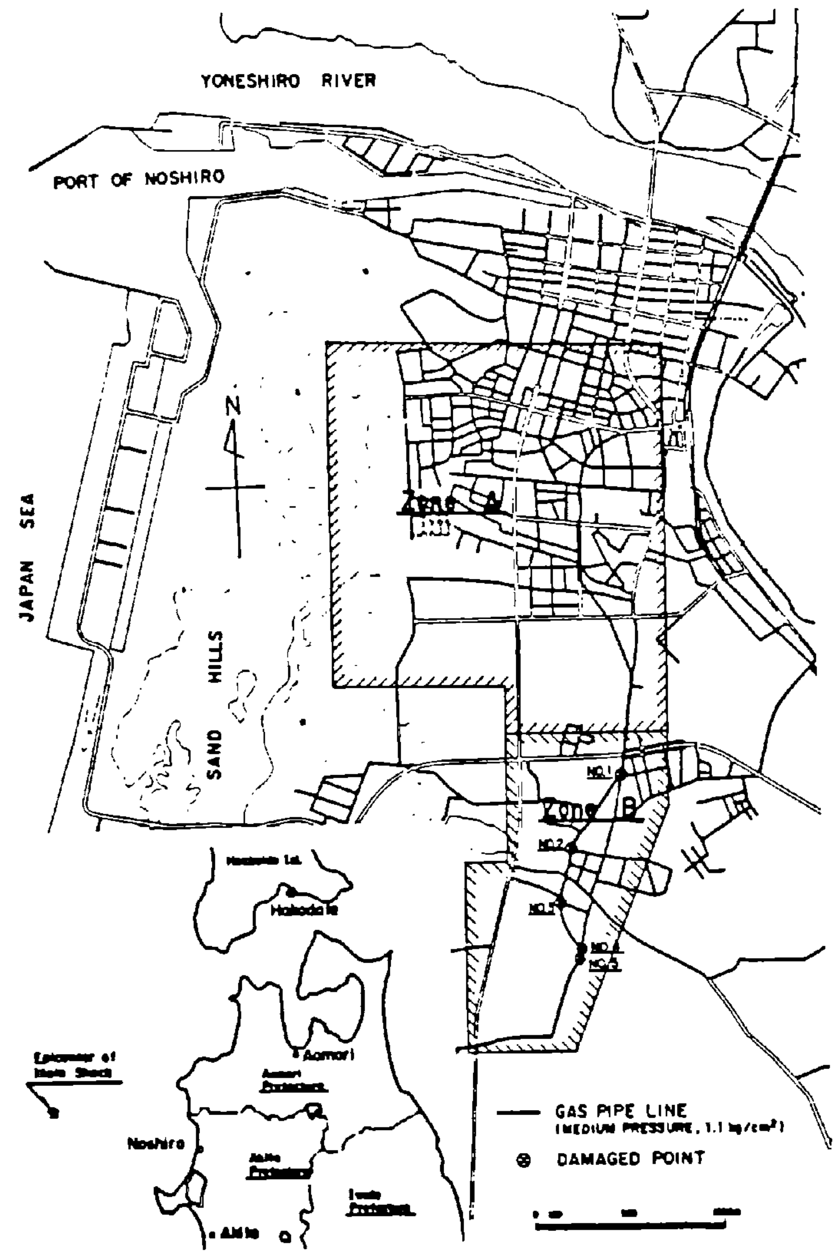


FIGURE 2-10. Zones A and B in Noshiro City Affected by Lateral Spreading [after Hamada, et al., 1985] (© ASME. Used by Permission.)

section of the zone, where the ground surface is essentially flat. In Zone B, the larger displacements occurred along the lower slopes of the Maeyama Hill. In this zone, the maximum horizontal displacement was greater than 5.0 m.

From evidence of displacements of vertical wells, liquefaction is assumed to have occurred just below the groundwater table at a depth of roughly 3 m. Field observations disclosed that lateral spreads had developed, leaving cracks perpendicular to the direction of flow near the head of the slides. At the toe of the lateral spreads, there was compression and swelling as the soil moved out and slightly up. The lateral spreads also were accompanied by sand boils.

Figure 2-11 shows the relationship between the magnitudes of horizontal displacements and the damage to lifeline systems in Zone A of Noshiro City. Magnitudes and directions of horizontal soil displacements are shown by the arrows in Figure 2-11a. In Figures 2-11b and 2-11c, locations of damage to low pressure gas lines and to water supply lines, respectively, are mapped. There is a clear correlation between the magnitude of horizontal displacements and the extent of damage to buried pipelines. The figure also shows that the most severe damage was sustained by the gas distribution network.

The study analyzed the damage to one 80-mm-diameter welded steel pipeline transporting natural gas. This pipeline was ruptured or cracked in five locations along a 1.5-km section. Two locations of failure of the gas pipeline, shown in Figure 2-10, were studied in detail. At location No. 2, the pipe buckled by compressive axial forces, and the bend broke by large bending stresses. From inspection of the pipe and from numerical analysis, it was concluded that the initial cause of the pipe failure was the compressive permanent ground displacement. At location No. 4, the direction of the lateral spread was nearly perpendicular to the axis of the pipeline, with a maximum displacement over 2.0 m. Here the pipe was broken by bending deformation at the elbow.

Axial tensile forces were found to be important. Inspection of the damaged pipe revealed that the pipe was separated for a distance of 700 mm at the elbow.

Another study [Kawashima, et al., 1985] was conducted on the damage to sewage pipelines in Noshiro City from the 1983 earthquake. In the precast reinforced concrete sewage network, 8,757 m of the 60,000 m of pipeline were

FIGURE 2-11. Relationship Between Ground Displacements and Damage to Pipelines in Zone A, Noshiro City [after Hamada, et al., 1985] (© ASCE. Used by Permission.)

damaged, the majority of which were located in the zones affected by liquefaction. Offset and pull-out of pipe joints were the most common sources of damage resulting from the extensive lateral spreading. Cracking and rupture of pipes also developed near structures such as manholes.

SECTION 3

CASE STUDY - 1906 SAN FRANCISCO EARTHQUAKE

3.1 INTRODUCTION

The aim of this section is to describe and quantify liquefaction-induced ground movements observed during the April 18, 1906 San Francisco earthquake. This work draws upon the results of previous studies [e.g., Youd and Hoose, 1978; Hovland and Darragh, 1981; Roth and Kavazanjian, 1984], all of which have focused on the San Francisco area. It differs from previous investigations in the detail with which ground movement patterns are mapped and related to subsurface soils and buried topographical features. Moreover, this study evaluates the pattern of damage in the water pipeline system at the time of the earthquake by mapping pipeline breaks in relation to ground movements and subsurface conditions.

The 1906 San Francisco earthquake was chosen for study, not only because of the extensive documentation available, but also because the lateral spreading associated with this earthquake was principally responsible for the disruption of the water supply system in the downtown and business districts. Without an adequate water supply, the city was severely hampered in its fire-fighting efforts, with the consequent destruction of over 500 city blocks [Himmelwright, 1906].

The section begins with a brief description of the 1906 earthquake and the extent of damage in the downtown area of San Francisco. Zones of permanent ground deformation are identified and discussed, with emphasis on the patterns of liquefaction-induced soil displacements. Damage to the water distribution piping system is related to zones of permanent ground movement, and pipeline ruptures are plotted relative to subsurface soil and groundwater conditions. A general summary is given of liquefaction-induced ground movements, with emphasis on displacements caused by lateral spreading.

3.1.1 Description of Earthquake and Extent of Damage

The 1906 earthquake struck the middle coast region of California on the morning of April 18 at 5:13 a.m. A magnitude of 8.3 has been assigned to the earthquake [Richter, 1958]. The duration was approximately one minute

[Brown, et al., 1932]. The shock was felt as far north as Coos Bay, Oregon to as far south as Los Angeles, California, and to the east as far as Winnemucca, Nevada, or 90 km inland from the coast [Lawson, et al., 1908]. Ground failures were triggered by the earthquake over a 600-km-long segment of the Coast Range [Youd and Hoose, 1978].

Within the City of San Francisco, damage resulting from the earthquake was estimated at roughly 500 million dollars, with a possible loss of 800 lives [Gilbert, et al., 1907]. For San Francisco, it is important to distinguish between damage directly caused by the earthquake and damage resulting from fire following the earthquake. The report of the Earthquake Commission [Lawson, et al., 1908] estimated that the damage resulting solely from earthquake effects was roughly 20 percent of the total. The destructiveness of the fire, however, was itself a consequence of the earthquake [Richter, 1958].

Figure 3-1a shows the variation of earthquake intensity in the San Francisco peninsula. The earthquake intensity is expressed in terms of the Modified Mercalli system as proposed by Richter [1958] from an intensity scale originally developed for the San Francisco earthquake, which was reported by Lawson, et al. [1908].

Most of the City of San Francisco was influenced by an intensity of MMI VII to VIII, whereas only about 5% of the built-up area was affected by MMI IX to X. Zones of MM IX to X intensity are useful for delineating locations of earthquake-induced ground failures. An enlarged map of northeast San Francisco is shown in Figure 3-1b to define more clearly the zones of high intensity in the most heavily developed portion of the city.

The large ground movements, which occurred in only a relatively small portion of the city, had a profound influence on the fire damage. For example, lateral spreads and subsidence along Valencia Street ruptured 400 and 500-mm-diameter pipelines, leading directly to the loss of 43,000 m^3 of water from the College Hill Reservoir in 24 hours [Schussler, 1906]. This loss of water contributed to the difficulties in controlling and extinguishing fires. More than 10.6 km^2 were eventually burned, destroying 490 city

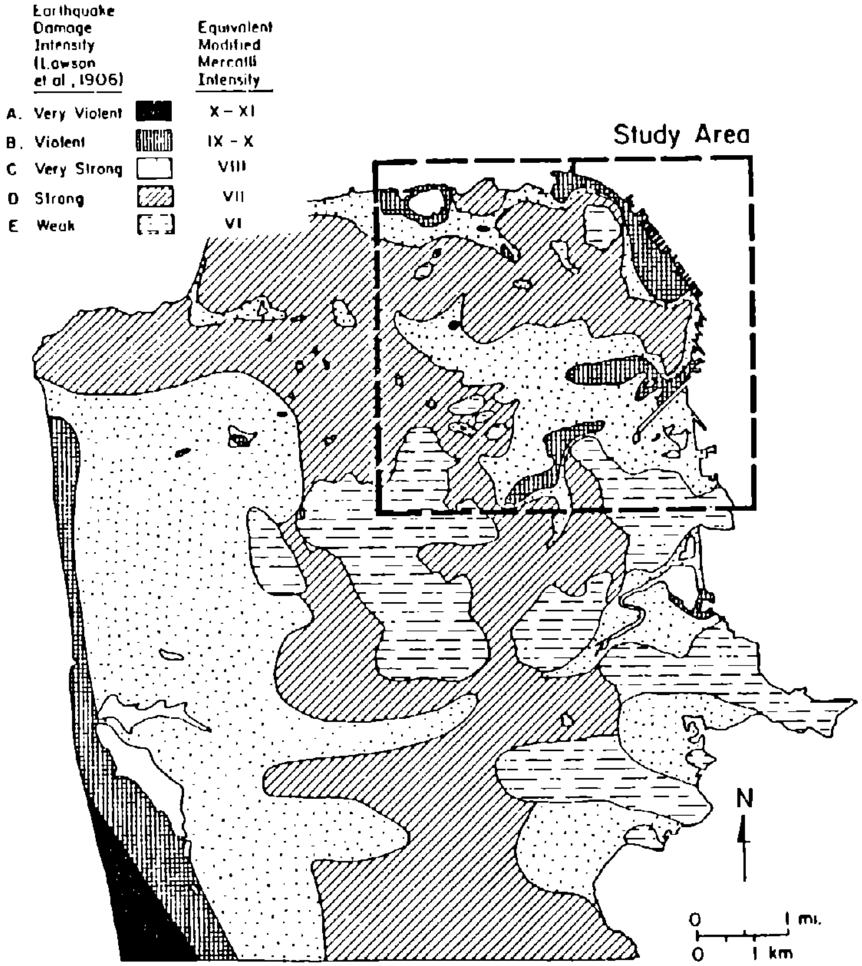


FIGURE 3-1a. Distribution of Earthquake Intensity (Modified Mercalli Scale), 1906 Earthquake, San Francisco County [after Youd and Hoose, 1978; Lawson, et al., 1908]

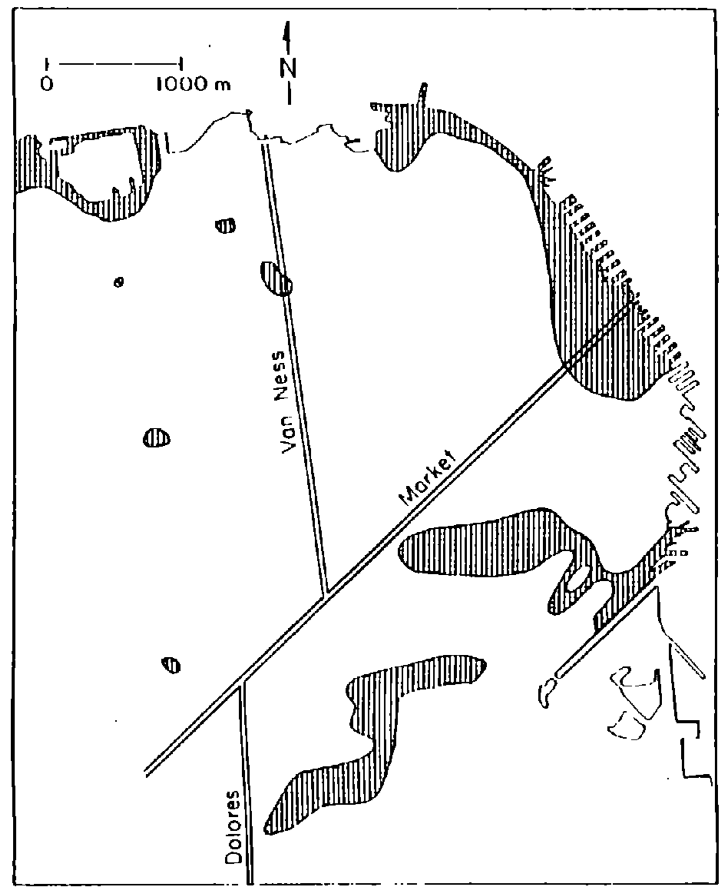


FIGURE 3-1b. Zones of MM IX–X Intensity in Study Area, Northeast Section of San Francisco [after Lawson, et al., 1908]

blocks and causing partial destruction of an additional 32 blocks [Gilbert, et al., 1907].

3.1.2 Zones of Liquefaction-Induced Ground Movements

It has been pointed out by several researchers [e.g., Lawson, et al., 1908; Youd and Hoose, 1978] that severe movements at the time of the earthquake occurred in areas where fill had been placed along the water front, inlets, coves, marshes, and ravines. In Figure 3-2a, areas of fill [Schlocker, 1974] are mapped in conjunction with zones of large permanent ground deformations delineated by Youd and Hoose [1978] and Hall [1906]. Three major zones of ground failures were identified by Youd and Hoose as the Mission Creek, South of Market, and Foot of Market Zones, and each of these are shown in the figure by heavy solid lines. In Figure 3-2b, areas of fill are mapped in conjunction with zones of MM IX to X intensity. It can be seen from Figures 3-2a and b that there is a close correspondence between areas of fill, zones of high earthquake intensity, and locations of large permanent ground movements.

Figure 3-2b provides a convenient reference for identifying locations of potentially large earthquake-induced movements, and serves in this section as a means of organizing the discussion according to three general areas of displacement. The first area of study includes the zones of permanent soil movements mapped by Youd and Hoose [1978] as the Mission Creek, South of Market, and Foot of Market Zones. The second general area includes the Marina and North Beach Districts along the northern shore line. Substantial amounts of fill were placed between 1908 and 1930 in these two areas, with the result that the soil conditions are similar to those in areas which experienced extensive ground deformation during the 1906 earthquake. The third general area includes several small zones which were identified on the basis of high earthquake intensity (equivalent to MM IX) by Lawson, et al. [1908] at the following locations: 1) Duboce Park, 2) Steiner and Sutter Streets, 3) Union and Steiner Streets, 4) Vallejo Street and Van Ness Avenue, and 5) Lombard and Octavia Streets.

3.1.3 Scope of Work

To delineate the boundaries of the filled areas at the time of the

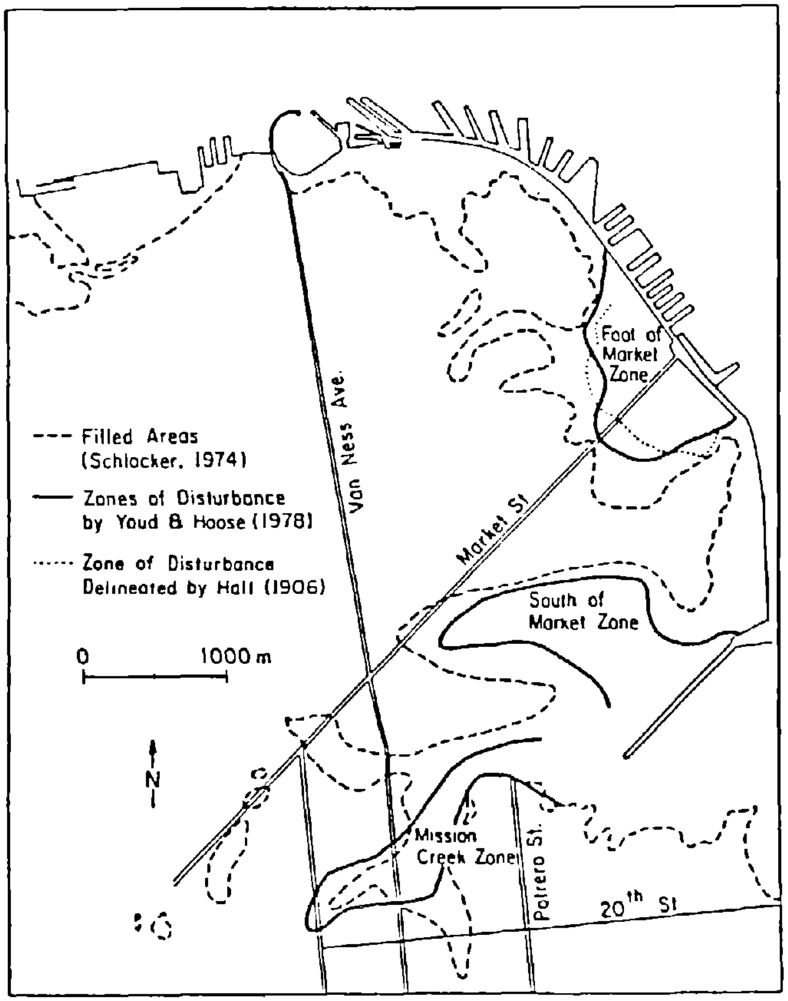


FIGURE 3-2a. Zones of Disturbance [after Youd and Hoose, 1978; Hall, 1906] versus Areas of Fill [after Schlocker, 1974] in Study Area

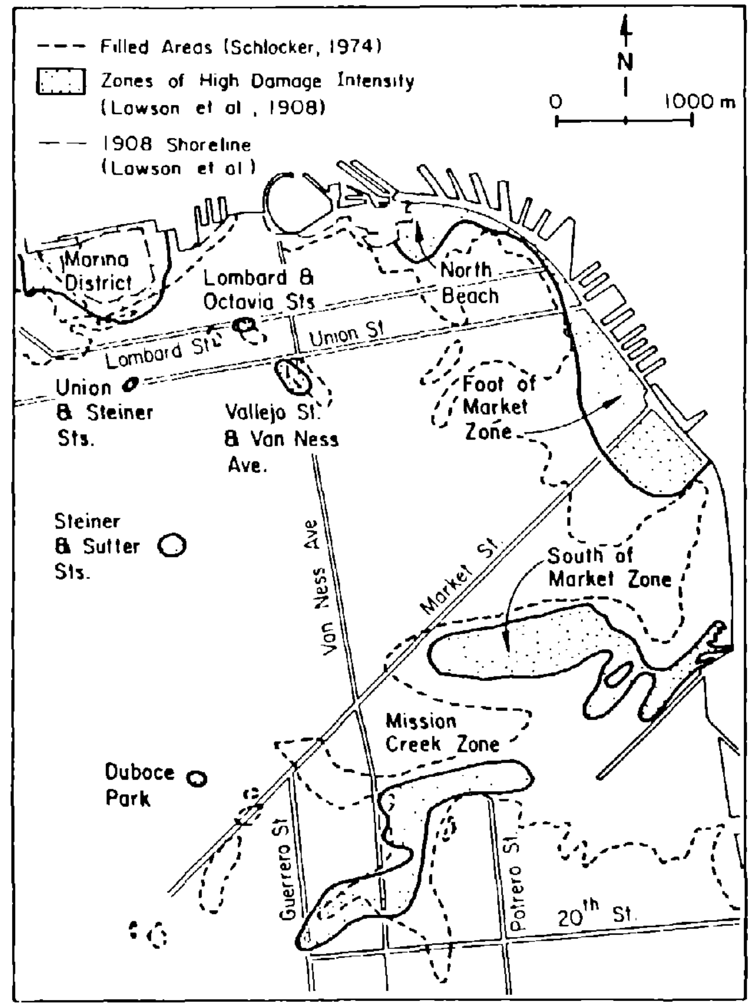


FIGURE 3-2b. Zones of MM IX–X Intensity versus Fill in Study Area

earthquake, the original topographical features drawn on the 1853 U. S. Coast Survey of the City of San Francisco and Its Vicinity were related to the 1908 street maps, soil surveys [Schlocker, 1974], and current street and topographic maps. Features such as Nob Hill, Potrero Hill, and Russian Hill were used to fix the superposition of the original 1853 survey and later maps showing the street system. Available literature was also reviewed for information on subsurface conditions in each zone [Youd and Hoose, 1976, 1978; Hall, 1906; Hovland, 1980; Schlocker, 1974], the history and nature of the fills [Olmsted, et al., 1977; Roth and Kavazanjian, 1984; Hovland and Darragh, 1981], and the liquefaction potential of subsurface soils [Roth and Kavazanjian, 1984; Youd and Hoose, 1978].

From a review of photographs and historical accounts [Schussler, 1906; Youd and Hoose, 1976, 1978; Lawson, et al., 1908; Gilbert, et al., 1907; Duryea, et al., 1907; Himmelwright, 1906; Hyde, 1906; Leonard, 1906; Derleth, 1906; Kurtz, 1906; Jordan, 1907; Hall, 1906; Newman, 1906; Hovland, 1980], a catalog of ground displacements was compiled. Several hundred photographs were examined, of which approximately 85 were selected for detailed study of the ground displacements within the zones of high damage intensity. Cultural objects with standard dimensions, such as curb stones, bricks, and the separations between utility poles, were used to judge the magnitude of movement and distance between points of displacement. Table A-I, given in the Appendix, lists the documented permanent ground deformations from both photographs and literature.

The interpretation of ground movement patterns was aided by reference to a map of earthquake-induced pipeline breaks prepared by Manson [1908], and a report on the pipeline system performance by Schussler [1906].

3.2 HISTORY AND NATURE OF FILLED AREAS

The zones of "made" or filled land have been identified as areas of high damage intensity during the 1906 earthquake. In this section, the history and nature of these filled zones are discussed to provide a better understanding of their behavior during the earthquake. A summary of the changes in the shore line of San Francisco is shown in Figure 3-3 relative to the streets, thus giving a picture of the progression of filling operations

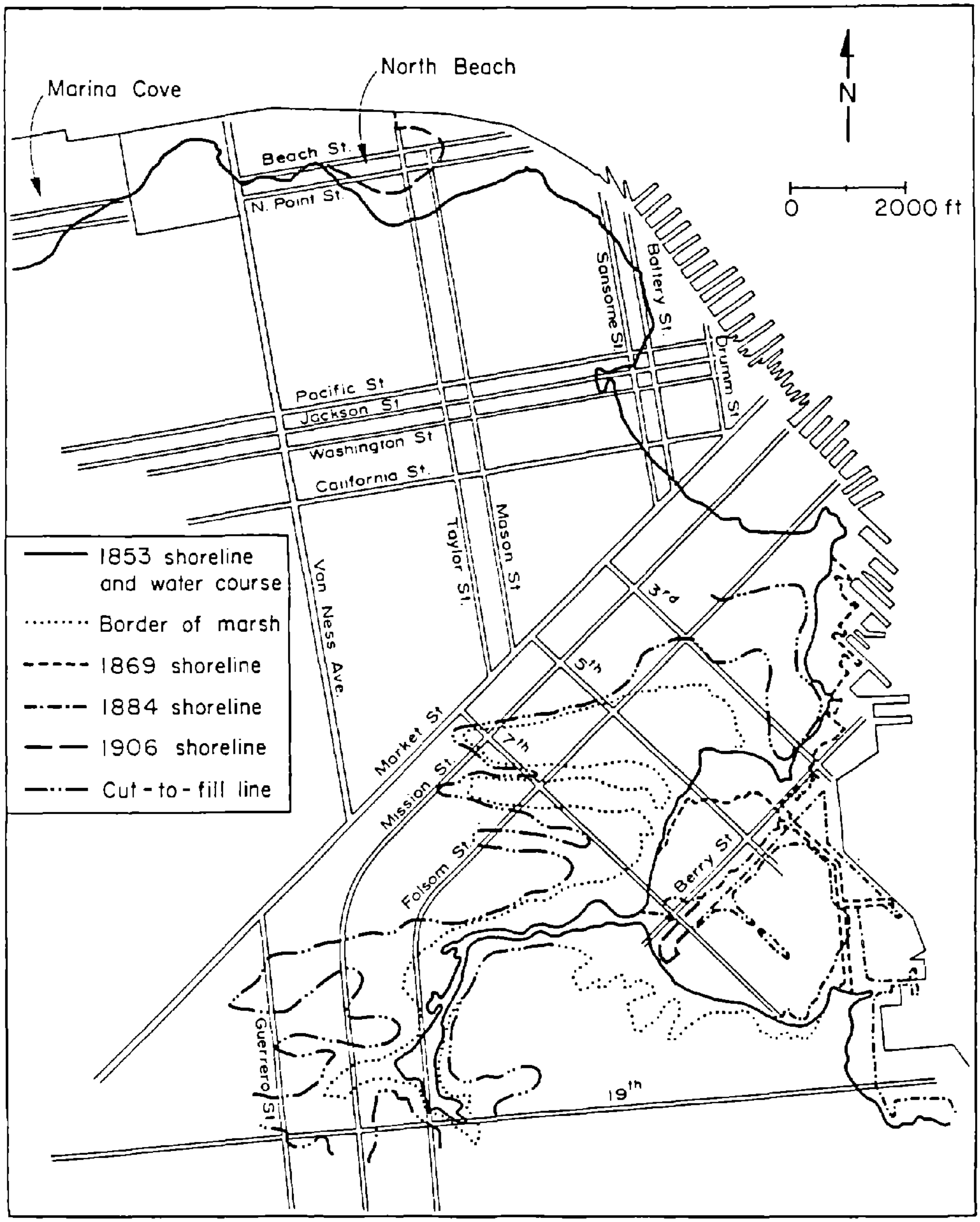


FIGURE 3-3. The Development of the Shore Line in San Francisco Since 1853 [after Olmsted, et al., 1977; Roth and Kavazanjian, 1984; Hovland and Darragh, 1981]

relative to the time span over which they occurred.

Filling of the Yerba Buena Cove was begun in the 1850's. Long wharves were built into the deeper water to accommodate ocean-going ships at Market, California, Washington, Jackson, Pacific, and Broadway Streets. Crosswalks were built perpendicularly to connect the wharves (later, these crosswalks became Sansome, Battery, and Drumm Streets). A program of cut and fill was then initiated in which the sand dunes to the west were systematically excavated and the material was loose-dumped between the wharves and crosswalks. When a sea wall was constructed, beginning in 1867, the bay mud dredged for its foundations was also used for fill [Roth and Kavazanjian, 1984]. Refuse from the city and from several industries was also dumped into the water lot areas [Hall, 1906; Olmsted, et al., 1977; Goldman, 1969]. Filling operations in the cover were completed by 1900 [Roth and Kavazanjian, 1984].

Before the development of the city, several valleys cut through the high rocky ridges which rimmed the eastern shore of the former Mission Bay. The most prominent of these was Mission Valley, which opened into the bay between Rincon and Potrero Points [Lawson, et al., 1908]. One arm of the valley extended northwestward towards 7th and Mission Streets (south of Market Zone). This was formerly the site of a salt marsh known as Sullivan's Marsh [Hall, 1906]. The marsh has been described as "subterranean lakes, forty to eighty feet deep (12 to 24 m), crusted with a ten-foot (3-m) layer of peat strong enough to bear the weight of a small house..." [Brown, et al., 1932]. The former Mission Creek wound around Potrero Hill and then south and westward into a tidal lagoon and contiguous salt marsh [Lawson, et al., 1908] as far south as 19th and Guerrero (Mission Creek Zone). South of the city was another prominent valley, Islais Creek, which emptied into Mission Bay as well.

Difficulties were encountered in filling sections of Folsom Street, which crossed Sullivan's Marsh. The marsh was filled mainly with sand, which often settled as much as 2 m overnight, displacing the mud and causing it to heave [Brown, et al., 1932]. By 1869, filling had been completed in Sullivan's Marsh as well as a portion of the north shore of Mission Bay, although Mission Creek was still shown on the 1869 U. S. Coast survey map [Olmsted, et al., 1977]. The 1877 Sanborn Index Map [Olmsted, et al., 1977] showed that filling had been completed in the Mission Creek area, with additional fill having been placed along the north shore of Mission Bay as far as Berry Street.

The southern portion of Mission Bay below the channel was filled during the period 1884 to 1906 [Hovland, 1980]. During this time, the sand dunes had already been removed, and thus the fill probably consisted of rock and refuse debris [Hovland and Darragh, 1981]. Lawson, et al. [1908] state that much of the material used in filling the bay was broken rock taken from the grading of neighboring rocky hills.

To protect the water lots at Marina Cove, a sea wall (known as Fair's Sea-wall) was begun in 1893 along the alignment of the current Marina Boulevard. The fill consisted of sand and rock from the San Bruno Mountain quarry, and spoil from nearby excavations. Beginning in 1912, fill was placed south of the sea wall to prepare the site for the 1915 Panama-Pacific Exposition. This fill was sand and mud pumped from offshore [Olmsted, et al., 1977]. Although the majority of filling was completed by 1915, filling north of Marina Boulevard continued into the 1930's [Olmsted, et al., 1977].

Along the northern shore of San Francisco were two coves, too shallow to serve as a site for anchoring ships, known as North Beach and Marina Cove in Figure 3-3. These areas were largely undeveloped at the time of the earth-quake. In the North Beach District, the majority of the filling was done between 1906 and 1915 [Roth and Kavazanjian, 1984]. The fill along North Point Street up to Mason Street dates from the mid-1880's, and at Taylor Street up to Beach Street from before 1905.

3.3 MISSION CREEK AND MARKET STREET ZONES

3.3.1 Mission Creek Zone

The location of the study area for the Mission Creek Zone is shown in Figure 3-4a. The Mission Creek Zone is the site of a former tidal creek and neigh-boring salt marsh. The locations of the former water course and marsh are shown in Figure 3-4b as horizontally hatched and dotted areas, respectively.

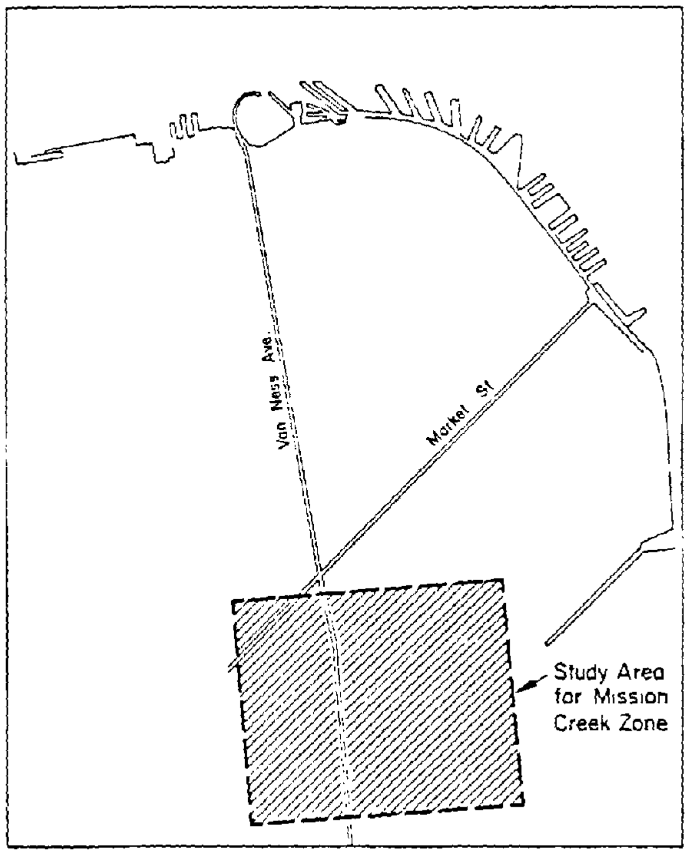


FIGURE 3-4a. Location of Mission Creek Zone

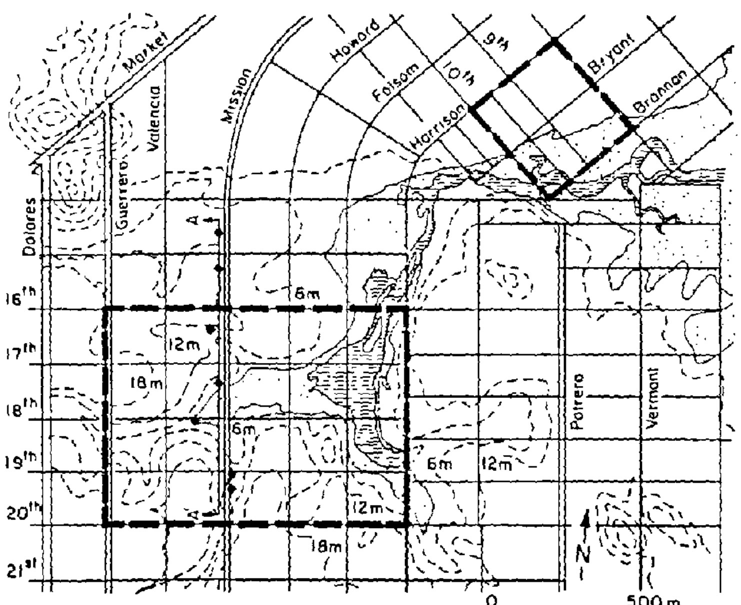


FIGURE 3-4b. Mission Creek Study Area

3.3.1.1 Subsurface Conditions

A geologic cross-section traversing the western section of the filled zone [Youd and Hoose, 1978] is shown in Figure 3-5, and is located by the line A-A' in Figure 3-4b. Figure 3-5 shows the fill to be a generally very loose fine sand. Underlying the fill is a layer of alternating soft clayey sands and soft silty sands (bay mud). Beneath these materials are alternating layers of firm sandy clay and dense clayey sand. Standard penetration resistance (SPT) values [ASTM, 1984] for the fill vary between 2 and 14, with an average value of 5.

The cross-section shows the water table at a depth of 1.5 to 5 m between 17th and 18th Streets, where large horizontal and vertical ground movements were reported after the 1906 earthquake [Youd and Hoose, 1978]. The average slope for this district is relatively flat; for example, between 19th and Guerrero Streets and the freeway near 14th Street, the slope is on the order of 0.6 percent (0.3 degrees) [Youd and Hoose, 1978].

3.3.1.2 Ground Displacements

Figure 3-6 is a map of ground movements which occurred during the 1906 earthquake superimposed on the original topography in the Mission Creek Zone. There are two distinct regions in this zone where large displacements were observed, Zones A and B, as shown in the figure.

There is a close relationship between the original topography and the direction and distribution of soil displacements, as seen in Figure 3-6c. Even relatively small topographical features, such as the ravine underlying 19th Street between Guerrero and Valencia Streets, influenced ground deformation. At this location, soil movements were canalized by the course of the buried creek. The direction of lateral spreading changed through 90 degrees, from a northerly direction on 19th Street to an easterly direction near Valencia Street.

The most severe distortions occurred in areas where the former ravines narrowed, thereby restricting movements to a limited zone. In the region formerly known as "The Willows" [Hall, 1906], along Valencia Street between 18th and 19th Streets, some of the most extreme disturbances occurred. The

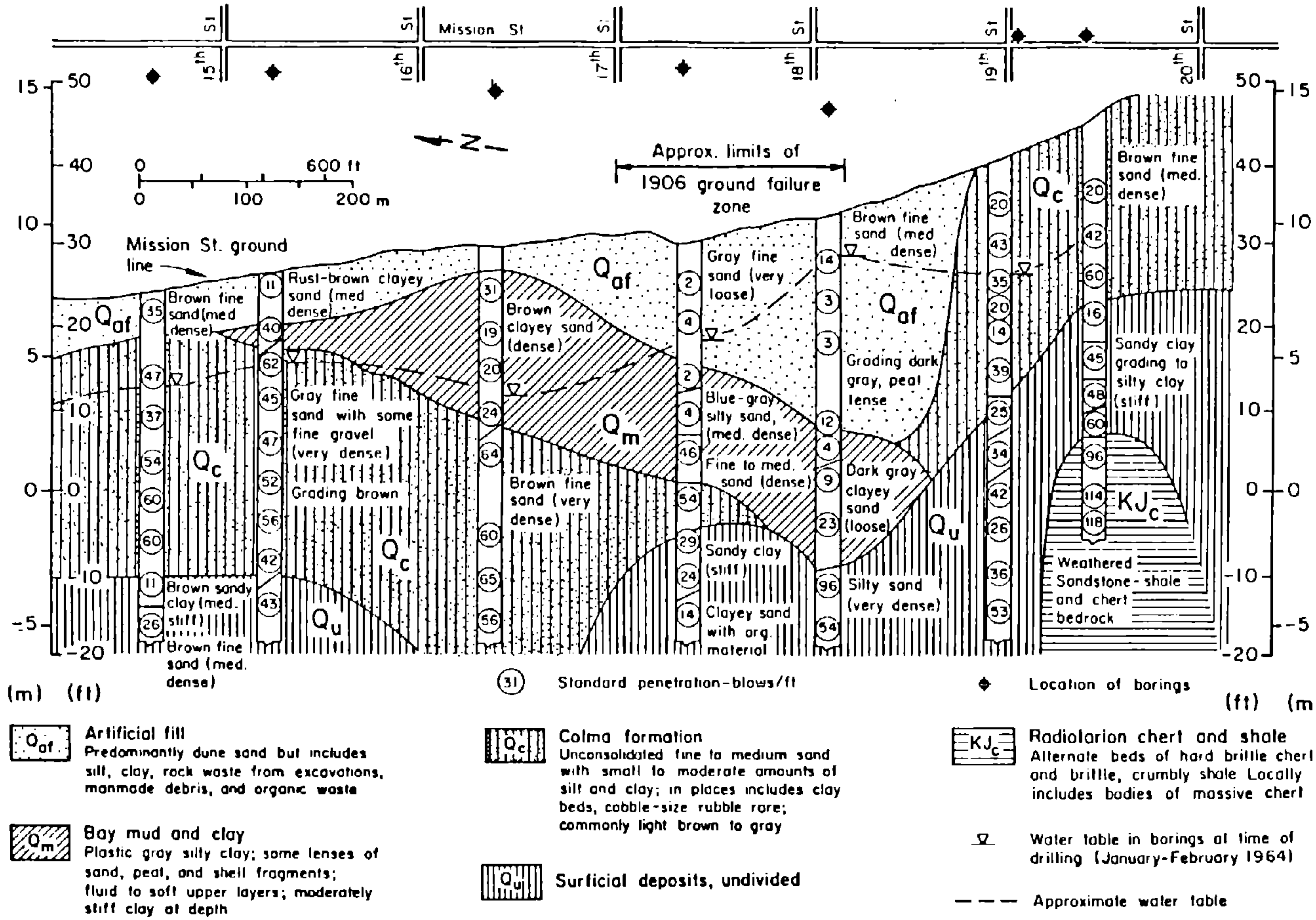


FIGURE 3-5. Geologic Cross-Section in Mission Creek Zone [after Youd and Hoose, 1978]

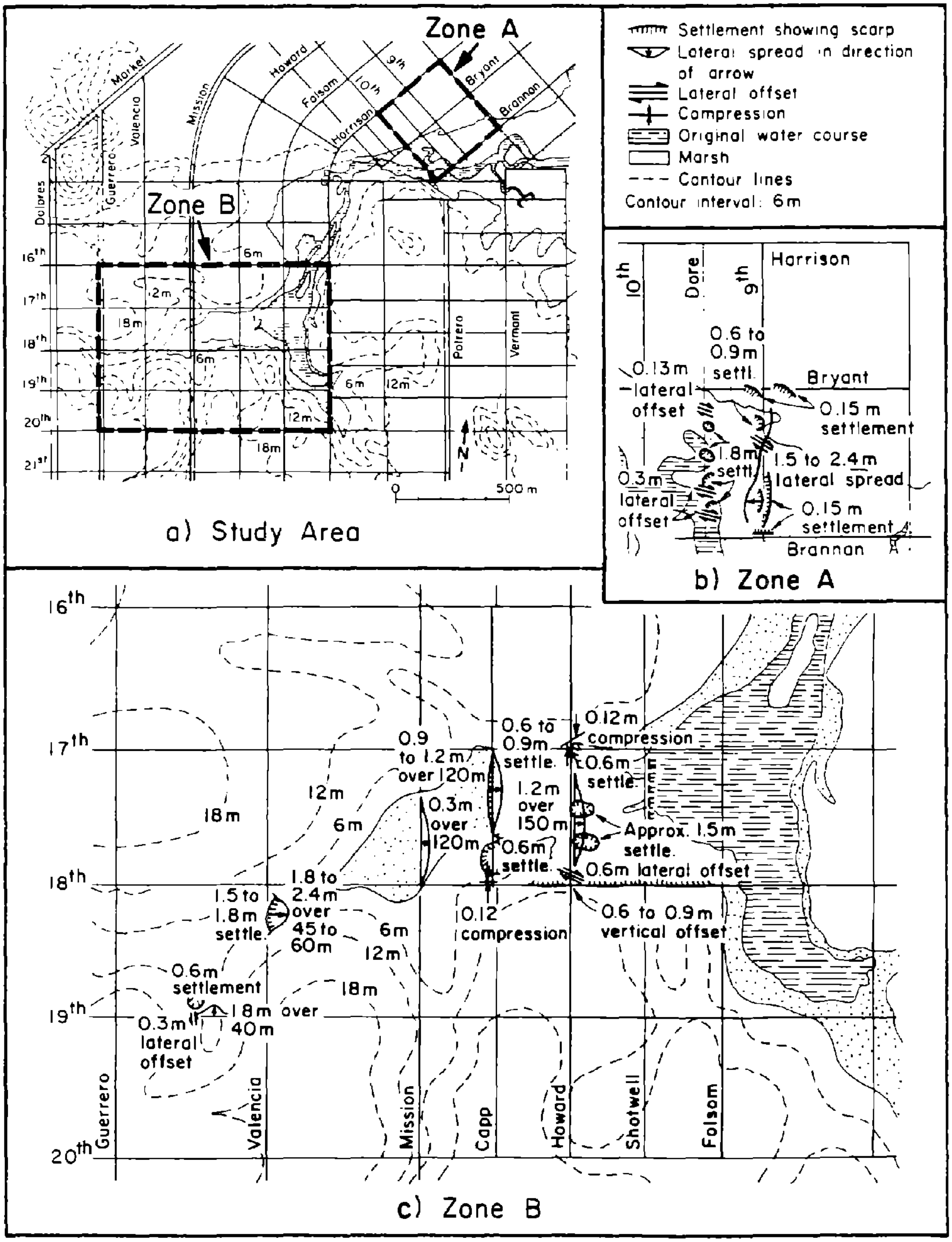


FIGURE 3-6. Earthquake-Induced Ground Movements in Mission Creek Zone

ground spread eastward down the center of the former channel of the stream, with maximum displacements of 1.8 to 2.4 m over a distance of 45 to 60 m. The displacements were largest directly in front of the former Valencia Hotel, which collapsed in response to the movements. The street sank a maximum of 1.5 m in front of the hotel. The combined horizontal and vertical movements were responsible for the destruction of two water mains, a brick sewer, gas mains, electric and telephone conduits, and cable car tracks.

On Howard Street, toward the center of the block, examples of bearing capacity ground failure can be seen in photographs of wood frame houses which were left tilting severely off vertical. The street and the adjacent land in front of the houses settled considerably, up to as much as 1.5 m in areas. Immediately after the earthquake, only about one-third of all the buildings within a four-block area, between 17th and 18th Streets from Folsom to Valencia Streets, were left in a vertical position [Lawson, et al., 1908].

Two 0.12-m compressions were manifested in the buckling of a granite curb on Capp Street and the arching of cable car tracks on Howard Street. These locations of compression are inconsistent with the general pattern of lateral spreading and subsidence in the area. As soil subsides and spreads laterally, it should result in a net tension at the boundaries of the displacement zone. Tensile fractures can be traced along 18th Street, between Folsom and Capp Streets, where slumping of 0.6 to 0.9 m left pavement blocks pulled apart laterally.

One explanation for the strong compressional features is that they are remnants of dynamic compression strains, accentuated because of the liquefied nature of the soil and the abrupt boundary conditions. This type of incoherent motion has been described by Youd [1984] as the result of ground oscillations within zones of soil liquefaction

The eastern limit of the horizontal ground movements was Folsom Street, the edge of the former estuary. Although street settlement was reported along Folsom and Harrison Streets between 18th and 13th Streets [Schussler, 1906], no photograph of ground movement in this vicinity could be found.

The other zone of large ground distortions, Zone A in Figure 3-6b, lies in a two-block area near the mouth of the former Mission Creek, bounded by Brannan, Bryant, 9th, and 10th Streets. In this section, the course of the former creek narrowed as it wound around Potrero Hill immediately to its south, and then widened towards its outlet into Mission Bay. Much of Dore Street towards the northern end of the block overlies a former tributary to Mission Creek, and the southern portion of Dore Street near Brannan Street is directly over a bend in the creek itself. Because of this region's close proximity to the former sand dunes, it is probable that the fill here is mostly dune sand. The liquefaction of this material would explain the large ground movements which occurred in the two-block region.

Large wave-like deformations were reported and photographed along Dore Street. Although these deformations had the appearance of wave forms, they were most likely a result of subsidence. A close inspection of several photographs shows that these "waves" can be explained as a succession of local settlement depressions.

The lateral displacement along Dore Street was eastward, except for a prominent westward offset near Bryant Street. This offset was part of the general southwestern slippage toward the former creek channel from Bryant Street. Along 9th Street, slumping and lateral spreading were westward towards the former channel. A general pattern of slumping can be detected in Figure 3-6b towards the center of the block bounded by 9th, Dore, Bryant, and Brannan Streets.

3.3.2 South of Market Zone

The South of Market Zone is the site of the old Sullivan Marsh, a tidal marsh which was once contiguous with two small tidal streams. Figure 3-7, which locates the South of Market Zone in the downtown area of San Francisco, also shows the outline of the former marsh and of the original shore line of Mission Bay to the south of Brannan Street. The ground displacements of the South of Market Zone all fall within the boundary of this former salt marsh. This area was filled during the years between 1850 and

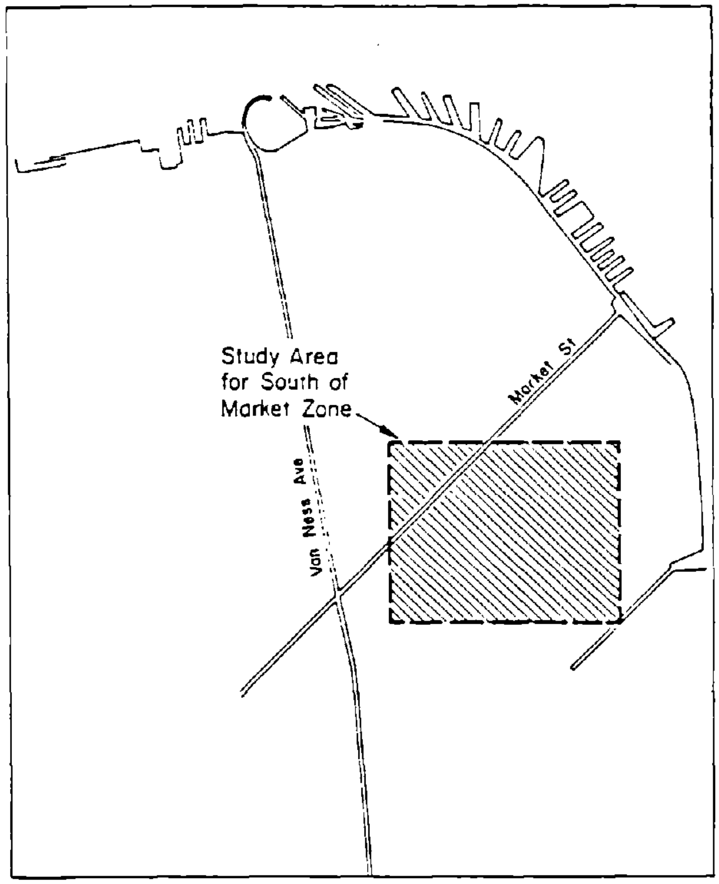


FIGURE 3-7a. Location of South of Market Zone

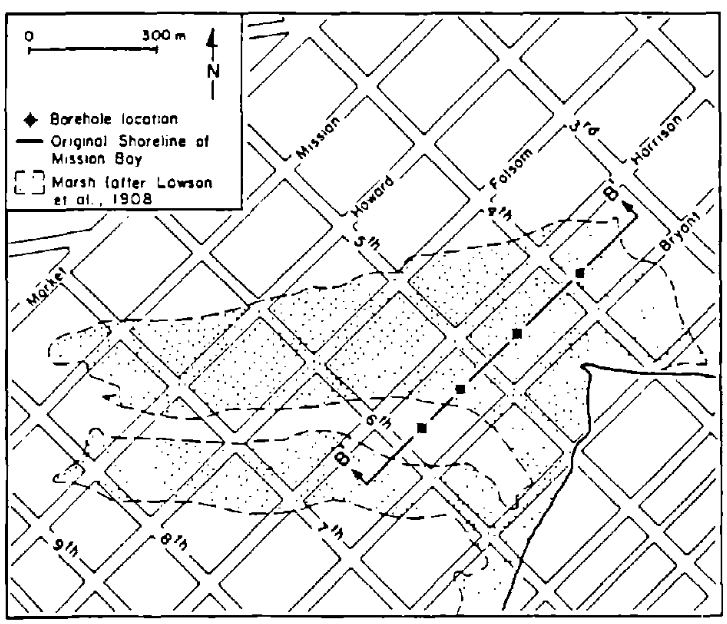


FIGURE 3-7b. South of Market Study Area

1860, predominantly with material excavated from the nearby sand dunes [Roth and Kavazanjian, 1984].

3.3.2.1 Subsurface Conditions

A geologic cross-section [Youd and Hoose, 1978], shown in Figure 3-8, is located in Figure 3-7b by the line B-B'. This figure shows the artificial fill to be composed of an upper layer, 1.5 to 2.4-m thick, of a loose rubbley gravelly silty sand and sandy gravel, underlain by a 2.1 to 3.4-m-thick layer of loose fine sand. Beneath the fill is a deep deposit of soft peaty clay. Standard penetration resistance values of the fill range from 4 to 12, indicating the fill to be a generally loose deposit. The groundwater table within the filled zone was within 1 to 3 m from the surface.

Roth and Kavazanjian [1984] characterized the fills in this zone as being roughly 4.6 to 6.1 m thick. An idealized soil profile of the fill would consist of two layers: a 3.0-m layer of sandy gravel to gravelly sand, underlain by 3.0 to 6.0 m of silty fine sand. Both layers are interspersed with clay seams. The fill appears to contain an increasing amount of gravel and bricks towards the south near China Basin. The rubble sand fill was probably material excavated from the former Steamship Point. The lower layer of fill, that of the dark gray fine sand, was probably derived from excavation of the nearby dune sand. Roth and Kavazanjian [1984] identified as potentially liquefiable layers of silty sand at depths to 3 m, and poorly graded dune sand at depths of 1.5 to 4.6 m.

Although this region had substantial deformations, the average slope is very flat. Along the central axis of the region, from near 8th and Mission Streets to 4th and Brannan Streets, the slope is only 0.8 percent (0.5 degrees) [Youd and Hoose, 1978].

3.3.2.2 Ground Displacements

Figure 3-9 shows the ground movements relative to the shore line of the former marsh in the South of Market Zone. The general direction of flow in this zone was west to east, with maximum horizontal displacements of 0.9 to 1.8 m near the center of the channel. Wave forms, with their crests parallel to the direction of flow, were common. The amplitude and length of the

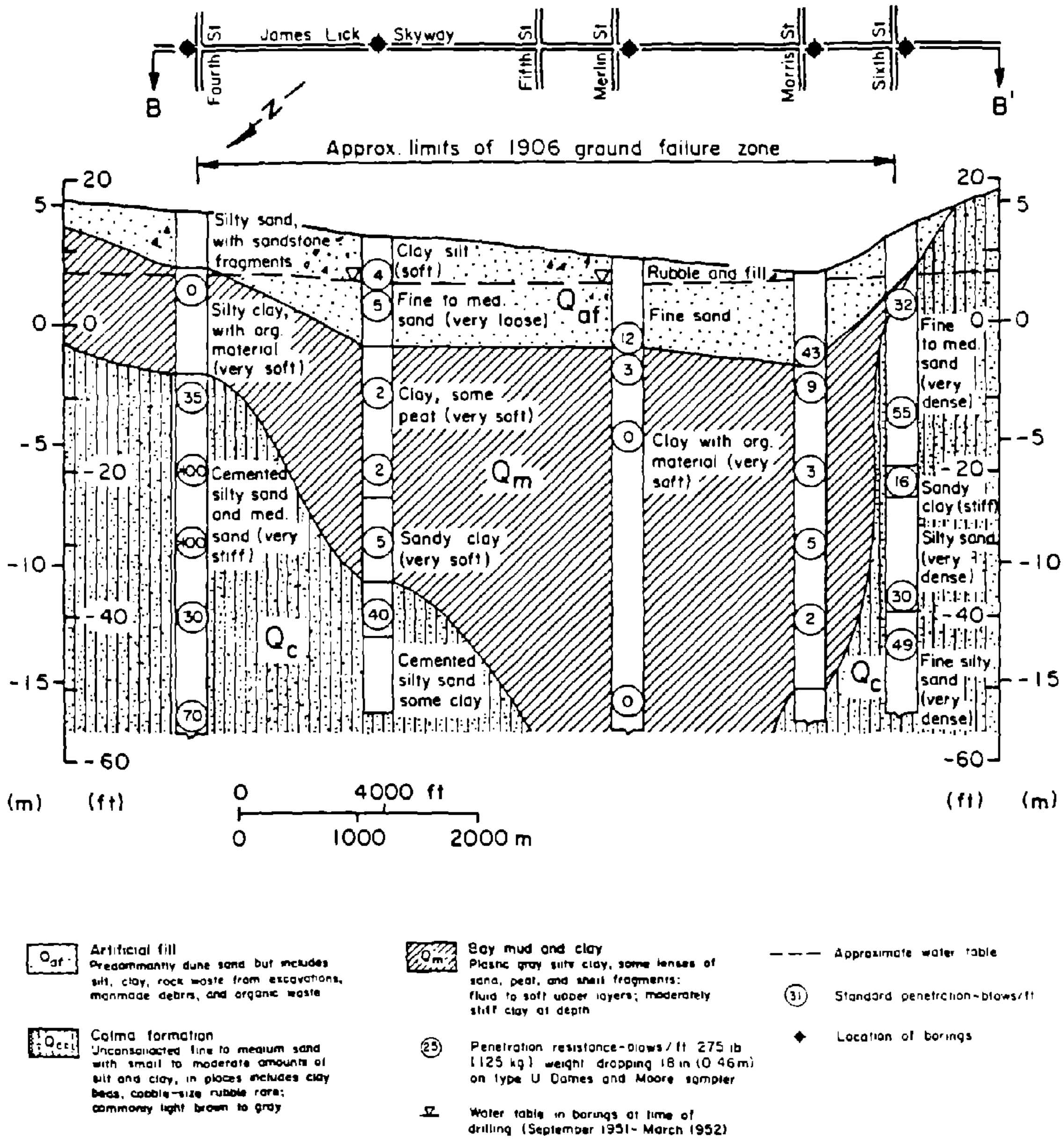


FIGURE 3-8. Geologic Cross-Section in South of Market Zone [after Youd and Hoose, 1978]

waves were indefinite and irregular [Lawson, et al., 1908].

The northern limit of ground movements in this zone was Mission Street at

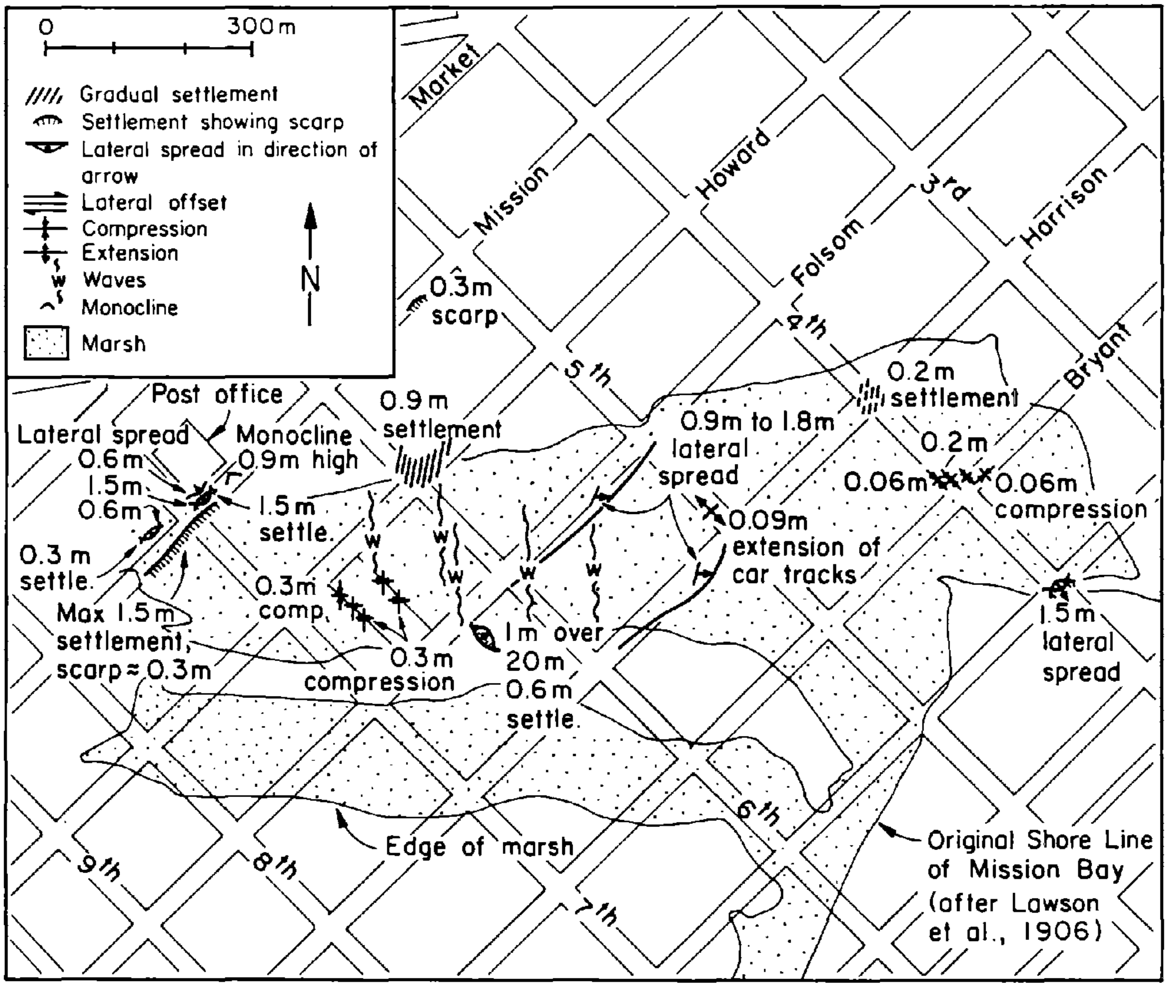


FIGURE 3-9. Earthquake-Induced Ground Movements in South of Market Zone

7th Street near the head of the former marsh. The post office, the southern corner of which was founded on filled ground, settled 0.6 m at the building line and 1.5 m at the curb. The ground displaced laterally 1.5 m to the southeast at the building line. A 0.9-m-high monocline in front of the southeast corner of the building bent cable car tracks and pavement blocks into an arch.

Gradual settlements were seen along the northern boundary of the marsh. At the intersection of 6th and Howard Streets, the street settled 0.9 m relative to the sidewalk, which was founded on good material. The corner of 4th and Harrison Streets settled 0.2 m beneath the rails of the cable car tracks.

Near the southeastern limit of the zone, at 4th and Bryant Streets, compressional features were seen. On 4th Street, rail tracks were buckled 0.06 m and 0.2 m in the southeast-northwest direction. On Bryant Street, rail tracks were buckled 0.2 m in the southwest-northeast direction. At this point, the flow movement was presumed to have been restricted by the sandstone outcrop of Rincon Hill, thus causing the compressive effects, and deflecting the flow to a more southerly direction [Lawson, et al., 1908].

Southeast of Brannan Street, in filled areas of the former Mission Bay, the effects of the earthquake were less intense. Lawson, et al., [1908] noted that the material used in filling the bay tended to be a more rubbley material, derived from grading for street construction in the neighboring rocky hills. This material would be less likely to liquefy. It should be recognized that this region was sparsely developed at the time of the earthquake, and so observation of structural damage or ground deformations would have been limited.

3.3.3 Foot of Market Zone

Figure 3-10, which locates the Foot of Market Zone in the downtown area of San Francisco, also shows the original high-water shore line of the former Yerba Buena Cove. Development of this area began in the 1850's and continued until 1900 [Roth and Kavazanjian, 1984].

3.3.3.1 Subsurface Conditions

A geologic cross-section along Market Street [Youd and Hoose, 1978], located by the line C-C' in Figure 3-10b, is shown in Figure 3-11. The figure shows the fill to range from a maximum depth of 12.5 m near the water, thinning out at its western extent at Sansome Street, with an average depth of 6 m. The artificial fill is primarily composed of loose fine sand or silty sand with rubble. A deep deposit of silty clay underlies the fill to the east of

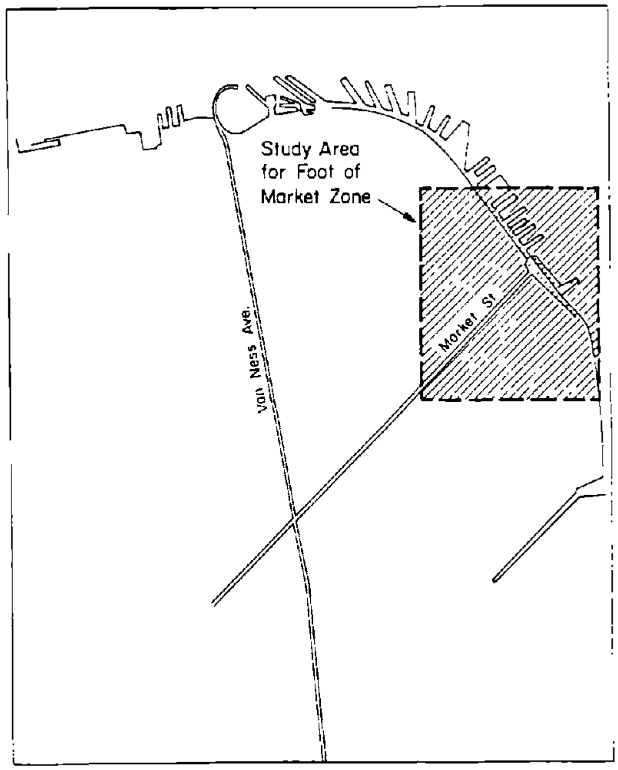


FIGURE 3-10a. Location of Foot of Market Zone

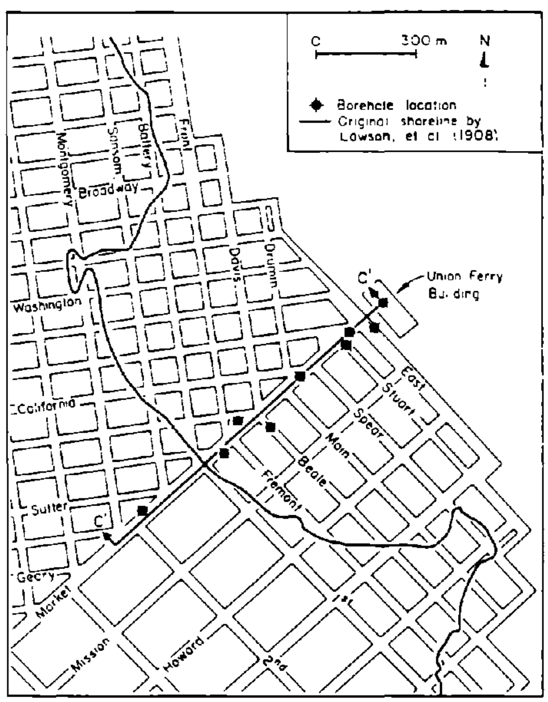


FIGURE 3-10b. Foot of Market Study Area

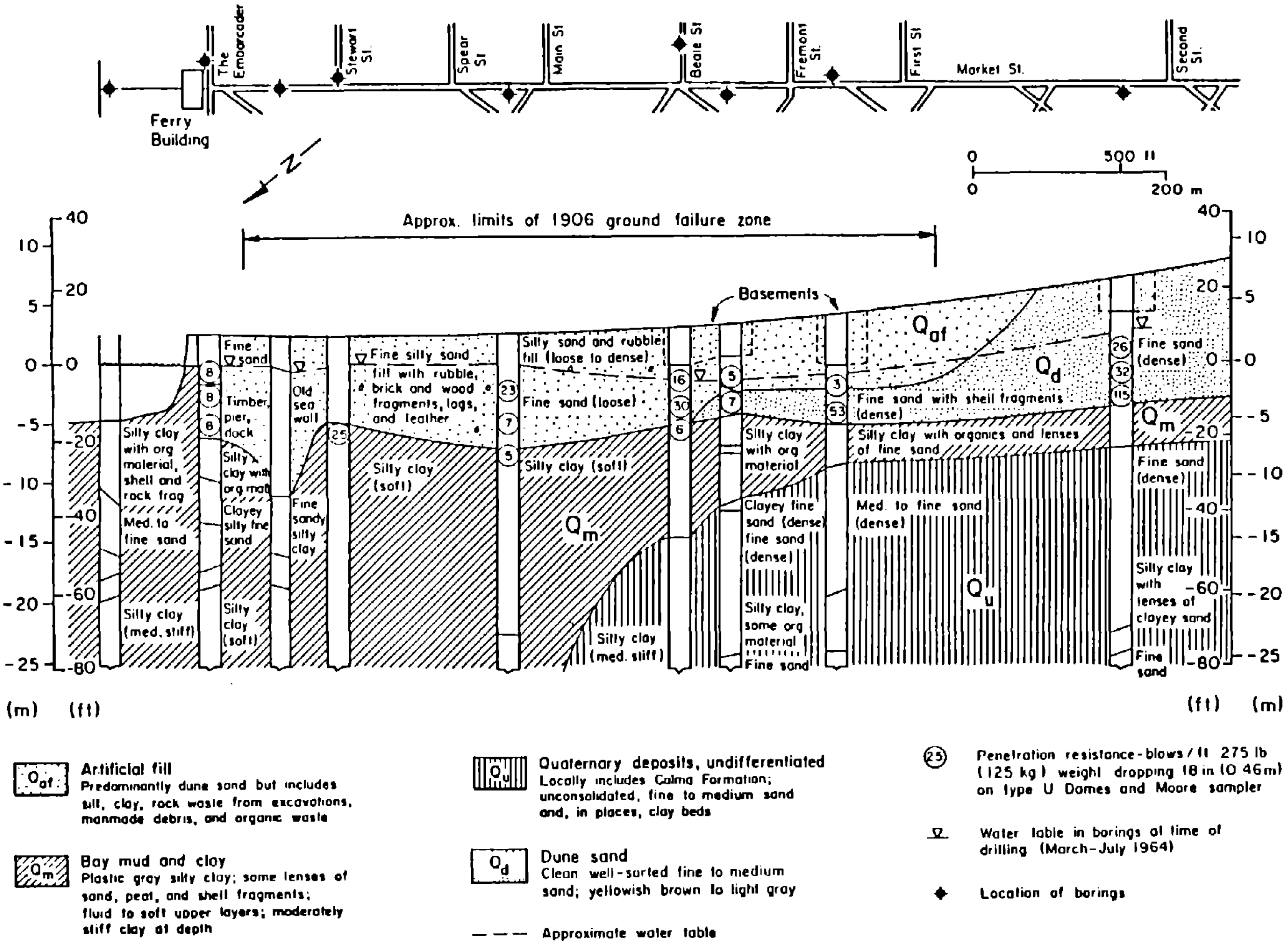


FIGURE 3-11. Geologic Cross-Section in Foot of Market Zone [after Youd and Hoose, 1978]

Beale Street. To the west of Beale Street, a layer of dense clayey fine sand underlies the fill.

The penetration resistance values shown in Figure 3-11 are not based on the Standard Penetration Test. The N values were defined as the number of blows per foot required to drive a Type U Dames and Moore sampler using a 275-lb hammer falling 18 inches (460 mm). Penetration resistance values for the artificial fill range from 3 to 30 [Youd and Hoose, 1978].

Roth and Kavazanjian [1984] idealized the fill profile in this zone as containing two characteristic layers of the fill: 3 m of a brown medium to fine sand layer (SP) underlain by a 3 m silty or clayey sand layer (SM/SC). The variability of methods of performing the SPT test for their borings required that the data be correlated to the standard procedure. Roth and Kavazanjian [1984] describe a method to relate non-standard values such as these to SPT values using a wave-equation analysis. The corrected mean SPT resistance value was found to be 12, with a standard deviation of 9 for the silty sand fill, and 27, with a standard deviation of 22, for the dune sand fill. Both layers of fill were deemed potentially liquefiable.

The groundwater table was observed at depths of 2.4 and 5.5 m near the water front and Sansome Street, respectively [Youd and Hoose, 1978]. The average ground slope along Market Street from Sansome Street to the water front is less than 0.5 percent (0.3 degrees) [Youd and Hoose, 1978].

3.3.3.2 Ground Displacements

Figure 3-12 summarizes the ground displacements in the Foot of Market Zone in relation to the original high-water shore line of the former Yerba Buena Cove. The boundary of the disturbance in the Foot of Market Zone delineated by Hall [1906] falls outside the original high-water shore line. The limit of the fill on Market Street is at the crossing of Sansome Street, but the depth of fill coincides with the groundwater table only in the vicinity of First Street.

Wave forms were again prevalent on most of the streets eastward of this boundary, with the crest of the waves perpendicular to the general direction

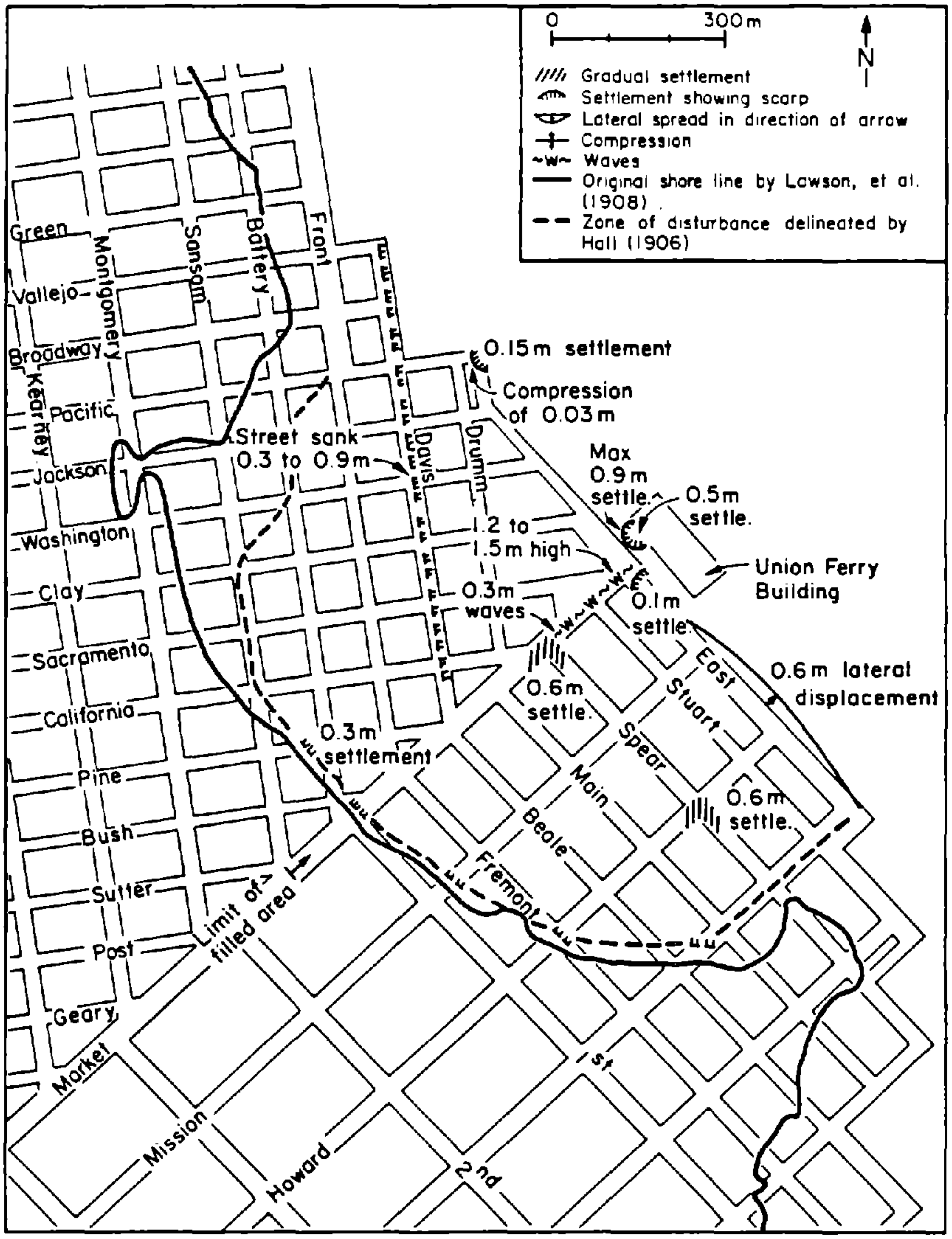


FIGURE 3-12. Earthquake-Induced Ground Movements in Foot of Market Zone

of flow. The height of the waves ranged from 0.15 m to 1.5 m. Lower Market Street is a good example of this phenomenon. The wave troughs on Market Street at Spear Street were about 0.3 m and increased in height to about 1.5 m in front of the Union Ferry Building. Waves up to 1.2 m in height were also common along East Street.

Much of the area also experienced settlements. Along Davis Street between Vallejo and California Streets, subsidence of 0.3 to 0.9 m was observed at every street crossing [Hall, 1906]. Settlements of 0.6 m also occurred along Spear Street at both Howard and Market Streets. The street settled relative to the sidewalk in front of the northwest corners of both the Aetna Building (Spear and Market Streets) and the Folger Building (Spear and Howard Streets). The settlements cited by Hall [1906] on Pine, Market, Mission, Howard, and Folsom Streets were parallel to the outline of the original shore line.

East Street, perhaps, had the most extensive damage in this zone. The street, in general, subsided 0.15 to 0.3 m and shifted eastward 0.15 to 0.6 m. The pavement was shoved up against water front structures. Twenty-five meters of frontage along the Ferry Building subsided from 0.5 m to a maximum of 0.9 m at the northwest corner.

3.4 MARINA AND NORTH POINT DISTRICTS

As shown by Figure 3-13, the Marina and North Point Districts are located along the northern shore and separated by Black Point. Although some filling had begun in these areas prior to 1906, the majority of the development took place after the earthquake and subsequent fire. Figure 3-13 shows the 1908 shore line, which is approximately the same as that of 1906. Zones of MM IX to X intensity are delineated in the figure by dotted areas. The original coast line, as determined from the 1853 coastal survey, coincides well with the inland boundary of the MM IX to X intensity zones along the northern shore, identifying these areas as fill.

3.4.1 Marina District

The Marina District had only a little "made" land along the shore by 1906 [Olmsted, et al., 1977]. This area was formerly a tide-marsh between the shore and a sandbar, known as Strawberry Island, located near what is now Broderick Street. The filled land was at the mouth of the marsh in the proximity of what is now the intersection of Bay and Scott Streets. Since the district was only sparsely developed in 1906, very little damage was

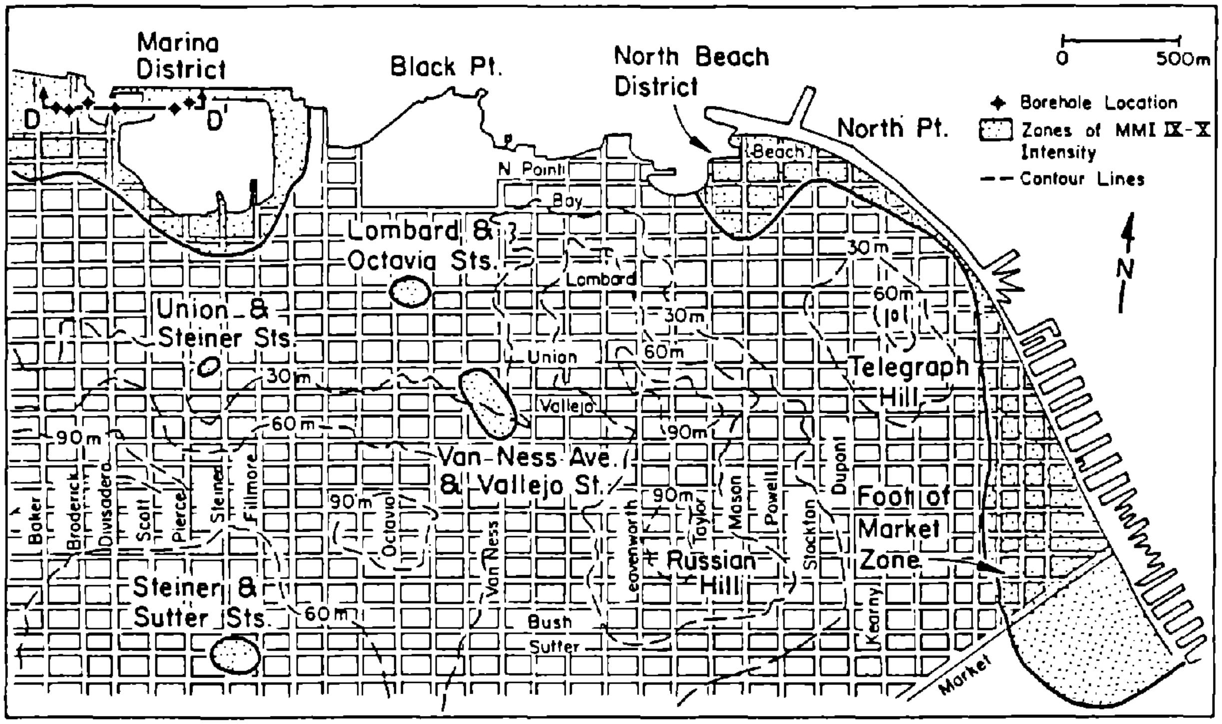


FIGURE 3-13. Location of Marina and North Beach Districts

documented.

3.4.1.1 Subsurface Conditions

Figure 3-14 shows a geologic cross-section [Dames and Moore, 1977] along Marina Boulevard. The cross-section is identified in Figure 3-13 as line D-D'. The fill did not extend as far north as Marina Boulevard, along which the cross-section is located, at the time of the earthquake. The majority of the filling in this zone occurred after the earthquake, between 1912 and 1930. A closer examination of the subsurface conditions is warranted, however, since ground conditions are potentially similar to those of other zones which did experience ground movements during the 1906 earthquake.

Figure 3-14 shows the fill to be, on the average, 10 m thick. At the west end of the section, between Baker and Broderick Streets, the fill layer is approximately 2 m thick and overlies a 7-m layer of medium dense fine sand. Underlying this layer is a soft silty clay. This vicinity was originally

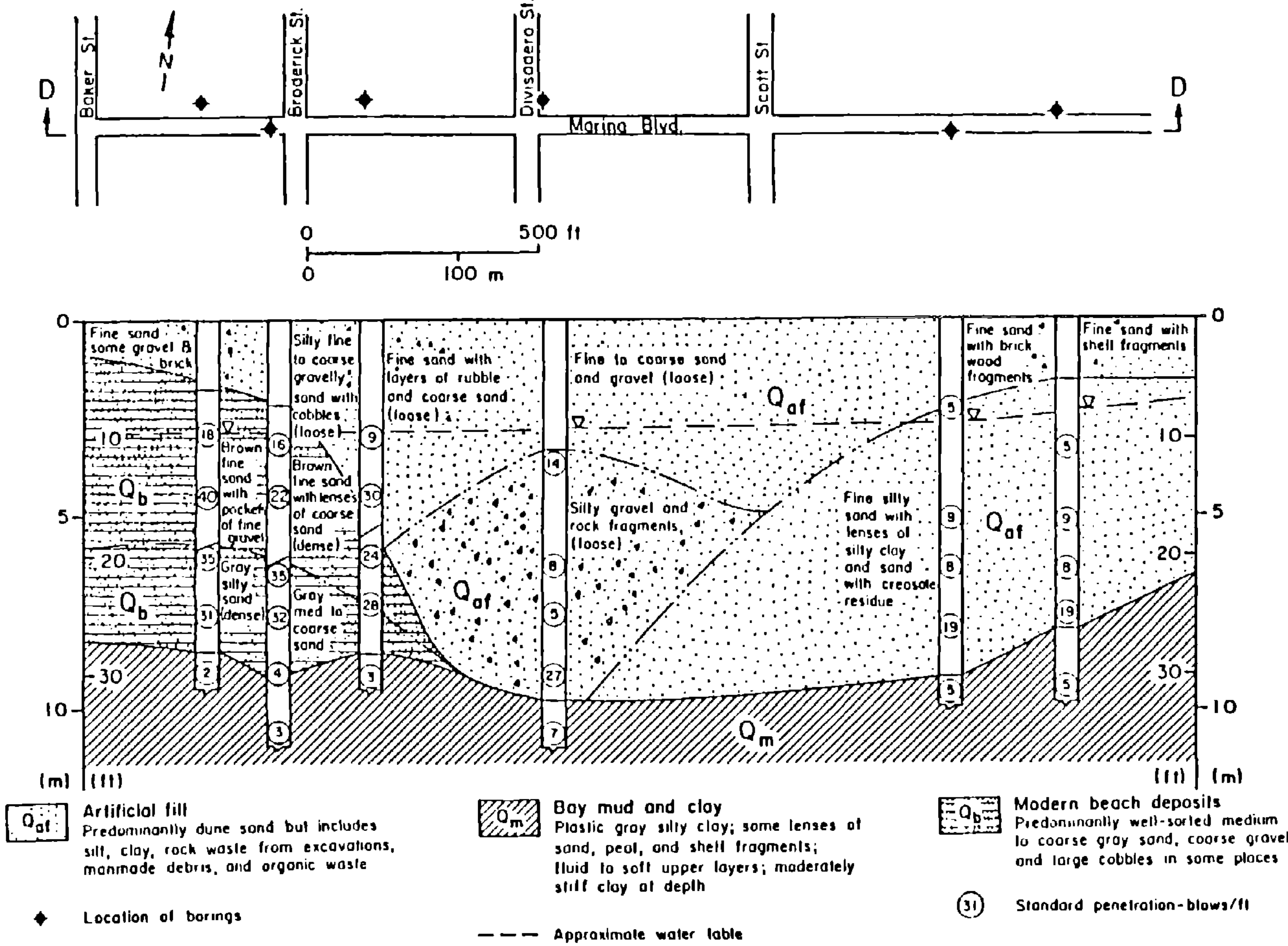


FIGURE 3-14. Geologic Cross-Section in Marina District [after Dames and Moore, 1977]

known as Sand Point on Strawberry Island, and the layer of medium dense sand is assumed to be a modern beach deposit [Schlocker, 1974]. The average SPT resistance value for the modern beach deposit is 26, with a standard deviation of 12. The fill is characterized by a loose fine silty sand with gravels, cobbles, and some debris. Near Divisadero Street, 6 m of silty gravel and rock fill were encountered below 4 m of the fine sand fill. West of Scott Street, two sandy fill layers overlie the bay mud. A 2-m-thick layer of brown fine sandy silt with gravel and debris overlies a 7 to 8-m-thick gray fine sand layer. The lower fill layer contains many pockets of gray highly plastic clay and of sand containing creosote residue. The average SPT value for this layer is 8, with a standard deviation of 6. The groundwater table in the corss-section was encountered at a depth of 2.5 to 3 m.

3.4.1.2 Ground Displacements

Of those areas filled by 1906, two small localities within the Marina District were documented as experiencing extensive damage from the earthquake. In the vicinity of Buchanan and Fillmore Streets, and from Bay Street to the water front, several structures of the San Francisco Gas Light Company were founded on an earthen mole. The filled strip extended over 300 m into the cove from near Webster Street. These buildings suffered great damage from considerable differential settlement [Gilbert, et al., 1907].

A small area at the western edge of the district was greatly damaged by the earthquake. The area, bounded by Lyon, Baker, and Broderick Streets north of North Point Street, is near the former sand point of Strawberry Island. It is underlain by both fill and natural sand deposits. Much of this area had undergone extensive grading by 1906. At this location, several timber frame fuildings were thrown out of vertical and a sewer was broken on Baker Street [Lawson, et al., 1908].

3.4.2 North Point District

3.4.2.1 Subsurface Conditions

Although the filling of the region was started in 1865, the majority of the fill was placed after 1906. The fill is highly variable, including much debris from the earthquake and fire. Roth and Kavazanjian [1984] simplified the fill in this zone as consisting of two layers: 3 m of sandy gravel to gravelly sand, underlain by 3 to 6 m of silty fine sand. Clay seams are interspersed throughout the profile. The fill contains much debris, including brick, wood, and rock fragments. The most likely source for the top layer of fill was located between Jones and Stockton Streets, just south of the North Beach District (see Figure 3-13) [Roth and Kavazanjian, 1984]. This deposit is mapped as an undivided, surficial deposit consisting of the Colma Formation and slope debris. The mean corrected SPT value for this layer is 15, with a standard deviation of 15, although the values are variable. A dune sand deposit originally located at the base of Russian Hill was the most likely source for the lower silty sand fill layer [Roth and Kavazanjian, 1984]. The mean corrected SPT value was 17, with a standard deviation of 12. Both fill layers were identified as potentially liquefiable [Roth and Kavazanjian, 1984]. The groundwater table is at an average depth of 2.4 m in this area [Roth and Kavazanjian, 1984].

3.4.2.2 Ground Displacements

No photographs showing ground movements resulting from the 1906 earthquake could be found for the North Beach District. However, settlements of several streets were shown on a map presented by Schussler [1908]. A three-block region experienced settlements, starting at Bay Street from Taylor to Mason Streets, and then along Mason Street from Bay to North Point Streets, and continuing along North Point Street from Taylor to Powell Streets. Two other streets, Stockton and Dupont (now Grant) Streets, also experienced settlements for less than a block in length just north of North Point Street.

3.5 OTHER ZONES OF HIGH INTENSITY

Several additional zones experienced an appreciably high intensity during the 1906 earthquake. These zones are shown in Figures 3-2b and 3-13. Damage in these zones can be correlated with the presence of artificial fills resulting from street construction or the filling of small ravines.

3.5.1 Duboce Park

A small one-block area near the corner of Waller and Portola Streets showed high intensity. This zone is located on Figure 3-2b. Unlike the other zones described in this section, this zone was not on "made" land. This block occupies the lower slopes of a small narrow valley and is underlain by a thin strata of sand. Effectively, this sand layer shifted downslope, cracking foundation walls and moving houses eastward. Street pavement was also buckled and broken [Lawson, et al., 1908].

3.5.2 Steiner and Sutter Streets

In the neighborhood of Steiner and Sutter Streets, located in Figure 3-13, extensive damage occurred to several structures. A large church on the corner of Bush and Steiner Streets collapsed and several small frame buildings nearby were knocked from their underpinnings. Likewise, on Geary Street above Fillmore Street, two wooden-framed brick buildings were completely destroyed. No permanent ground movements were documented in this district. This area, located in the Upper Hayes Valley, is underlain by a thick layer of dune sand [Schlocker, 1974].

3.5.3 Lombard and Octavia Streets

Another small district, less than a block in extent, on Lombard Street between Gough and Octavia Streets, had a high intensity of damage. This is a filled area, formerly the site of a small fresh water lagoon known as Washerwoman's Lagoon [Lawson, et al., 1908]. The location of this zone is shown in Figure 3-13. Although the earthquake damage in this area was of MM IX intensity, no permanent ground movements were recorded [Lawson, et al., 1908]. Brown, et al. [1932] also mention this area as suffering severely during the earthquake.

3.5.4 Vallejo Street and Van Ness Avenue

The corner of Van Ness Avenue and Vallejo Street experienced considerable ground movements. This is the site of a former deep ravine leading northwestward down to the former Washerwoman's Lagoon. The location of damage can be circumscribed by an ovoid area two blocks long, as shown in Figure 3-13. The fill is roughly 12 m deep at this point [Lawson, et al., 1908].

This whole area settled, the greatest subsidence being roughly 0.6 m at the intersection of Van Ness Avenue and Vallejo Street. Lateral spreading to the north occurred on Van Ness Avenue, with a magnitude of approximately 0.9 m at the intersection with Vallejo Street, decreasing to about 0.3 m at Green Street [Schussler, 1906]. The ground movements caused much damage to the streets, pipelines, and structures. Buildings shifted and were thrown out of vertical, resulting in their foundations being crushed. Sidewalks were thrust over the curbs, and street pavements were fissured and down-faulted [Lawson, et al., 1908]. To the east, south, and west of the intersection of Vallejo Street and Van Ness Avenue, the sewers were broken for a distance of about 45 m. Water mains were also broken. At one location, a 510-mm-diameter pipe was broken into 0.6 m sections over a length of about 13 to 17 m [Gilbert, et al., 1907].

3.5.5 Union and Steiner Streets

Another district of extensive damage, less than a quarter of a block in length, was found on Union Street between Pierce and Steiner Streets. This area is located in Figure 3-13. This area had been filled as a result of grading the streets. Some of the largest ground movements were found at this location. The north sidewalk moved about 3 m to the north, settling about 3 m below original grade. The south sidewalk settled several centimeters and shifted northward about 1 m.

Although the type of ground displacements seen here resemble those seen in other zones, it is important to note that this slide was not caused by liquefaction. The displacements were a result of unconsolidated fill material shifting downhill into a vacant lot to the north.

3.6 PIPELINE BREAKS

San Francisco's water supply system was severely disrupted by permanent ground movements during the 1906 earthquake. In 1906, the water trunk and distribution network was composed principally of cast iron pipes connected by lead caulked bell-and-spigot joints. The system was particularly vulnerable to ground movements because of the brittle nature of the cast iron and the relatively low pullout capacity of the joints. The response of the system to the 1906 earthquake is of significance today, since 85 to 90% of the Auxiliary and Municipal Water Supply Systems in San Francisco still are composed of cast iron mains.

Figure 3-15 presents a map of the location of water main breaks resulting from the 1906 earthquake, as reported by Schussler [1906] and Manson [1908]. The dotted lines in the figure represent the extent of street subsidence as reported by Schussler [1908]. The pipeline breaks and street settlements are overlain on selected former topographical features, such as the original shore line and border of marshes shown as solid lines, and selected elevation contours represented by the dashed lines. These contour lines and shore line bound the extent of fill at the time of the earthquake.

There is a remarkable correlation between the locations of pipeline breaks and the subsurface topography. Approximately 50% of all pipeline breaks south of Market Street fall on or within the zero contour lines which mark the boundaries of the in-filled marshes and former bay. Approximately 80% of all pipeline breaks south of Market Street fall on or within the 12 m contour lines. As shown in the figure, all of the street settlements occurred in filled areas.

If a pipeline network is viewed as a system of linear extensometers, the pipeline breaks can be used to identify zones of ground displacements. Data are acquired according to the binary output of damage or lack of damage. The resulting pattern of breaks provides direct evidence of ground displacement. The settlement of the streets, viewed in conjunction with the pipeline breaks, essentially delineates the zones of permanent ground movements.

Several trends are apparent in studying pipeline breaks in the three major zones of ground displacements. In the Mission Creek Zone, pipeline breaks are concentrated where elevation contours converge or loop outward, signifying buried ravines and narrow stream channels. A large proportion of the breaks lie along the boundary between solid and "made" ground, particularly along the border of the former marshes in Mission Creek and South of Market Zones, and along the shore line of the former Yerba Buena Cove in the Foot of Market Zone. The differential movements are most severe along the margins of the in-filled areas, and therefore pipelines are particularly vulnerable in this zone. As Schussler [1906] describes (p. 14):

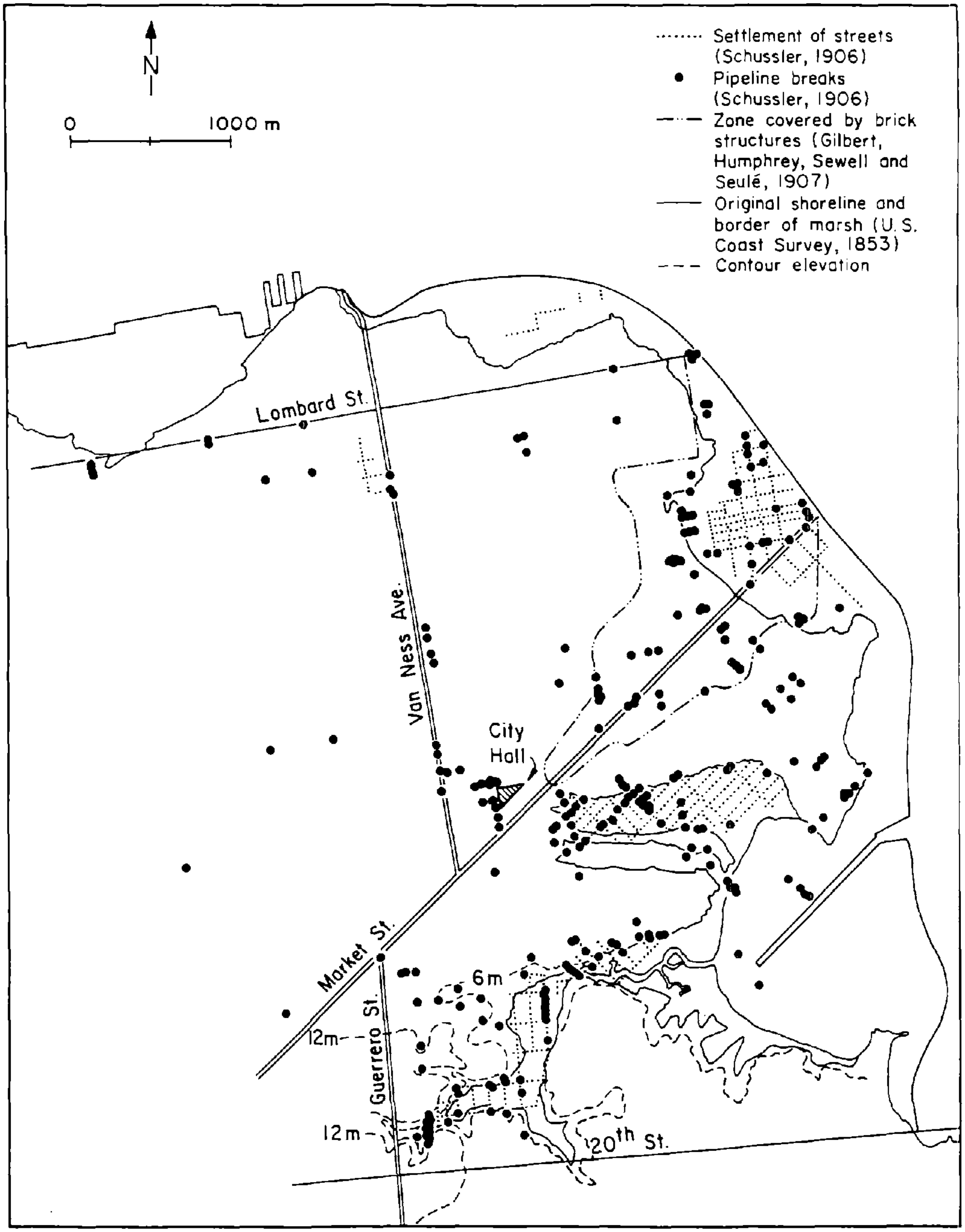


FIGURE 3-15. Pipeline Breaks and Street Settlements in San Francisco After 1906 Earthquake [after Schussler, 1906; Manson, 1906]

> ...in the large majority of cases, principally confined to and caused by the sudden sinking of the streets over the old swamps, which movement (the same as happened to the city's sewers there) tore the pipe over the swamp away from the pipe on terra firma.
>
> In the streets within the main body of the sunken swamp districts, which during the earthquake rapidly moved up and down and sideways, back and forward, like jelly in a shaken bowl, there were also, naturally, a number of breaks in the street pipes, caused by the twisting and rapidly undulating motion of ground in which the pipes had to be laid.

Outside the three principal zones, several breaks can still be attributed to permanent ground movements. For example, at the intersection of Lombard and Octavia Streets, and at Van Ness Avenue and Vallejo Street, pipeline breaks are associated with the smaller zones of MM IX to X intensity.

It should be noted, however, that many of the breaks on solid ground are not associated with permanent ground movement. The damage to pipelines on Market Street between 4th and 5th Streets was a result of blasting when buildings were leveled in an attempt to contain the spread of the fire [Schussler, 1906]. Others are results of the rupture and explosion of gas mains which destroyed sections of streets, along with all underground conduits and pipelines [Hyde, 1906]. Hyde [1906] also postulated that another cause of breaks in gas mains may have been the intense heat of the fire which broke out following the earthquake.

Still other breaks on solid ground may have been caused by the collapse of masonry structures. Water main breaks were concentrated near City Hall. The building, a massive brick structure, was completely wrecked by the earthquake. Photographs [Himmelwright, 1906] show entire sections from the upper part of the heavy brick walls which were shaken down, several intact blocks of which weighed as much as several tons. The falling debris was most likely the cause of many of these breaks. The dashed and dotted line shown in Figure 3-15 outlines the section of downtown San Francisco which contained primarily brick structures. The high number of pipeline breaks in this zone may be associated in part with the collapse of brick structures.

Several pipeline breaks were described or photographed in the literature and are cataloged in Table A-II of the Appendix. In addition to the numerous breaks to the water supply system, street subsidence also crushed many of the brick sewers, such as on 14th and Howard, 17th and Howard, and on Valencia Street.

3.7 SUMMARY OF GROUND MOVEMENTS

In comparing the original topography with maps of soil movements (see Figures 3-6, 3-9, and 3-12), several observations can be made. The ground movement patterns show a close relationship with the original topography, with the direction and magnitude of lateral spreading controlled in large measure by morphological details. As an example of the influence of underlying topography, at 19th and Guerrero Streets, the flow direction changed through 90 degrees as movements were canalized by the course of a buried ravine. The greatest displacements occurred in areas where the contour lines of the original topography converged, indicating a narrowing of the valley or ravine. In these areas, movements were restricted to a relatively narrow zone.

A second important observation involves the close correspondence between locations of lateral spreading and subsidence. Areas of large settlement, some of which resemble sink-holes, developed at the same locations as large lateral displacements. This implies a relatively complex mechanism of deformation, in which lateral movements follow a downslope course at the same time as volumetric loss in the underlying soil results in caving-type distortion, with prominent surface depressions. This complex relationship between lateral and vertical movement is illustrated in the Mission Creek Zone (see Figure 3-6), in which volumetric losses in the soil between Dore and 19th Streets resulted in a large subsidence feature superimposed on a general southeastern trend of lateral spreading.

Compressive deformations were observed in all areas of lateral spreading and subsidence. Several of these compressive features are unusual in that the corresponding mechanism of permanent ground displacement at these locations should have resulted in a net tension. The compressional features are interpreted as evidence of dynamic deformations which occurred as a result of ground oscillations within the zones of soil liquefaction.

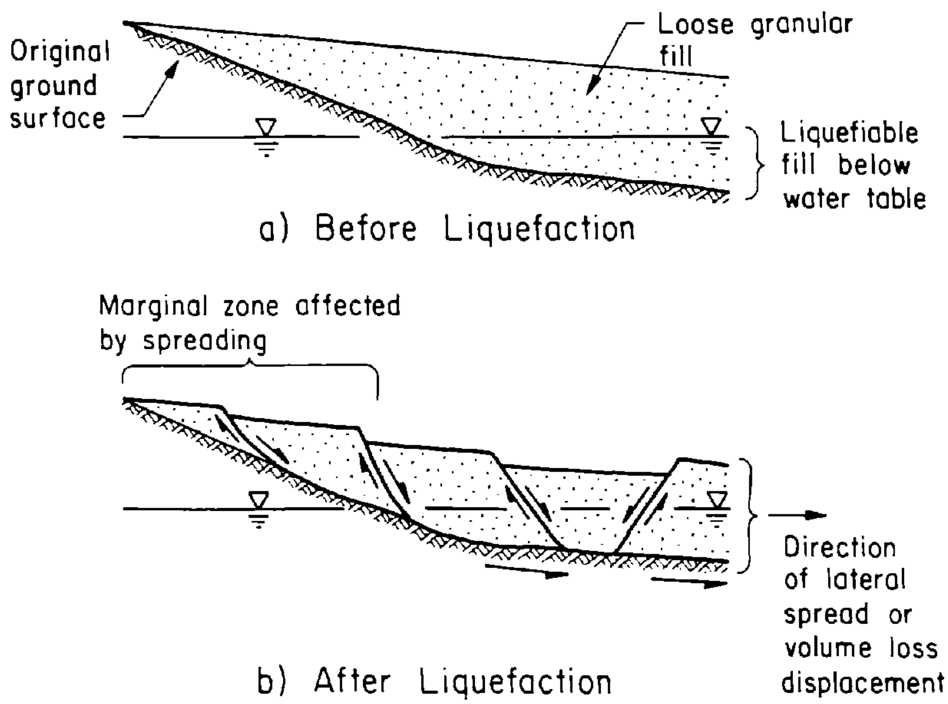


Figure 3-16. Idealized Section Through Lateral Spread

The distribution of pipeline breaks along and slightly removed from the margins of the previous marsh areas is also noteworthy and deserves further explanation. Figure 3-16 shows a cross-sectional view of an area filled with loose granular soil. As liquefaction leads to lateral spreading and consolidation of the underlying sediments, there is a tendency for soil displacements to converge toward the center of the filled area. This lateral extension of the soil promotes subsidence scarps and graben-type features that are consistent with the occurrence of surface depressions. The movement relieves the horizontal restraint against the margins, causing slumping of the fill, even at fill locations above the water table. The slumping is abetted by seismic shaking.

This mechanism of marginal slumping effectively extends the zone of influence of soil liquefaction. It leads to a pattern of displacement and associated pipeline damage which is removed from the exact location of the former marsh and bay areas. As a consequence, pipeline breaks occur along elevation contours higher than the original water levels.

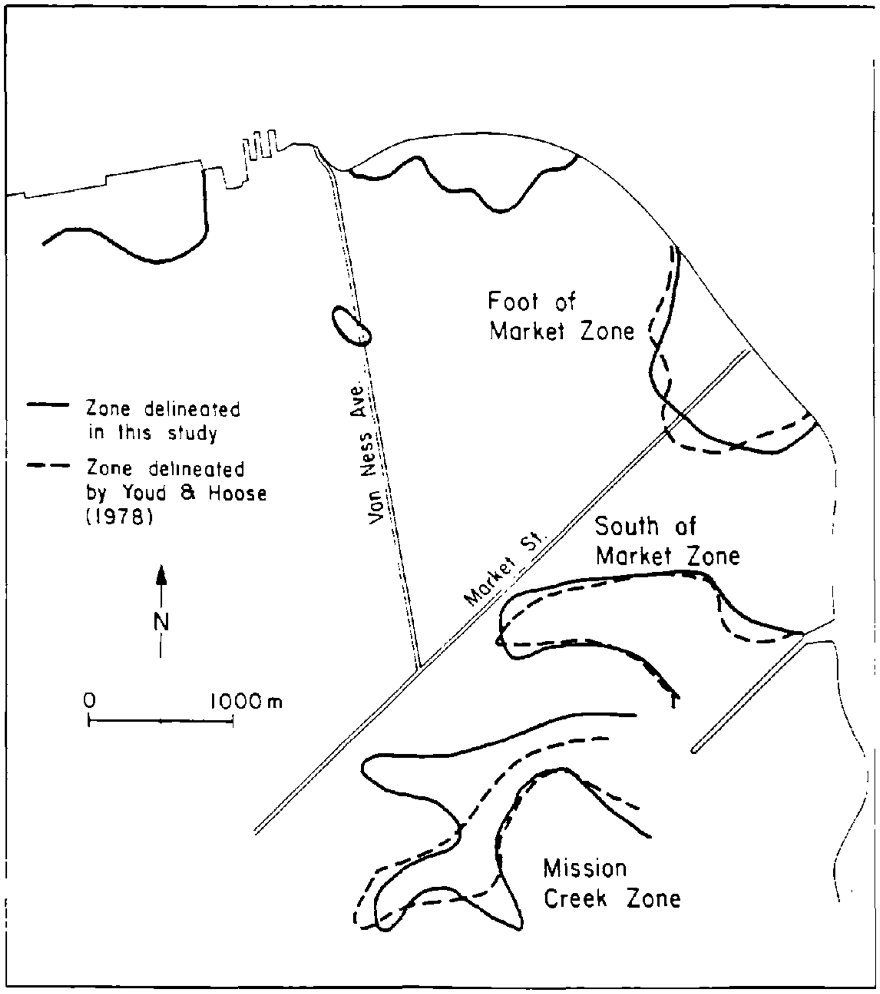


FIGURE 3-17. Zones of Potentially Large Ground Displacements in San Francisco Resulting from Soil Liquefaction

By combining the locations of pipeline breaks, patterns of ground displacement, and previous topographical features, it was possible to develop a system for mapping zones of liquefaction hazards in San Francisco. Figure 3-17 shows the zones of potentially large ground movements, mapped as a result of this study, in relation to the zones of ground failure delineated by Youd and Hoose [1978]. There is a close agreement between the two studies for the Foot of Market and South of Market Zones. Additional information about the previous topography and locations of pipeline breaks has permitted refinements in the delineation of the Mission Creek Zone. The zone mapped in this study is more extensive than indicated by previous investigations.

3.8 CONCLUSIONS

The results of this study have several important implications for lifeline earthquake engineering. A careful study and mapping of previous terrain and morphological features can be used to identify locations of pipeline deformation and even define the pattern and approximate magnitude of potential distortion. For pipeline systems composed of brittle components, such as cast iron mains, the delineation of liquefaction hazards will point out areas of severe breakage which may be isolated with special valves or supplemented by redundant lines sited to avoid the hazardous areas. For ductile trunk and transmission lines, such as large-diameter welded steel water or gas mains, the magnitude and distribution of soil displacements plotted in this section provide a basis of evaluating soil-structure interaction.

Even though relatively large dynamic distortions can be inferred for areas of liquefaction, the evidence in this study shows that the most severe deformations were associated with permanent ground movements from lateral spreading and subsidence. Accordingly, pipeline design based on the maximum permanent deformation should suffice to cover worst case conditions.

SECTION 4

PARAMETRIC STUDY - SOIL/PIPELINE INTERACTION MODEL

4.1 INTRODUCTION

Pipelines usually are constrained to follow rights-of-way or otherwise conform to existing properties, so that poor ground conditions and potentially unstable sites cannot always be avoided. These constraints can be important for large transmission pipelines which cross broad geographical areas. Accordingly, the design of a pipeline in seismic zones may need to account for large soil displacements, especially if the pipeline is located near or within an area of potential soil liquefaction.

Lateral spreads are among the most troublesome liquefaction hazards for buried pipelines. Because the ground movement pattern caused by lateral spreading can be very difficult to predict, computer analyses can play an important role in testing the sensitivity of the pipeline response to various geometric and material variables associated with lateral spreading. Moreover, general patterns of pipeline response to large horizontal soil displacements can be identified through computer simulations, thereby providing a basis for design.

In the preceding sections, case histories have been reviewed in which ground movement measurements and observations of pipeline performance were summarized. Special attention was given to the 1906 San Francisco earthquake, during which damage was concentrated primarily in cast iron pipelines. Modern pipelines are often made of ductile materials, such as steel, which have a much larger capacity for deformation. Steel pipelines may offer substantial advantages in resisting liquefaction-induced movement. A steel pipeline can provide a high strength link through zones of potentially large movement so that vital services can be available even under conditions which would severely damage other pipes.

This section focuses on the response of ductile pipelines to lateral spreading by considering the performance of a 610-mm-diameter continuous girth welded pipeline composed of X-60 grade steel. The performance is evaluated in relation to ground displacement patterns similar to those of the 1906 earthquake. The sensitivity of the pipeline is investigated as a function of various lateral spread characteristics, such as maximum displacement, distance over which the maximum displacement develops, and soil properties of the displaced mass. The first part of this section discusses the soil/pipeline interaction model, and is followed by a description of the computer model analysis. The last section describes the parametric study and summarizes the analytical results.

4.2 SOIL/PIPELINE INTERACTION MODEL

4.2.1 Orientation of Pipeline

The types and magnitudes of pipeline strain depend in large measure on the orientation of the pipeline within the zone of ground movement. Figure 4-1 illustrates how pipeline deformation is affected by orientation with respect to a lateral spread. In Figure 4-1a, the pipeline crosses the lateral spread perpendicular to the general direction of soil movement. In this orientation, the pipeline is subject to bending strains and extension. As shown in Figure 4-1b, the pipeline will undergo bending and either tension or compression at the margins of the slide when the crossing occurs at an oblique angle to the general direction of soil movement. The deformation is analogous to that at an oblique fault crossing, which has been discussed extensively in the literature [e.g., O'Rourke and Trautmann, 1980; Committee on Gas and Liquid Fuel Lifelines, 1984]. Figure 4-1c shows a pipeline oriented parallel to the general direction of soil displacement. At the head of the lateral spread, the displacements resemble normal faulting; under these conditions, the pipeline will be subjected to both bending and tensile strains. At the toe of the slide, the displaced soil pushes against the material of the more stable zone, producing compressive strains in the pipeline.

Steel pipelines have inherent ductility in tension and can accommodate tensile strains on the order of 2 to 5% [Committee on Gas and Liquid Fuel Lifelines, 1984]. A pipeline cannot accommodate similar strains in compression. Compressive strains on the order of 0.3 to 1.0% can lead to local wrinkling and/or buckling of the pipeline, after which further geometric distortion is concentrated at a single location [Committee on Gas and Liquid Fuel Lifelines, 1984].

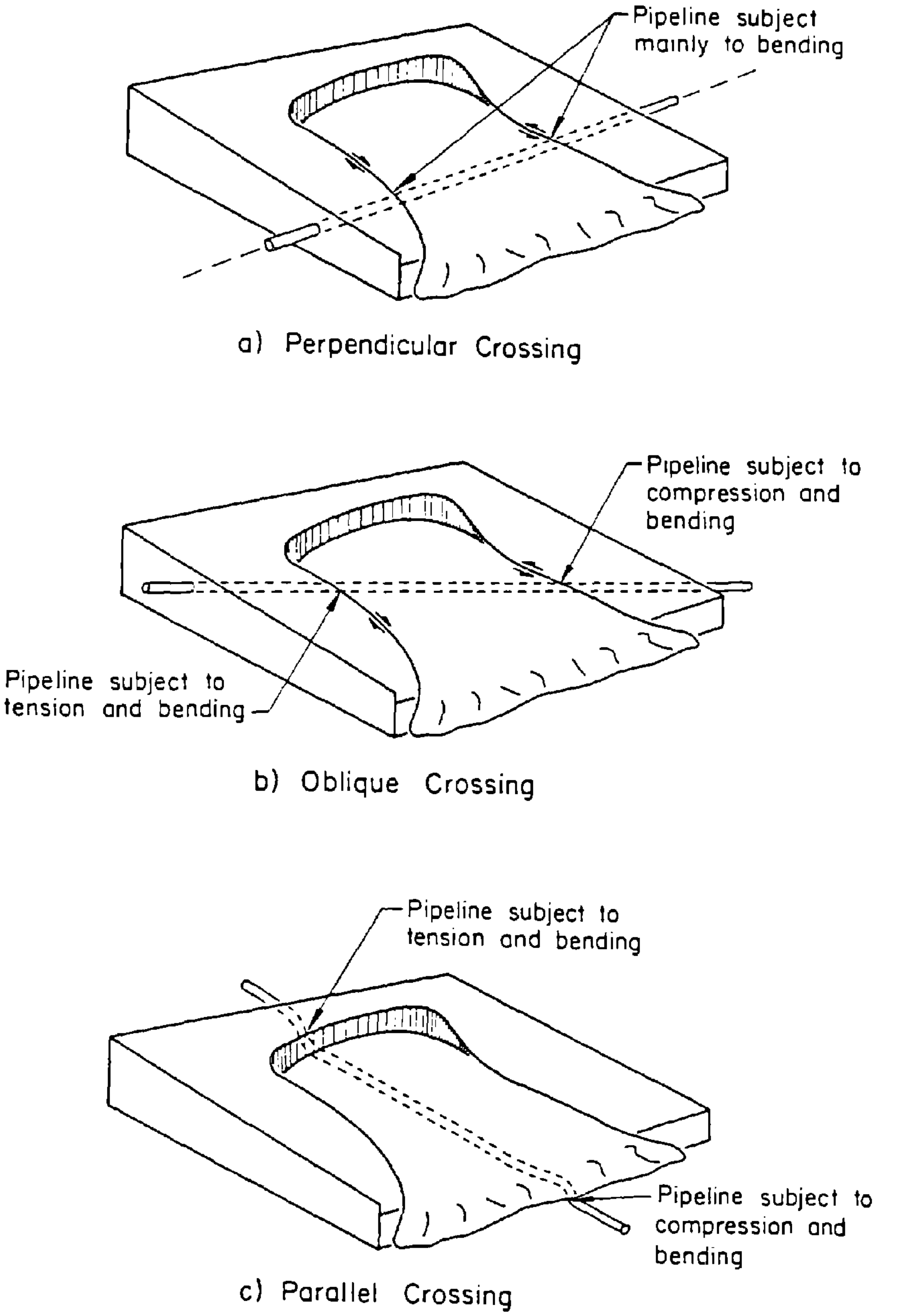


FIGURE 4-1. Pipeline Orientations and Associated Deformations within Lateral Spreads

For a pipeline which cannot be rerouted to avoid potential lateral spreading, Figure 4-1a shows the optimal orientation for the crossing. With this configuration, the soil displacements will subject the pipeline to bending and tension, and will minimize the potential for axial compression. Moreover, there is an advantage in terms of construction because this orientation will often follow a contour of equal elevation, thereby minimizing the amount of cut and fill needed.

4.2.2 Pattern of Soil Displacement

The pattern of soil displacement associated with lateral spreading is controlled by geologic and topographic characteristics. Underlying structures, such as buried ravines, ancient stream channels, or irregularities in the surface of shallow bedrock, can have a significant influence on the distribution of soil movements. Because there are few detailed measurements of differential movements caused by lateral spreads and landslides, there is little factual basis for selecting a model for distributed soil displacements. Tawfik and O'Rourke [1986] pointed out that a suitable model should be able to represent symmetric and skewed distributions. In this way, it is possible to analyze pipeline response to a large range of movement patterns that better reflect the various geologic and topographic characteristics affecting movements.

Tawfik and O'Rourke [1986] used the modified beta distribution [Abramowitz and Stegun, 1972] to describe soil displacement patterns. This model is versatile enough to represent the distribution, or profile, of ground displacements caused by lateral spreads and landslides. The formulation is given by:

$$u = M(s/s_m)^{r-1} [(1-s)/(1-s_m)]^{t-r-1};\ 0 < s < 1 \qquad (4\text{-}1)$$

in which:

- u is the component of the displacement vector (transverse, horizontal, or vertical),
- s is the normalized distance measured across the slide,
- s_m is the locations of the mode (peak value) of the distribution pattern,

M   is the maximum displacement, and

r and t   are parameters controlling the shape of the curve.

The normalized distance, s, is calculated by dividing the actual distance, s', measured from one end of the slide across its width, by the width of the slide, B.

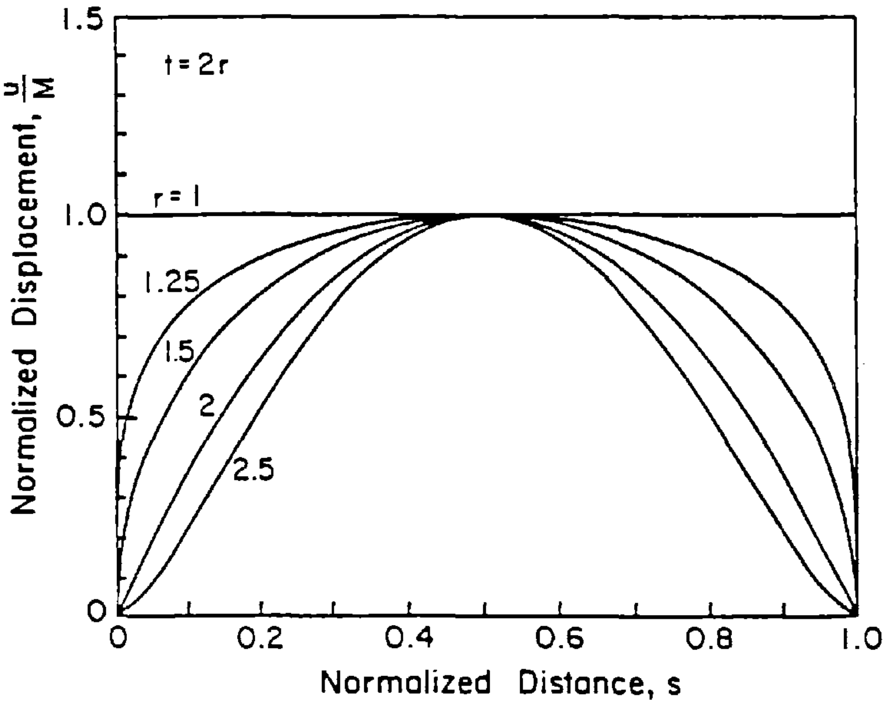
Figures 4-2a and b show different soil displacement patterns possible with this model, which are produced by varying the parameters, r and t. A symmetric pattern is given by a combination of shape parameters, with $t = 2r$, as shown in Figure 4-2a. Varying r changes the curvature of the pattern. In Figure 4-2b, a skewed pattern is given by t/r ratios greater than 2.

4.2.3 Soil Restraint Model

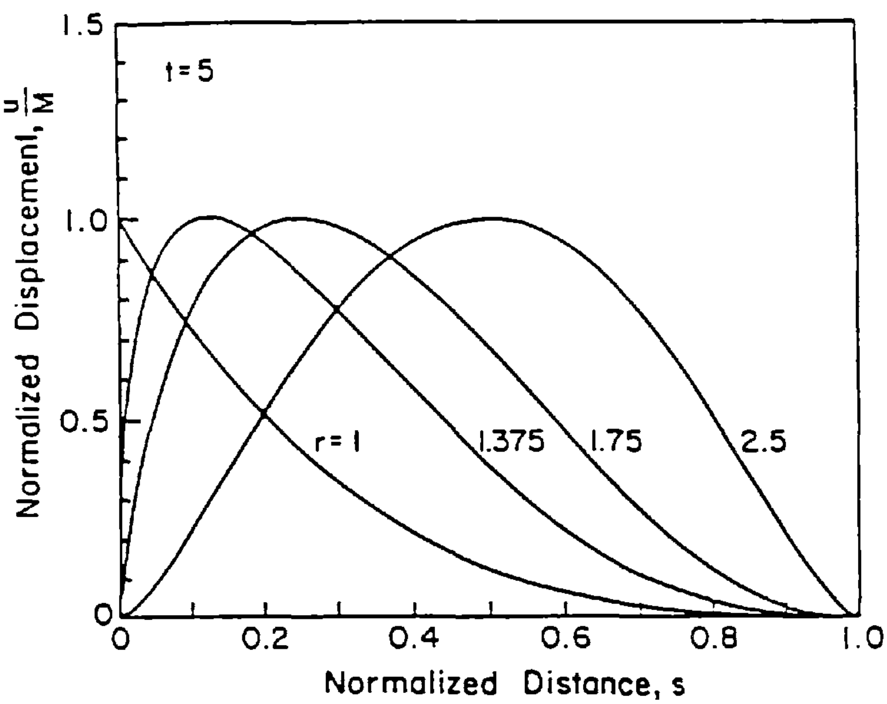
A pipeline which crosses a lateral spread will be displaced by horizontal soil movements. As the pipeline moves, it reacts with the surrounding soil such that loads are imposed on the pipeline.

Figure 4-3 shows a schematic drawing of distributed soil displacements imposed on a pipe resulting from a lateral spread. The magnitude of lateral pressure imposed on the pipeline is a nonlinear function of the amount of relative displacement between the soil and the pipe. As shown in the figure, the force-displacement relationship is often modeled by a spring/slider element when performing finite element or related numerical analyses [Committee on Gas and Liquid Fuel Lifelines, 1984]. The spring/slider elements represent the elasto-plastic behavior of the soil.

The loads imposed on the pipeline by soil displacements can be divided into three components: 1) axial, 2) transverse horizontal, and 3) transverse vertical. Figure 4-4 shows schematic relationships between the relative displacements between the pipe and soil in the three directions, u_x, u_y, and u_z, and the force per unit length transmitted to the pipe by the soil, f_x, f_y, and f_z. In general, there exists a maximum force per unit distance that can be transmitted between the pipe and the soil, which is represented in Figure 4-4 by F_x, F_y, and F_z. The maximum force occurs after some limiting displacement, designated for the longitudinal, lateral, and vertical directions as U_x, U_y, and U_z, respectively.



a) Symmetric Pattern.



b) Skewed Pattern.

FIGURE 4-2. Plots of Symmetric and Skewed Displacement Patterns Generated by a Modified Beta Distribution [after Tawfik and O'Rourke, 1986]

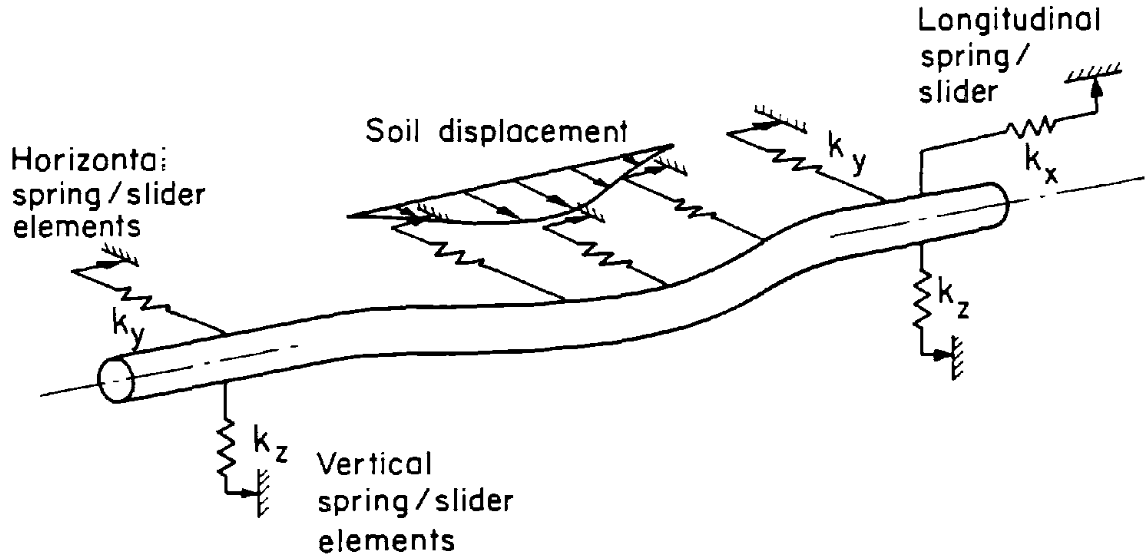


FIGURE 4-3. Schematic Representation of Soil Reactions

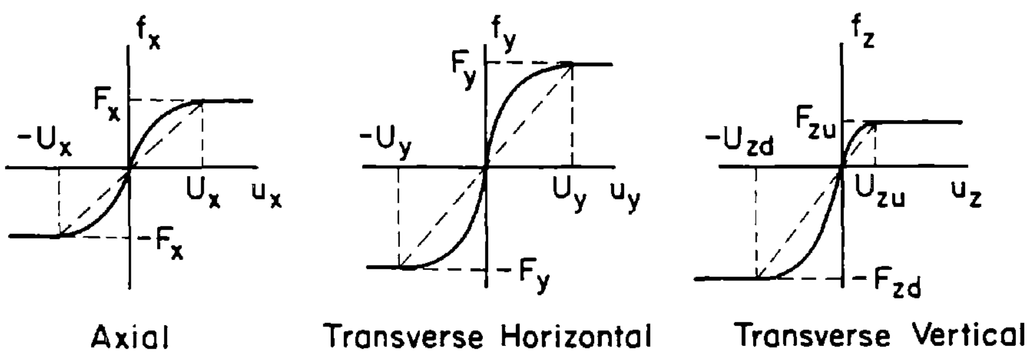


FIGURE 4-4. Force-Displacement Curves for Soil-Pipeline Interaction [after O'Rourke, et al., 1985; Committee on Gas and Liquid Fuel Life-Lines, 1984]

The resistance developed along the sides of the pipeline, or the axial component of force, resembles that of skin friction along piles. A bilinear model, as shown in Figure 4-4a, can be used to represent the longitudinal response. The transverse horizontal component of force is the restraint or load that the soil provides in response to the horizontal displacement of the pipe. This type of soil/pipeline interaction is analogous to that of a vertical anchor plate. The relationship between displacement and horizontal force has been investigated by experiments on buried pipes in dry uniform sand [Audibert and Nyman, 1977; Trautmann and O'Rourke, 1985]. The results of these studies suggest that the horizontal force/displacement relationship can be given by a hyperbolic function. Trautmann and O'Rourke [1985] describe a method for selecting a simplified, bilinear representation of the transverse horizontal soil reaction, which is based on the selection of a secant value for the horizontal stiffness.

Unlike the axial and horizontal components, the vertical restraint is not symmetrical in the upward and downward directions, as indicated in Figure 4-4c. For the case of downward motion, a bilinear elasto-plastic model is often assumed. Because the pipeline acts as a cylindrically-shaped strip footing, the maximum downward force can be given by the conventional bearing capacity equation [e.g., Sowers, 1979] with the displacement at maximum force estimated as 0.10 to 0.15 the pipe diameter.

For the case of upward motion, the results of full-scale laboratory tests have been used to define empirical formulae for the relationship between force and displacement. Trautmann, et al. [1985] have proposed both hyperbolic and bilinear relationships between the force per unit pipe length and relative upward movement. The bilinear relationship is defined simply on the basis of a maximum force which depends on the pipe diameter and depth of burial, soil unit weight, and soil angle of shearing resistance. Their experimental results showed that the maximum force was developed independent of density at a displacement of approximately one percent of the pipe depth.

4.2.4 Model for Pipeline Material Properties

Under large ground deformations, pipelines can be subjected to loads which will deform the pipe well into the plastic range. Analytical models therefore must account for the nonlinear behavior of both the soil and pipeline material.

Tawfik and O'Rourke [1986] developed a formulation for the true-stress/

true-strain relationship of steel given by:

$$\sigma = a_0 + a_1 \epsilon + a_2 e^{-b_1 e} + a_3 \epsilon^{b_2} \qquad (4\text{-}2)$$

in which:

- σ is the true axial stress,
- ϵ is the true axial strain, and
- a and b are constants determined from laboratory stress-strain data.

True stress is differentiated from engineering stress in that it is defined as the applied load divided by the actual area of the cross-section, rather than the original cross-sectional area. Similarly, true strain is defined as the elongation of the test specimen, due to an increment of applied stress, divided by the length of the specimen prior to the application of that increment. True-stress and true-strain can be related to engineering stress and strain, σ and ϵ, respectively, by the relationships:

$$\sigma = \sigma_n(1 + \epsilon_n) \qquad (4\text{-}3)$$

$$\epsilon = \ln(1 + \epsilon_n) \qquad (4\text{-}4)$$

For Equation 4-2, the a parameters are selected by linear regression, and the b parameters are estimated according to special procedures outlined by Tawfik and O'Rourke [1986]. Table 4-I lists the a and b parameters that can be used in Equation 4-2 to model the tensile stress-strain relationship for X-52, X-60, X-65, and X-70 grade steels. Figure 4-5 shows the true-stress/true-strain curves generated by Equation 4-2, with the parameters summarized in Table 4-I in comparison with actual test data.

The true-stress/true-strain curves were used by Tawfik and O'Rourke [1986] to calibrate a constitutive model for plastic behavior of the pipeline steel. The constitutive relationship was based on the two-surface plasticity model proposed by Dafalias and Popov [1976] and described in later work by Dafalias [1983].

In this study, the two-surface plasticity model of Dafalias and the

TABLE 4-I. Parameters for True-Stress/True-Strain Curves[1] [after Tawfik and O'Rourke, 1986]

Grade	b_1	b_2	a_0	a_1	a_2	a_3
X-52	970	0.5	339	-1301	-346	1015
X-60	850	0.5	369	-1856	-372	1324
X-65	900	0.5	416	-1673	-422	1191
X-70	850	0.5	497	-763	-525	691

[1]All a-values expressed as MPa.

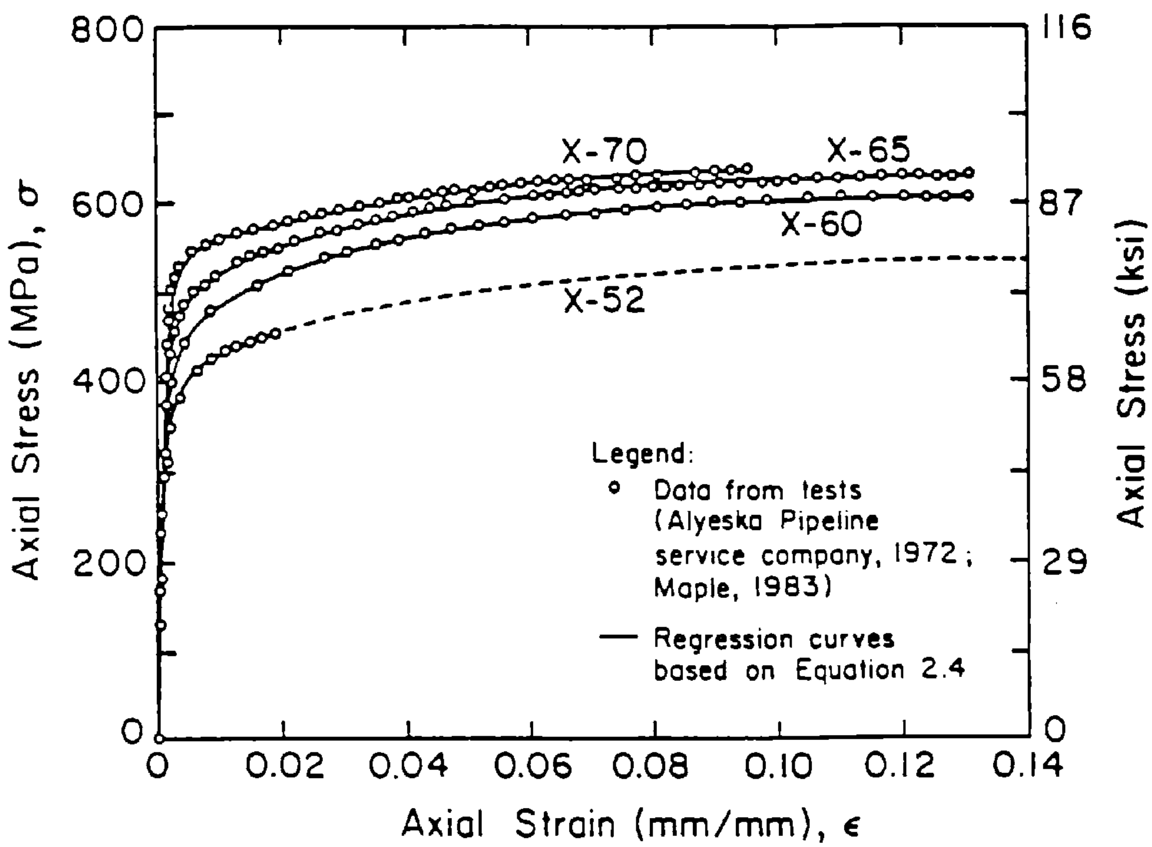


FIGURE 4-5. True-Stress/True-Strain Curves for X-Grade Steel [after Tawfik and O'Rourke, 1986]

formulation given by Equation 4-2 were used to represent the stress-strain performance of X-grade steel. This approach accounts for plastic deformation and strain hardening effects in a comprehensive way.

4.3 COMPUTER MODEL

A computer code, UNIPIP, was developed at Cornell University [Tawfik and O'Rourke, 1986] to solve a variety of soil/pipeline interaction problems resulting from earthquake-induced permanent ground displacements. This computer code can be used to evaluate buried pipeline response to various soil displacement patterns, including abrupt surface faulting and distributed movements caused by lateral spreading.

In most analytical models which evaluate soil/pipeline interaction, the pipeline is represented in two dimensions as a beam or cable. The models are not able to account for three-dimensional shell deformation. To have the capability of modeling such failure modes as compressive wrinkling and cross-sectional ovaling, UNIPIP was developed according to rigorous shell theories. The general shell formation allows for an evaluation of stress and deformation resulting from a combination of internal pressure, bending, and axial compression. The user has the choice of the degree of resolution by specifying the number of elements in the circumferential direction of the pipeline. A small number of elements will essentially reduce the problem to the thin-walled beam solution.

The UNIPIP model uses a numerical method based on a new technique for forming the structural stiffness matrix called the Finite Difference Jacobian Evaluation approach [Gill, et al., 1981; Dennis and Schnabel, 1979]. The numerical procedure is based on a weighted residual method using a total Lagrangian displacement formulation. The shell element is formulated using B-splines as basis functions. With B-splines, the number of numerical degrees of freedom can be minimized while still satisfying continuity of both displacements and curvatures between elements.

The input to the model includes the number of degrees of freedom and pipeline segments, boundary conditions, magnitude and pattern of soil displacements, and soil and pipeline properties. The output consists of the nodal displacements and stresses. The output values can be processed by the computer code to give both pipeline and soil displacements along the length of the pipeline at any increment of total applied soil displacement, or time step. Likewise, maximum strains, which are the combination of both the bending and axial strains, can be plotted along the length of the pipeline for a specified time step. The soil force mobilized along the pipeline length for a given time step is processed independently from the computer code. Similarly, the cumulative maximum strain the pipeline experiences for a given maximum soil displacement must be computed separately.

4.4 PARAMETRIC STUDY

4.4.1 Introduction

A parametric study was conducted to determine the sensitivity of buried pipeline response to several lateral spread characteristics. By holding constant all but one parameter for each trial, the influence of that parameter on the stresses and strains induced in the pipeline can be evaluated. The computer program, UNIPIP, was used to study the response of a buried steel pipeline as a function of the maximum displacement of a lateral spread, the width of the slide over which the maximum displacement develops, and the type of soil in which the pipe is embedded.

The pipeline was assumed to cross the spread perpendicular to the direction of ground movements, as shown in Figure 4-1a. This configuration was chosen because pipelines are often located along contours of equal elevation for ease of construction. Accordingly, this study focuses on the horizontal displacements imposed by the slide. Since only transverse horizontal displacements are considered, a two-dimensional model is used with no displacements in the vertical plane. It was further assumed that the horizontal movements were distributed across the width of the spread according to a symmetric pattern. This pattern is consistent with the distribution of ground movements that can be inferred from photographs of lateral spread deformations at several locations from the 1906 San Francisco earthquake. The analysis did not account for abrupt soil displacements.

The pipe cross-section is assumed to remain undistorted and plane as the pipeline bends, not allowing, in this case, for wrinkling and ovaling of the pipe wall. This assumption is consistent with the results of a study by Kennedy, et al. [1977], in which it was found that significant hoop ovaling should not occur, provided that the diameter to wall thickness of the pipeline, D/t, is less than 130 for a depth of burial to pipe diameter ratio, H/D, of 1.5, or less than 75 for a H/D ratio of 3.5. If wrinkling and ovaling are not considered, then beam-column formulations will be adequate for analyzing deformations [Tawfik and O'Rourke, 1986]. In this study, the pipeline was modeled according to beam-column theory.

4.4.2 Discussion of Model Parameters

Figure 4-6 presents the configuration of the pipeline subject to lateral spread displacements, as modeled in the parametric study. The orientation of the pipe with respect to the lateral spread is shown in Figure 4-6a. The distribution of soil displacements is assumed to be symmetric across the length of pipe, with a maximum at the center of the slide. The shape of the soil displacement curve is characterized by the beta probability density function, as described in Section 4.2.2. The parameters which describe the soil displacement pattern are shown in Figure 4-6b. The width of the lateral spread is given by the distance, B. The maximum displacement is given by M, and s_m locates the peak. The variables, r and t, are the shape parameters used in Equation 4-1. The curve parameters for the distribution pattern used in the study were $s_m = 0.5$, $r = 2.5$, and $t = 5.0$. The pipeline was assumed to be anchored at a distance, A, from the margin of the slide such that, in the analysis, the bending strains induced in the pipeline near the anchor point were smaller than 1×10^{-5}.

The buried pipeline modeled in this study is a continuous girth welded pipeline composed of X-60 grade steel. The elastic/plastic characteristics of the pipeline are consistent with those of X-60 grade steel given in Table 4-I. The outside diameter of the pipe was assumed to be 610 mm and the wall thickness was taken as 9.5 mm. The depth of burial was assumed to be 1.2 m from the ground surface to the top of the pipe.

Bilinear representations of the soil force-displacement relationships, as discussed in Section 4.2.3, were used for the axial and horizontal components of the soil force. Since the problem is two-dimensional, the maximum

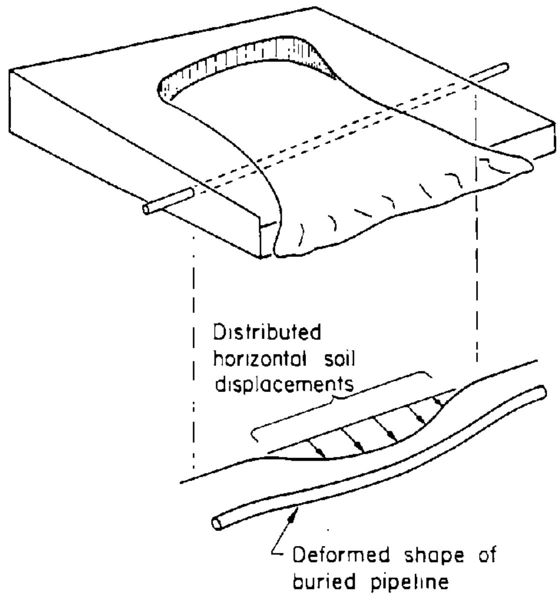


FIGURE 4-6a. Orientation of Pipeline in Parametric Study

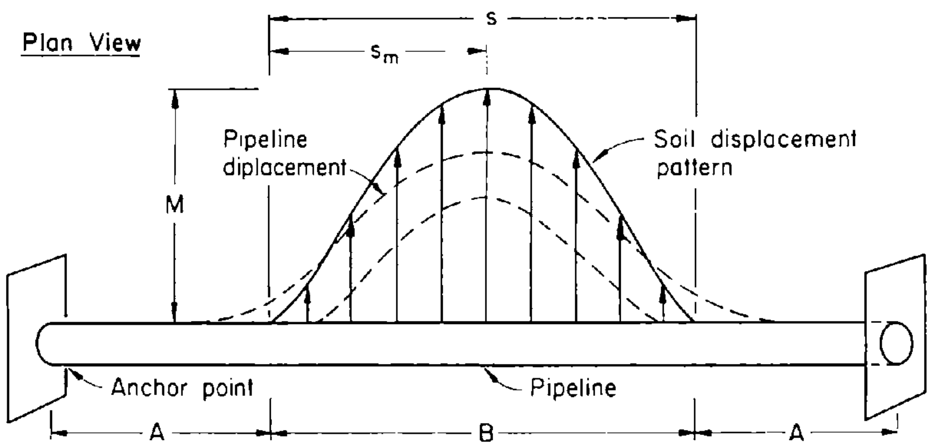


FIGURE 4-6b. Parameters Describing Lateral Spread Displacement Pattern

upward and downward soil forces, F_{zu} and F_{zd}, were not considered. The maximum forces, F_x and F_y, and the corresponding relative displacements, U_x and U_y, are inputs to the computer model. The computer code uses these values to define the bilinear representation of the soil/pipeline interaction.

For the numerical analysis, the pipeline was divided into a number of segments, with each segment having six numerical degrees of freedom. The pipeline was assumed to be fixed at the ends, with zero slope. The total number of degrees of freedom for the pipeline was given, therefore, by six times the number of segments, minus the six fixed end conditions. For a pipeline divided into 10 segments, the total degrees of freedom for the pipeline were 54. The number of increments over which the maximum displacement was applied to the pipeline, described as time steps in the model, was also specified. The numerical procedure was specified to use C^3 continuous quintic B-splines, which gives continuity of both displacements and curvature.

4.4.3 Trial Input

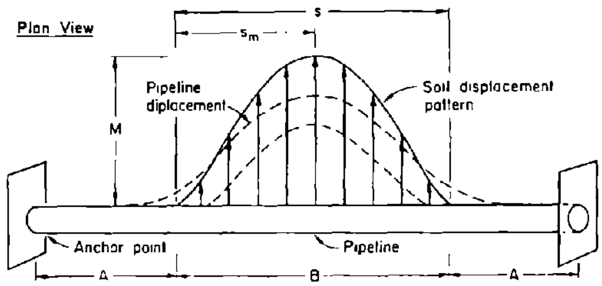
Five trials were run for the parametric study. In the first three trials, the width of the lateral spread was varied, holding the rest of the parameters constant. This test configuration permitted an evaluation of the effects of distributed movement over increasingly longer segments of the buried pipeline. In the last two trials, the soil properties were varied to evaluate the effect of soil density and shear strength on the stresses and strains induced in the pipeline for a fixed maximum displacement and width of slide. In all trials, a maximum displacement, M, of 1.5 m was applied.

Tables 4-II and 4-III summarize the input parameters for each trial. In Table 4-II, the parameters which characterize the lateral spread are presented. These parameters are defined by the figure given with the table. Also included are the number of segments into which the pipeline was divided and the number of increments over which the maximum displacement was applied. In the first three trials, the width of the lateral spread was increased from 10, to 30, to 50 m. In Trials 4 and 5, the geometric parameters were held constant and replicate those of Trial 2.

The soil properties used in each trial are presented in Table 4-III. In the first three trials, the soil was modeled as a medium dense partially saturated sand. In Trial 4, the pipeline was assumed to be embedded in a partially saturated dense sand. In Trial 5, the soil was a dense sand with higher shear strength than that of Trial 4.

Also summarized in Table 4-III are the maximum soil forces, F_x and F_y, and the corresponding displacements, U_x and U_y, at which these forces are mobilized. These parameters define the bilinear force/displacement model

TABLE 4-II. Geometric and Numerical Characteristics



Parameter	Trial Number 1	2	3	4	5
B, m	10.0	30.0	50.0	30.0	30.0
A, m	15.0	35.0	75.0	35.0	35.0
M, m	1.5	1.5	1.5	1.5	1.5
s_m	0.5	0.5	0.5	0.5	0.5
r	2.5	2.5	2.5	2.5	2.5
t	5.0	5.0	5.0	5.0	5.0
Time steps	100	100	100	100	200
Pipe segments	10	10	7	7	10

TABLE 4-III. Soil Properties and Restraint Parameters

Parameter	Trial Number 1	2	3	4	5
γ, kN/m^3	18.8	18.8	18.8	20.4	20.4
$\bar{\phi}$, degrees	35	35	35	40	45
δ, degrees	28	28	28	32	36
F_x, kN/m	23.4	23.4	23.4	29.8	31.7
U_x, mm	5.1	5.1	5.1	5.1	5.1
F_y, kN/m	77.0	77.0	77.0	106.2	125.0
U_y, mm	61.0	61.0	61.0	45.7	45.7

Note: γ = soil unit weight
$\bar{\phi}$ = soil friction angle
δ = soil/pipeline interface friction angle
F_x = max axial soil restraint
U_x = max axial displacement
F_y = max horizontal soil restraint
U_y = max horizontal displacement

described in Section 4.2.3. The Committee on Gas and Liquid Fuel Lifelines [1984] and Tawfik and O'Rourke [1986] provide comprehensive discussions on the selection of values for the maximum soil forces, F_x and F_y, and the corresponding maximum displacements, U_x and U_y.

4.4.4 Summary of Analytical Results

Figures 4-7 through 4-11 summarize the analytical results for each of the five trials. In part (a) of each of these figures, transverse horizontal soil displacements, given by solid lines, and the corresponding pipeline displacements, given by dashed lines, are plotted as a function of distance along the pipeline. Two sets of curves are shown for a maximum imposed soil displacement of 0.75 m and 1.5 m, respectively.

In part (b), maximum strains developed on the tensile and compressive sides of the pipe are plotted as a function of distance along the pipeline. Following the sign conventions of structural mechanics, tensile and compressive strains are plotted as positive and negative, respectively. The strains are a combination of the axial and bending strains.

The percentages of the maximum longitudinal and horizontal soil forces which were mobilized at 1.5 m of soil displacement are plotted as a function of pipeline length in parts (c) and (d), respectively, for each of the trials.

<u>Trial 1</u>. The displacements shown in Figure 4-7 are associated with a lateral spread width of 10 m, the smallest width studied. There is a substantial difference between the soil and pipeline movements, with a maximum 0.70 m relative displacement for a total soil movement of 1.5 m. The maximum pipeline strains correlate with the location of maximum pipeline curvature, and approximately correspond to the location of maximum curvature in the soil displacement profile. The largest strains of 2.00% in tension and 1.38% in compression are located at the center of the spread. The full longitudinal frictional resistance is mobilized from the point where soil and pipeline displacements are equal, at about 3 m from the center of the slide, to about 4 m from where the pipeline was anchored. Full horizontal soil resistance is mobilized across the central 15 m of the pipeline, with the exception of two relatively small lengths near the location of equal

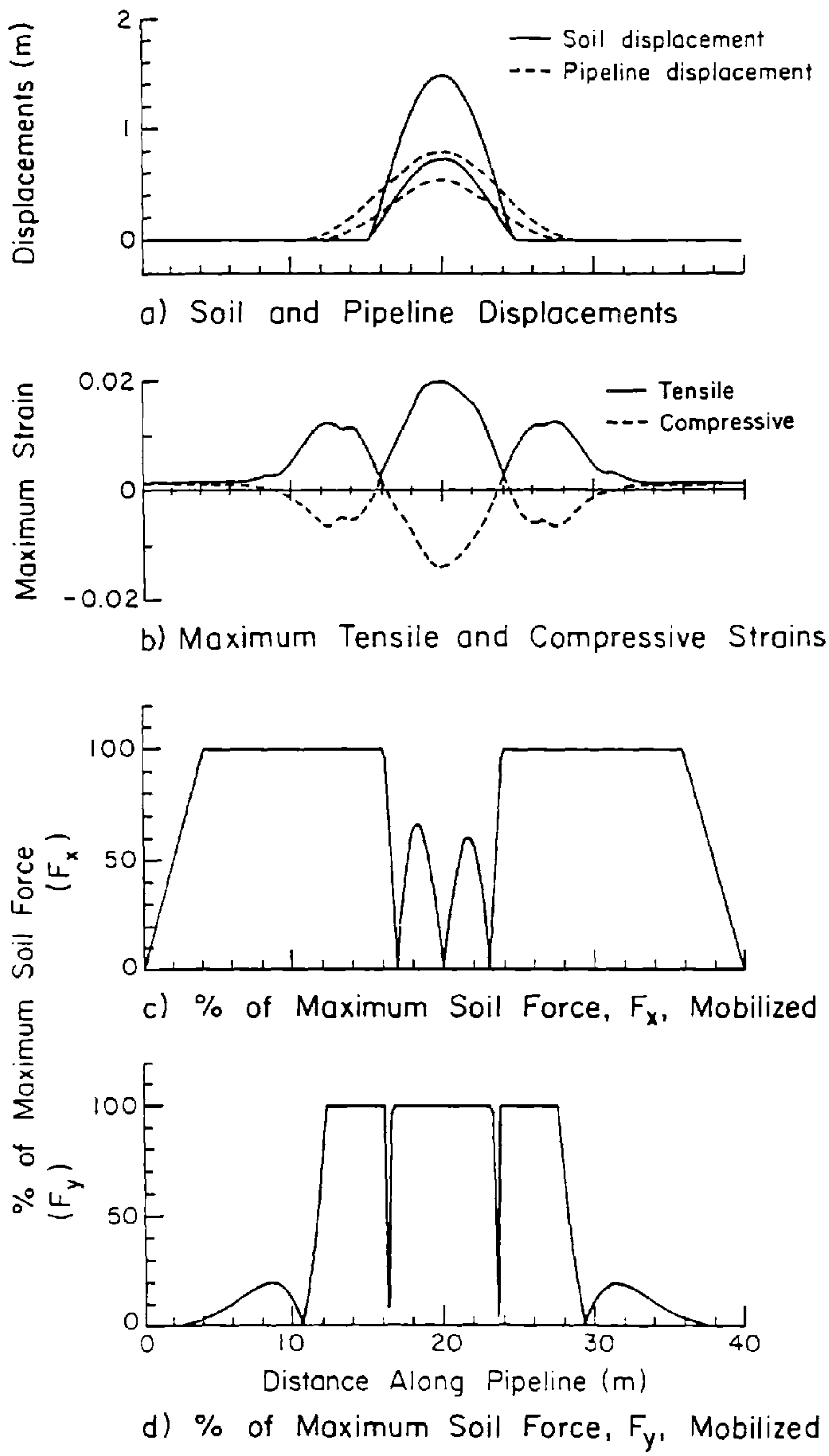


FIGURE 4-7. Analytical Results for Trial 1; B = 10 m, $\bar{\phi}$ = 35 Degrees

soil and pipeline movement.

<u>Trial 2</u>. A length of 43 m of pipeline displaced horizontally in response to a lateral spread width of 30 m in Trial 2. The relative displacement between the soil and pipeline was 0.23 m, which is significantly reduced from that of Trial 1. The maximum tensile and compressive strains, which occurred at the center of the slide, were 0.64% and 0.31%, respectively. Maximum longitudinal soil resistance was mobilized along much of the pipe length, except for the central 17.5 m and a distance of 8.5 m from each anchor point. The central 38 m of the pipeline mobilized the full horizontal soil resistance, except for two 2.7-m-long sections where the pipeline displacement approximated the soil displacement.

<u>Trial 3</u>. In Trial 3, the pipeline displacement closely approximated that of the soil, with the maximum relative displacement between the soil and pipeline being only 0.02 m. A 56-m-long segment of the pipeline displaced horizontally as a result of the 50-m-wide lateral spread. The maximum tensile and compressive strains induced in the pipeline were 0.31% and 0.16%, respectively. The reduction of strains at the center of the slide corresponds to a decrease in curvature in the pipeline at this point, manifested in the pipeline displacement curves by the blunted shape of the maximum point. Except for the central 26 m of the pipe and an additional 24 m at each end near the anchor points, the pipeline mobilized the maximum longitudinal soil resistance. The maximum horizontal soil resistance was mobilized, however, only over a small section of the pipeline.

<u>Trial 4</u>. The displacements shown in Figure 4-10 are associated with a lateral spread width of 30 m, equal to that of Trial 2. The soil density and shear strength were increased in this trial. The amount of pipeline displacement increased from that of Trial 2, with a relative soil displacement between the soil and pipeline of 0.13 m. The pipeline displacements, in this case, were distributed over a pipeline length of 46 m. The tensile and compressive strains of 0.81% and 0.35%, respectively, were also larger than those of Trial 2. The larger strains correspond to the greater curvature of the pipeline resulting from the increased pipeline displacement, which developed over roughly the same pipeline length as in Trial 2. The maximum longitudinal horizontal resistance was mobilized over all but the central 18 m of the pipe and an additional 5 m near each of the anchored ends. The

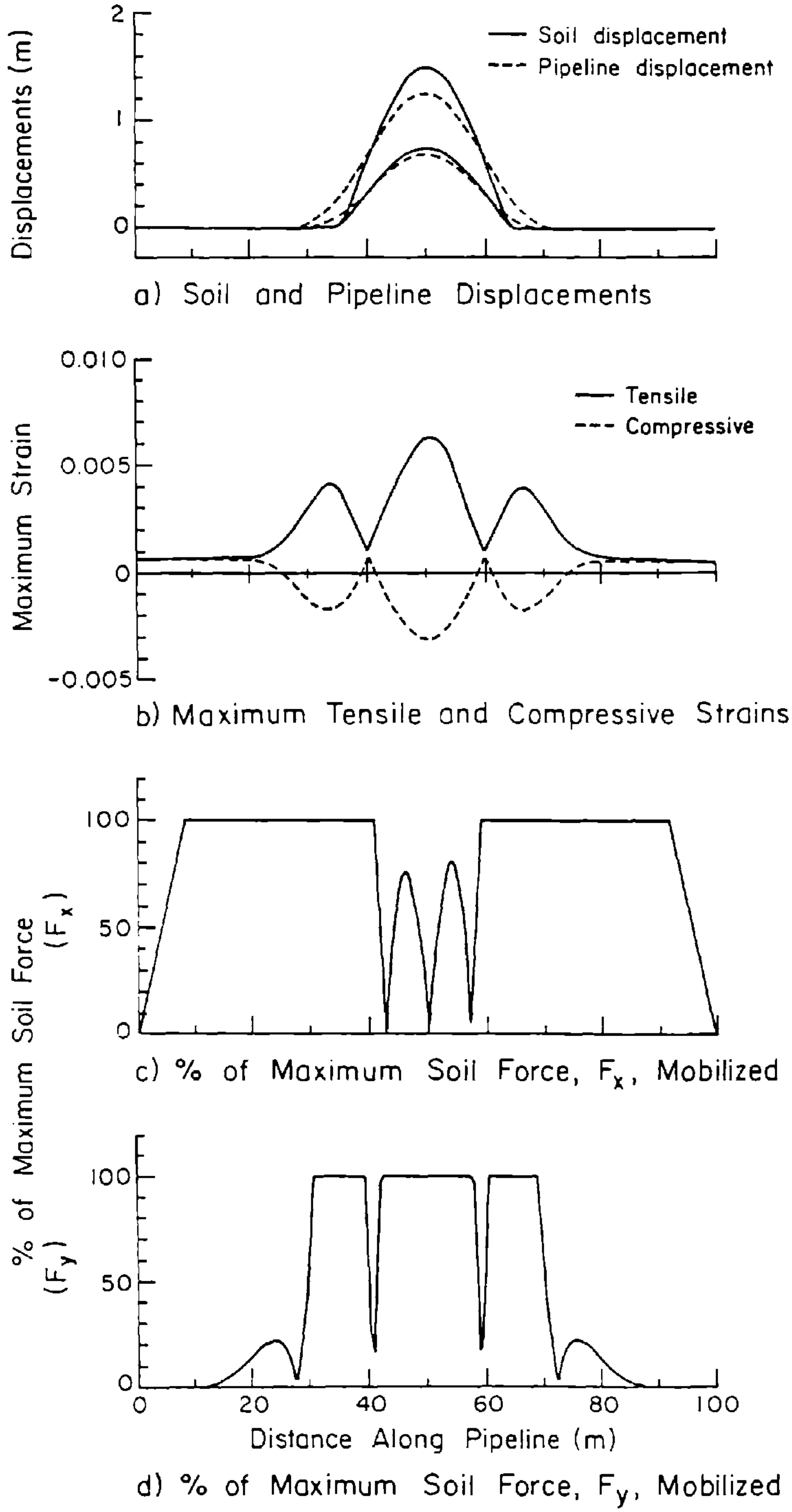


FIGURE 4-8. Analytical Results for Trial 2; B = 30 m, $\bar{\phi}$ = 35 Degrees

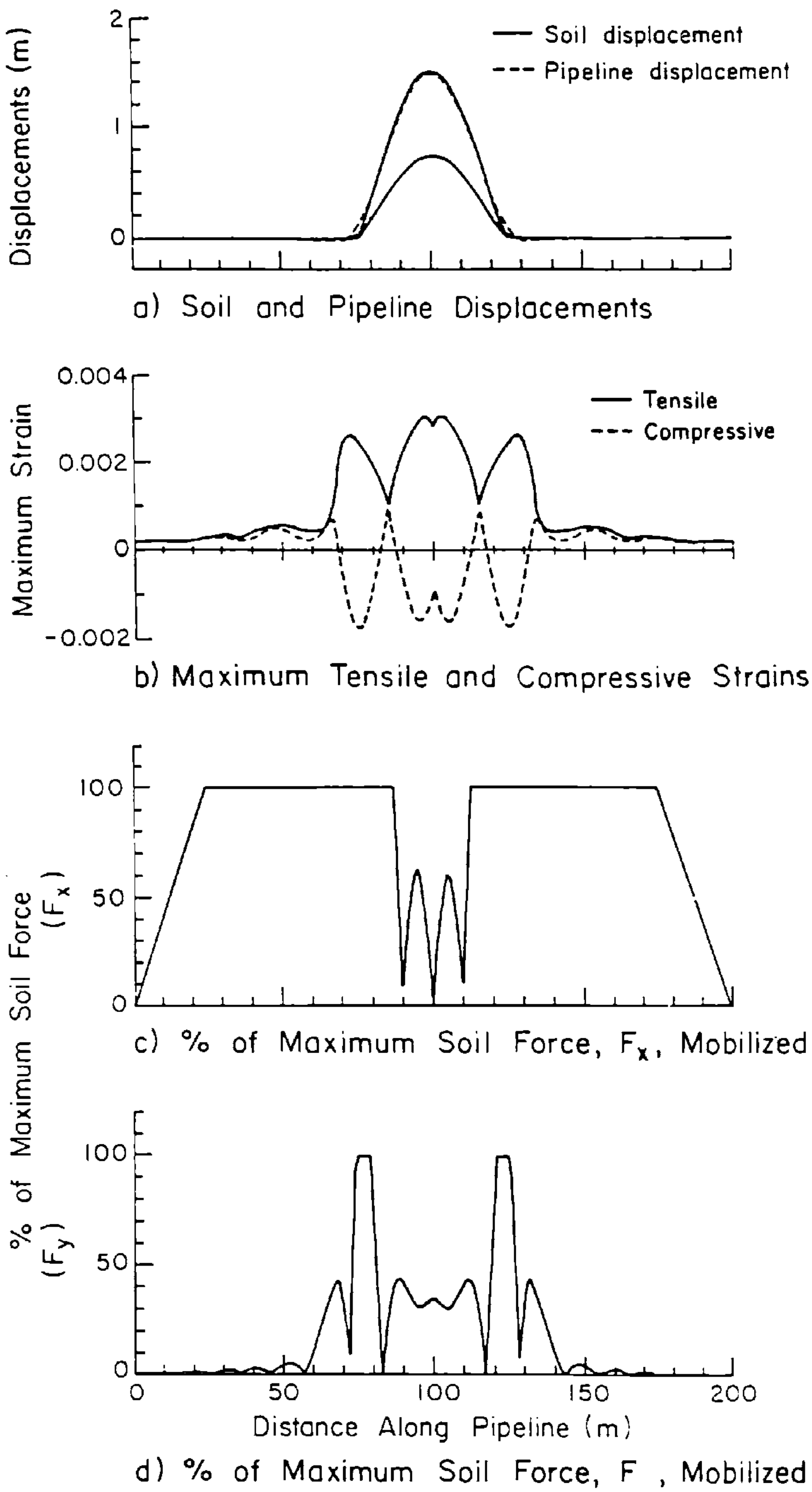


FIGURE 4-9. Analytical Results for Trial 3; B = 50 m, $\bar{\phi}$ = 35 Degrees

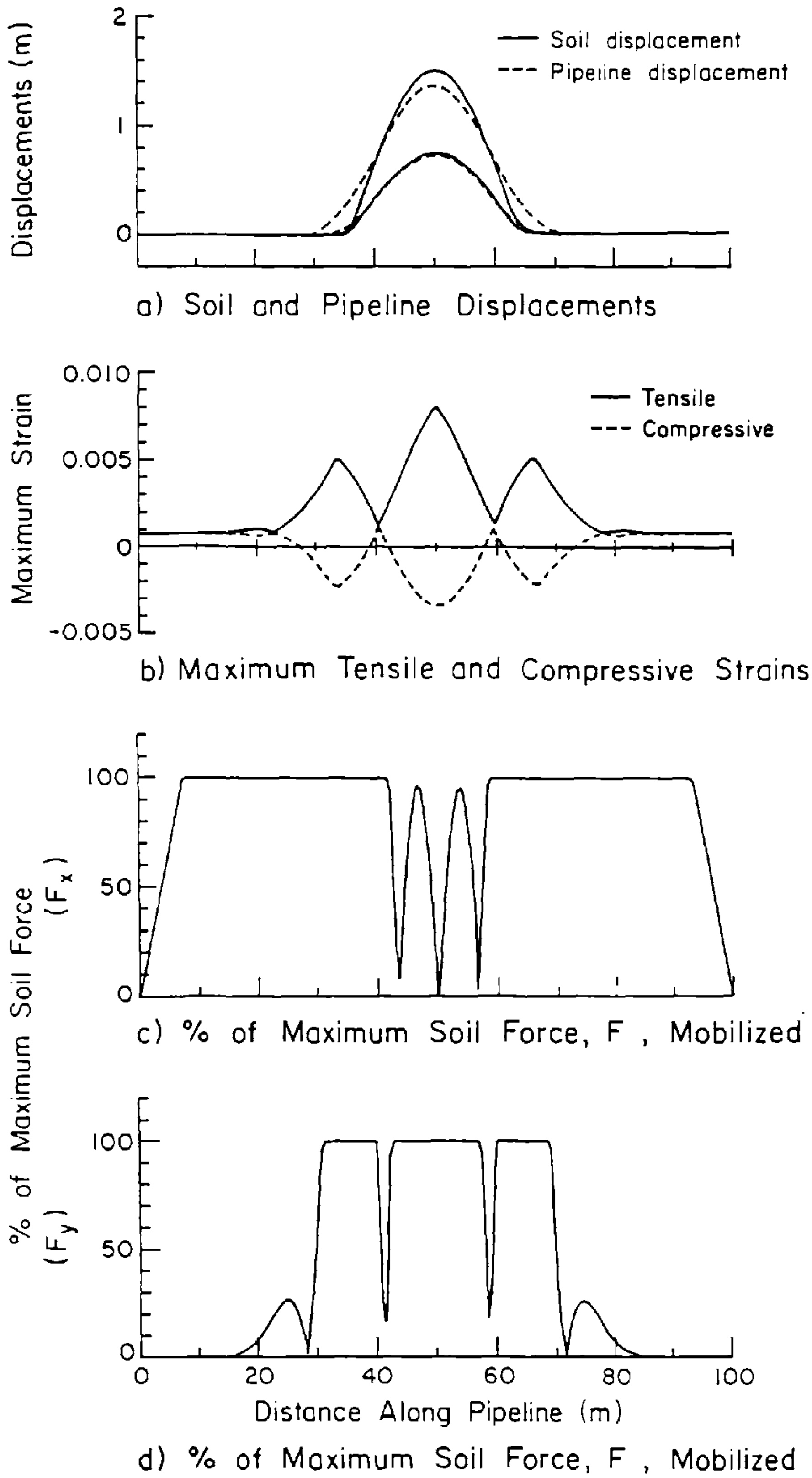


FIGURE 4-10. Analytical Results for Trial 4; B = 30 m, $\bar{\phi}$ = 40 Degrees

maximum horizontal resistance was mobilized over a 40-m-long central segment of the pipeline, with the exception of two small 2.8-m-long segments.

Trial 5. The imposed soil displacements in this trial were again distributed over a 30-m slide width. Because of the higher shear strength, the pipeline was displaced a greater distance than that of either Trial 2 or 4, with a relative displacement between the soil and pipeline of only 0.08 m. This displacement developed over a 40-m-long segment of the pipeline. The pipeline strains did not increase significantly from those of Trial 4, with a tensile strain of 0.82% and a compressive strain of 0.39%. The full longitudinal resistance was mobilized over most of the pipeline length, except the central 15.7 m of the pipeline and another 7.1 m at each end near the anchored points. The maximum horizontal resistance developed over 36 m of the central portion of the pipe, with the exception of two 3.6-m-long segments.

Table 4-IV summarizes the analytical results for each of the five trials. The values for maximum displacement and maximum tensile and compressive strains, which are given in the table, pertain to the maximum imposed soil displacement of 1.5 m.

4.4.5 Discussion of Analytical Results

Figure 4-12 shows the relationship between the maximum tensile strain at the center of the pipeline and the maximum soil displacement. The three curves represent the first three trials in which the soil properties were held constant and the width of the lateral spread was increased from 10 to 50 m. The curves show a large increase in maximum tensile strain as the slide width decreases from 50 to 10 m. At a maximum soil displacement of 0.50 m, there is a 7-fold and 15-fold increase in tensile strain when the 10-m-wide spread is compared with the 30 and 50-m-wide spreads, respectively. Similarly, at the maximum soil displacement of 1.5 m, there is a 3-fold and 6.5-fold increase in tensile strain between the 10-m-wide lateral spread and the 30 and 50-m-wide spreads, respectively.

The smallest lateral spread width leads to the largest relative movements between the pipe and soil, with the result that the full horizontal soil resistance is mobilized at a significantly smaller magnitude of lateral

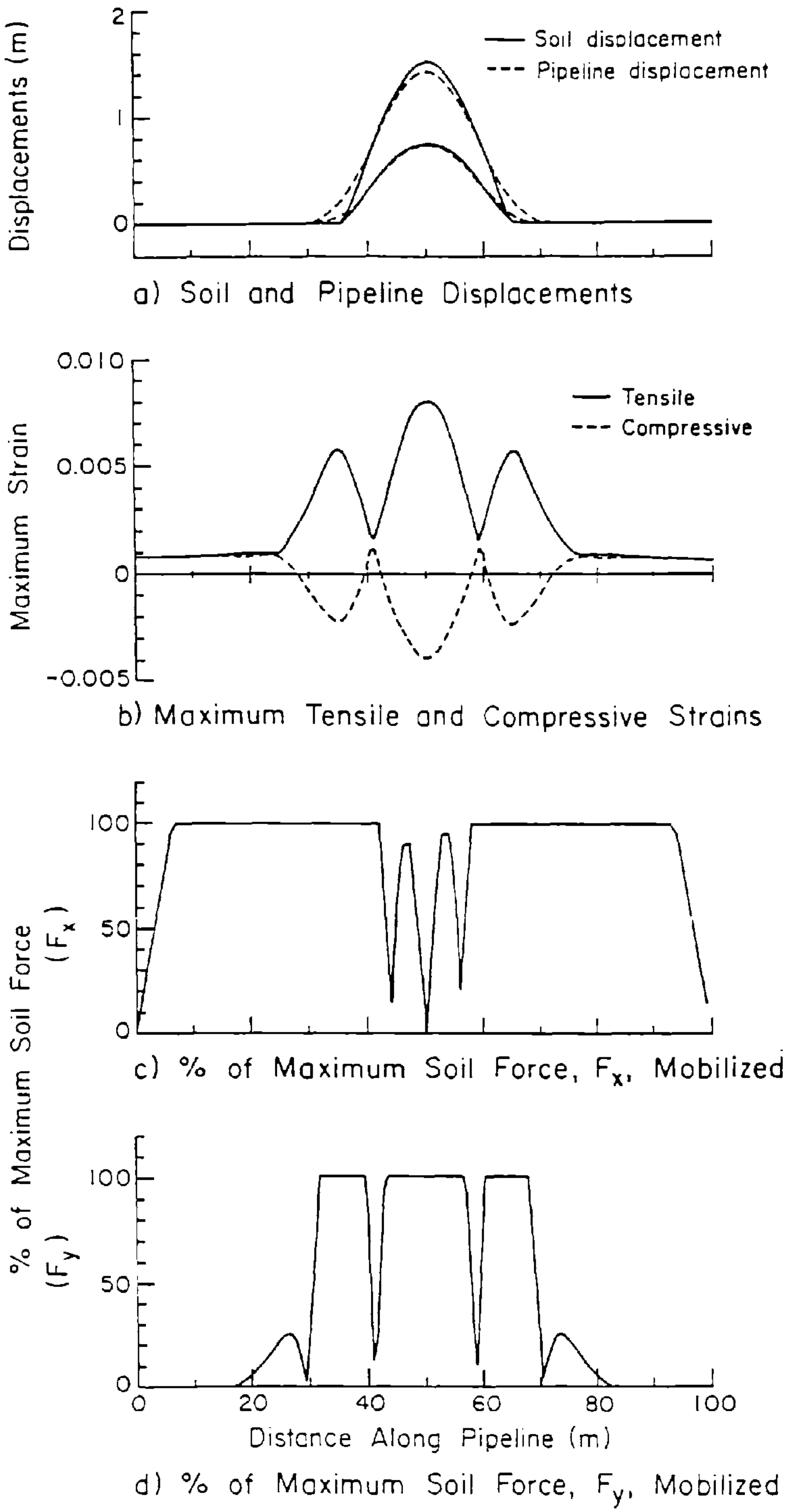


FIGURE 4-11. Analytical Results for Trial 5; B = 30 m, $\bar{\phi}$ = 45 Degrees

TABLE 4-IV. Summary of Analytical Results from Parametric Study

Results	Trial 1	2	3	4	5
Maximum pipeline displacements[1], m	0.80	1.27	1.48	1.37	1.42
Maximum strains[1], x 10^{-2}					
Tensile	2.00	0.63	0.31	0.81	0.82
Compressive	1.38	0.31	0.16	0.35	0.39

[1] At maximum imposed soil displacement of 1.5 m.

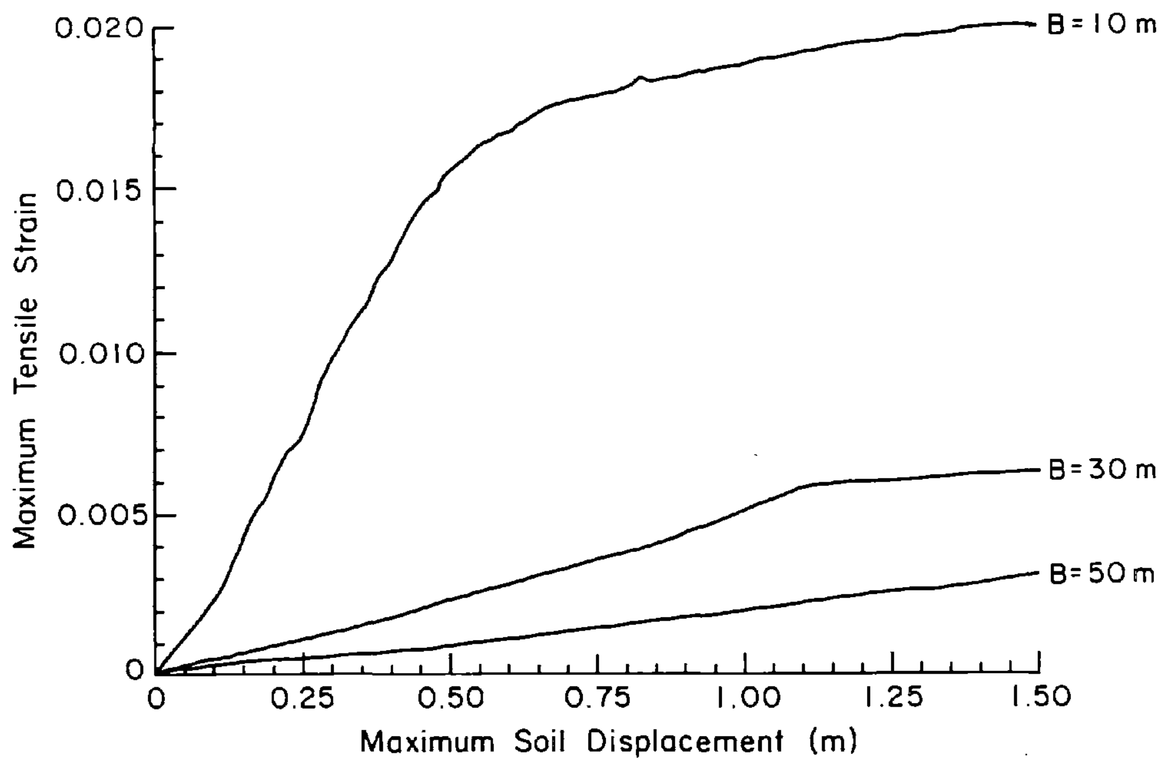


FIGURE 4-12. Maximum Tensile Strain vs. Maximum Soil Displacement for Various Lateral Spread Widths

spread movement. After the maximum horizontal forces have been mobilized, there is a rapid decline in the rate of strain increase relative to soil displacement. Figure 4-12 shows that the relationship between strain and soil displacement for the 50-m-wide spread is approximately linear for the full 1.5 m of imposed soil displacement. This pattern is consistent with the relatively small pipeline length over which the maximum horizontal soil force was mobilized, as evidenced in Figure 4-9.

The pipeline subject to the narrowest lateral spread reaches the onset of plastic yielding at a much smaller maximum imposed soil displacement than that associated with the wider spreads. Plastic strains developed in the pipeline of Trial 1 at displacements 5 and 12.5 times smaller than in the pipelines of Trials 2 and 3, respectively.

Figure 4-13 shows the maximum tensile and compressive strains in the pipeline as a function of the maximum lateral spread movement of 1.5 m over a width of 30 m. Three plots are shown in each of Figures 4-13a and b, corresponding to the different soil properties associated with Trials 2, 4, and 5. In the figure, the soil properties are referenced according to the soil angle of shearing resistance, $\bar{\phi}$. It should be recognized, however, that the parameters U_y and γ were held constant in Trials 4 and 5. Although $\bar{\phi}$ increases from 40 to 45 degrees for these two trials, the soil stiffness before plastic soil flow only increases by about 20%.

As shown in Figure 4-13, the relationship between strain and soil displacement is controlled by the linear elastic response of the soil up to soil movements of approximately 1.0 m. For displacements greater than 1.0 m, plastic flow of the soil relative to the pipe has a marked effect on performance, as evidenced by the break in slope of the strain versus displacement curves. There is little difference among the strain versus displacement plots in the lower elastic range of soil behavior. The principal differences occur in the tensile strain after 1.0 m of soil movement. These differences are related to the maximum soil resistance, as indexed by the soil angle of shearing resistance. There is a smaller difference between the three soil types for the compressive strains after the 1.0 m of soil displacement than there is for the tensile strains.

The tensile strains increase at a faster rate than compressive strains. At a lateral spread displacement of 1.0 m, the tensile strains are roughly 1.5 times as large as the compressive strains for each of the three soil types.

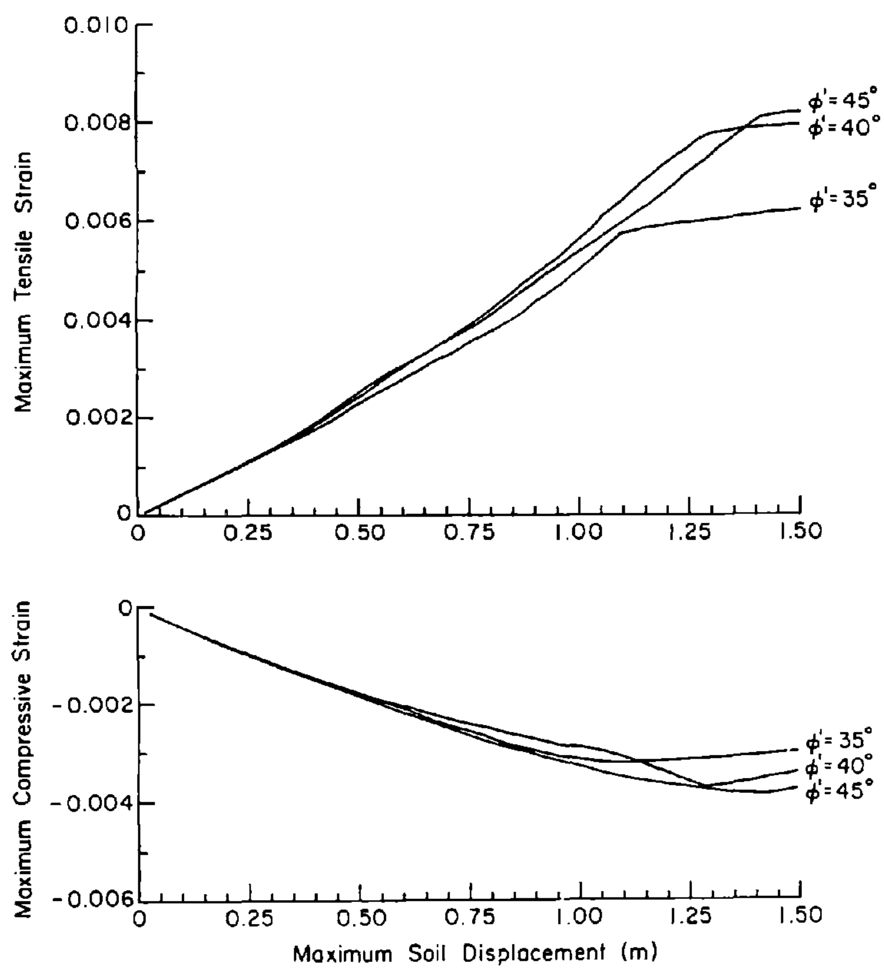


FIGURE 4-13. Maximum Tensile and Compressive Strains vs. Maximum Soil Displacement for Several Soil Types, Indicated by Internal Angle of Friction of Soil, $\bar{\phi}$

In Figure 4-13, it can be seen that the compressive strains begin to decrease after plastic flow of the soil, whereas the tensile strains continue to increase at a slow steady rate. This decrease in compressive strain may be attributed to a net elongation of the pipeline in the zone of maximum soil displacements. The somewhat larger values of strain for $\bar{\phi}$ = 40 degrees, when compared to those for $\bar{\phi}$ = 45 degrees, are most likely the result of differences in the numerical formulation of the problem. Trial 5 was run with 200 load steps, as compared with 100 load steps for Trial 4. Increased load steps will tend to accumulate larger amounts of strain under these highly nonlinear conditions.

A comparison of Figures 4-12 and 4-13 shows an important trend. The soil density and shear strength are shown to have only a small influence on the amount of strain induced in a pipeline from imposed soil displacements. The width of the lateral spread, on the other hand, has a marked effect on the magnitude of induced pipeline strains, the implications of which will be discussed in Section 4.5.

The analytical results summarized in Figures 4-12 and 4-13 apply for a pipeline with no internal pressure. Since most continuous steel pipelines carry fluids or gases at relatively high pressure, it is useful to consider pipeline response under these circumstances. Because the internal pressure imposes a radial stress which is very low compared with the hoop and longitudinal stresses, σ_θ and σ_ℓ, respectively, loading conditions for internal pressure can be simplified as a biaxial stress state. Accordingly, the von Mises [e.g., Johnson and Mellor, 1983] yield surface can be constructed from the following relationship:

$$\sigma_\ell = \frac{\sigma_\theta}{2} \pm \sqrt{\sigma_y^2 - \frac{3}{4}\sigma_\theta^2} \qquad (4\text{-}5)$$

in which σ_y is the yield stress.

Figure 4-14 shows the yield surface for X-60 steel in conjunction with the pipeline strain versus soil displacement plots of Figure 4-13. For a condition of no internal pressure, yielding occurs at the same strain in tension and compression. As shown in the figure, yielding is initiated in tension at a soil movement of approximately 0.43 m. In contrast, a maximum soil displacement of 0.57 m is required for the onset of compressive yielding. For an internal pressure of 207 MPa, which develops a hoop stress of 50% of the specified minimum yield stress, the onset of yielding occurs in compression at a maximum soil displacement of 0.35 m. The longitudinal tensile yield strength of the pipe is actually increased by the hoop stress.

Figure 4-14 illustrates how internal pressure affects the pipeline response to soil movement. Under conditions of high internal pressure, yielding would initiate in compression at soil displacements lower than those associated with the onset of yielding with no internal pressure. It should be emphasized that the onset of yielding does not imply failure. Additional soil movements can be sustained with plastic deformation of the pipe. The

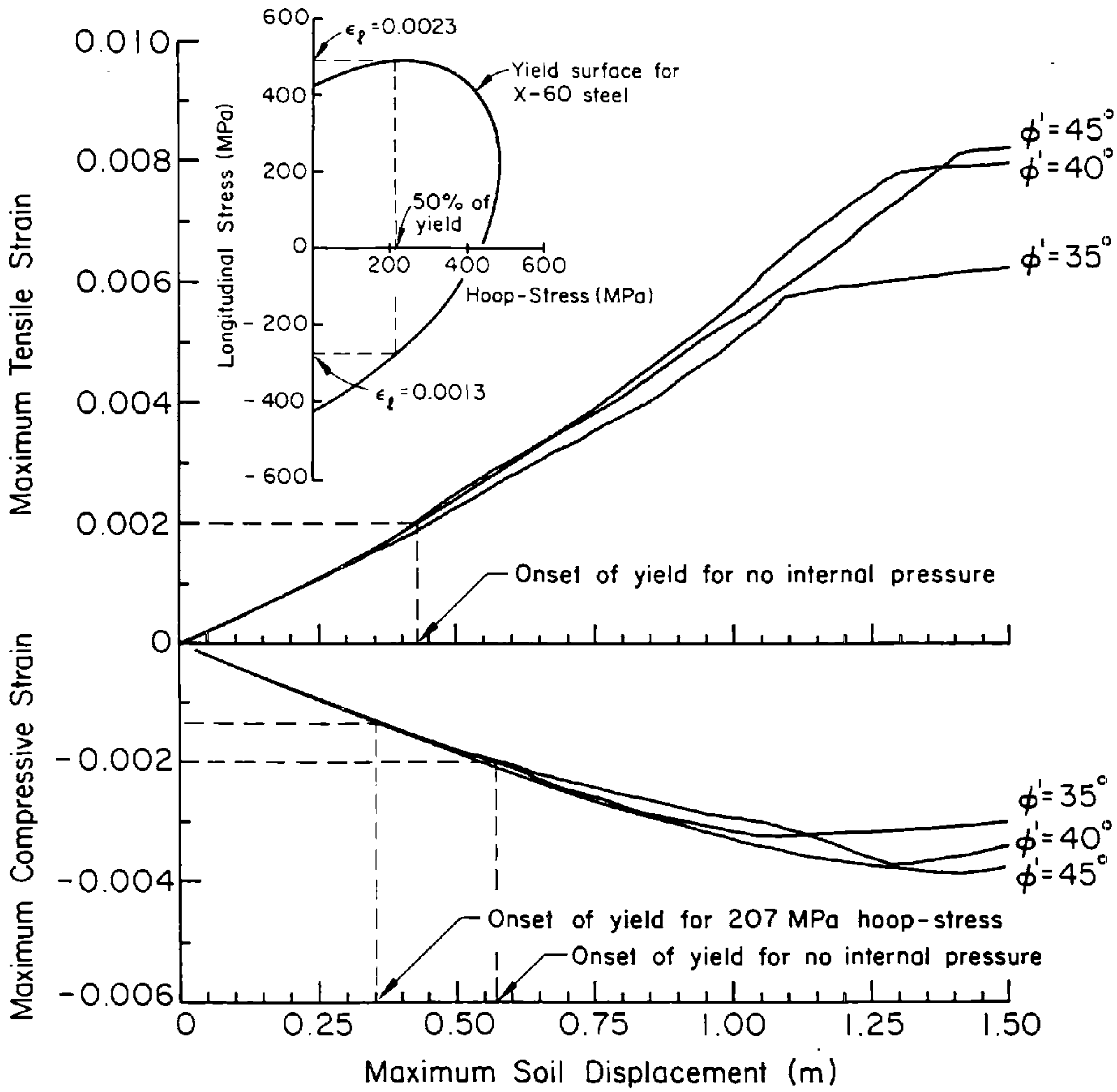


FIGURE 4-14. Influence of Internal Pressures in Pipeline on Onset of Plastic Yielding in Both Tension and Compression

accumulation of large plastic strain, however, will depend on the shell mode of deformation experienced by the pipe. This type of deformation is not clearly understood, although there have been notable recent contributions to the analysis of this problem [e.g., Tawfik and O'Rourke, 1986; Yun and

Kyriakides, 1986].

4.5 IMPLICATIONS FOR PIPELINE DESIGN

Several important aspects of buried pipeline response to lateral spread movements have been shown by this parametric study. An almost bilinear relationship exists between the pipeline strains and the maximum soil displacements, which is a function of the maximum soil resistance. Once plastic flow has been initiated in the soil, a steady but small increase in maximum tensile strain is indicated, with a similar decrease in compressive strain. In the region where plastic flow of the soil has been initiated, the pipeline is no longer subject to an increase in bending strain, only axial. Along those sections of the pipe where the soil restraint is still exhibiting elastic behavior, however, bending strains are still increasing. As the pipeline is displaced through interaction with the soil, an increasing length of pipeline undergoes deformation. Thus, as soil displacements continue, larger pipeline segments, which can significantly exceed the width of the slide, are affected.

A second important observation is that the lateral spread width has a much greater influence on the magnitude of pipeline strain than does the density and shear strength of the soil in which the pipeline is embedded. Therefore, in the siting and design of a pipeline, emphasis should be placed on understanding the geometric characteristics of the ground displacement patterns. The geologic conditions of the site will indicate potentially liquefiable zones, while geomorphologic and structural information will provide insight into the potential deformation patterns. A pipeline should be sited to avoid narrow zones of soil displacements. The parametric study indicates that lateral spread widths less than about 20 m may lead to relatively high plastic strains for transmission and trunk line facilities.

SECTION 5
CONCLUSIONS

5.1 LIQUEFACTION AND ASSOCIATED GROUND FAILURES

Liquefaction and associated ground deformations pose major hazards to lifeline systems in regions of seismic activity. Lifeline networks traverse large areas, and therefore will be exposed to a variety of geologic conditions. It is probable that sections of a pipeline system will cross faults and potentially liquefiable regions. Engineering measures to mitigate this hazard require: 1) an evaluation of site susceptibility to liquefaction, and 2) an assessment of the pattern of ground movement and soil failure resulting from liquefaction.

Even though the liquefaction susceptibility of a site may be known, it is extremely difficult to predict ground deformation patterns. Much of our knowledge of soil deformation caused by liquefaction has been learned from case history studies associated with actual earthquakes. This work summarizes observations of the magnitude and distribution of liquefaction-induced soil displacements associated with past earthquakes. Quantifiable patterns are important for testing predictive models and for evaluating limiting conditions of soil/structure interaction.

Liquefaction has been defined [Youd, 1973] as "the transformation of a cohesionless material from a solid state into a liquefied state as a consequence of increased pore pressure and reduced effective stress." During earthquake loading, the soil structure of a cohesionless deposit tends to decrease in volume in response to the application of cyclic strains. In a saturated soil, no decrease in volume can occur if the deposit is undrained, and thus there is a resulting increase in pore water pressure and a decrease in effective stress [Committee on Earthquake Engineering, 1985]. The soil deposits most conducive to liquefaction have been identified as recently deposited (i.e., Holocene or late Pleistocene) deltaic, river channel, flood plain, and eolian deposits, as well as loose or partially compacted fill.

Several ground failure modes are associated with liquefaction, which include: flow failures, lateral spreads, loss of bearing capacity, subsidence, buoyancy effects, and ground oscillation [Committee on Gas and Liquid Fuel Lifelines, 1984]. Seed [1968; 1973] also mentions another failure mode, that of landsliding caused by the liquefaction of an underlying sand seam. Flow failures are the most devastating of the ground failures associated with liquefaction. They generally occur in saturated loose sands with ground slopes ranging between 10 to 20 degrees [Youd, 1978]. During flow failures, large amounts of material may flow many tens of meters at relatively high speeds of tens of km/h.

Lateral spreads are one of the most common types of ground mass deformation associated with liquefaction during earthquakes. This failure mode is defined as the horizontal displacement of surficial deposits resulting from the liquefaction of an underlying soil layer. Lateral spreads normally develop on very gentle slopes between 0.3 and 3 degrees, and involve displacements often ranging from one to two meters.

Lateral spreading can be very disruptive to pipeline systems. Their relatively common occurrence during earthquakes means that pipelines which traverse large areas have a high potential for encountering this type of failure mode. Because lateral spreading involves the movement of competent soil, full passive soil pressure can be mobilized against a buried pipeline. Additionally, the locations of lateral spreads can be difficult to identify, since they develop on slopes as gentle as 0.3 degrees. Because of their special significance to pipeline siting and design, emphasis is given in this work to lateral spreading.

Loss of bearing capacity generally is not important for buried pipelines. Differential settlements as a consequence of volume loss caused by liquefaction may impose large deformations on underground structures. When soil deposits liquefy, buried structures tend to rise buoyantly. When large areas liquefy, buoyancy may not cause damage to a pipeline if it is composed of ductile material. Pipelines are generally buried at shallow depth so that maximum vertical deformation from buoyancy is limited to about one meter.

5.2 CASE STUDIES OF LATERAL SPREADING

Several historical earthquakes in which lateral spreads have been documented were reviewed in this work; these include the 1906 San Francisco earthquake, the 1964 Alaska earthquake, the 1971 San Fernando earthquake, and the 1983 Nihonkai-Chubu earthquake. Ground movement patterns were noted in the case studies, and damage to lifeline systems was correlated with the ground displacements.

Several observations can be made from these case studies. To start, there was a close relationship between the geologic and morphologic characteristics of a region and the damage to lifeline structures. McCulloch and Bonilla [1970] recognized that damage to the railroad system during the 1964 Alaska earthquake could be related to a few specific geologic units, such as tidal flats, active flood plains, alluvial fans, deltas, and to a lesser degree, inactive flood plains. It was suggested that, by mapping these deposits, a means could be established for predicting the type of ground displacements and severity of resulting damage during large earthquakes under similar conditions.

This same observation can be made for the 1906 San Francisco earthquake, in which the damage from lateral spreading to bridges, roadways, structures and pipelines, was extensive. Lateral spreading was most common in flood plains, tidal marshes, and in certain filled areas within the City of San Francisco. The largest lateral spreading associated with this earthquake occurred in the mud flats at Tomales Bay, where horizontal displacements of 7.6 m were reported.

Two other case studies, the 1971 San Fernando and the 1983 Nihonkai-Chubu earthquakes, provide additional information about damage to pipelines associated with lateral spreading. During the 1971 San Fernando earthquake, a 1.2-km-long lateral spread developed to the northeast of the Upper Van Norman Reservoir in an alluvial fan deposit. Four steel pipelines crossed the path of the slide. Two natural gas pipelines of 660-mm and 324-mm-diameter failed at a location where approximately 1.75 m of soil movement occurred over a width of roughly 340 m. These pipelines were oriented almost perpendicularly to the direction of flow. A 1370-mm-diameter water pipeline, which was subjected to a maximum of 0.7 m displacement distributed over roughly 380 m, was only slightly damaged. At one location, where this pipeline was connected to a concrete vault, the mechanical joint was severely battered.

The pattern of horizontal displacements associated with the San Fernando slide were strongly influenced by the topography of the area. The vectors of soil displacement show that the flow followed the slope of the lower valley, remaining perpendicular to the elevation contour lines.

The case study of the 1983 Nihonkai-Chubu earthquake shows a high degree of correlation between pipeline break locations and areas of lateral spreading. Prominent lateral spreading occurred on the slopes of two sandy hills, where the ground water table was within a few meters of the surface. The maximum soil displacement reported at these locations was approximately 5.0 m.

5.3 1906 SAN FRANCISCO EARTHQUAKE

An in-depth case study was performed in this work of the liquefaction-induced ground movements within the City of San Francisco during the 1906 earthquake. The 1906 San Francisco earthquake was chosen for study not only because of the extensive documentation available, but also because the lateral spreads associated with this earthquake contributed in a substantial way to the disruption of the water supply system in the downtown and business districts of San Francisco. The loss of water increased the difficulties in controlling and extinguishing fires, resulting in the destruction of over 490 city blocks, and the partial destruction of an additional 32 blocks.

Zones of high earthquake damage, associated with a Modified Mercalli Intensity of IX to X, which were identified in other studies [Youd and Hoose, 1978; Lawson, et al., 1908] were used as a focus for this study. These zones are associated with regions of large permanent ground movement. In this study, the zones of soil movements were studied by first plotting original topographical features, as determined by the 1853 U.S. Coast Survey of the City of San Francisco and Its Vicinity, relative to the street layout at the time of the earthquake. The locations of large surface displacements were then plotted relative to the existing street system. The location, magnitude, and distribution of surface movements were determined by reference to historical accounts, and to the extensive photographic record of San Francisco after the earthquake. The interpretation of ground movement patterns was aided by reference to a map of pipeline breaks caused by the earthquake [Schussler, 1906; Manson, 1908].

The maps of ground displacement and pipeline breaks show a close relationship with the topographic and geologic characteristics of the surficial deposits. A close relationship is seen between locations of high earthquake damage intensity and areas of fill. All of the ground displacements fall within certain zones where former marshes, gullies, streams, and bays had been filled to prepare for development. Approximately 80% of all pipeline breaks to the south of Market Street occurred within the 12-m elevation contour line of the buried topography, bordering former marsh areas.

The soil displacement pattern was closely related to the buried original topography, with the direction and magnitude of lateral spreading controlled, in large measure, by morphological details. As an example of the influence of underlying topography, at 19th and Guerrero Streets, the flow direction changed through 90 degrees as movements were canalized by the course of a buried ravine. The largest displacements occurred in areas where the contour lines of the buried topography converged, indicating a narrowing of the valley or ravine. In these areas, movements were restricted to a relatively narrow zone.

There was also a close correspondence between areas of lateral spreading and subsidence. Large areas of settlement, some of which resembled sinkholes, developed at the same locations as large lateral displacements. This implies a complex failure mode, in which lateral movements follow a downslope course at the same time as volumetric loss in the underlying soil results in caving-type distortions, with prominent surface depressions.

The distribution of pipeline breaks along and slightly removed from the margins of the previous marsh areas is indicative of another important phenomenon. As liquefaction led to the lateral spreading and consolidation of the underlying sediments in the buried marsh, there was a tendency for soil displacements to converge toward the center of the filled area. This lateral extension of the soil induces subsidence scarps and graben-type features. These movements are consistent with the occurrence of surface depressions. The inward displacement relieves the horizontal restraint against the margins, causing further slumping of the fill. The marginal slumping effectively extends the zone of influence of soil liquefaction, and results in a zone of pipeline breaks more extensive than one circumscribed by the border of the former marshes.

By comparing the locations of pipeline breaks, patterns of ground displacement, and features of the former topography, areas of potentially large ground movements in San Francisco were mapped. The mapping of pipeline breaks and buried topography relative to the mapping of ground displacements allowed for greater detail in the demarcation of these zones than has been available through previous studies.

5.4 PARAMETRIC STUDY

A parametric study was performed to evaluate the response of a steel pipeline to a distributed ground displacement pattern associated with lateral spreading typical of those seen in the case studies. The pipeline response was evaluated as a function of maximum lateral spread displacement, the width of the slide over which the maximum displacement develops, and the strength and density of the soil in which the pipeline was embedded.

The analysis was performed using a computer code, UNIPIP, developed at Cornell University. The soil/pipeline interaction model used in the computer code evaluates the elasto-plastic behavior of both the soil and pipeline material beyond yield, well into the plastic range.

In the analysis, a 610-mm continuous girth welded pipeline, composed of X-60 grade steel, was subjected to a symmetric, horizontal, distributed soil displacement pattern, with a maximum displacement of 1.5 m. The pipeline was assumed to be buried in competent granular soil at a depth of 1.2 m to the top of the pipe. Five trials were run to test the effects that different soil properties and geometric characteristics of lateral spreads have on the pipeline response. The width of the lateral spread was varied from 10 to 50 m, while all other parameters were held constant. The effects of soil density and shear strength then were tested by varying the soil properties and leaving the slide width constant at 30 m.

Several important insights regarding pipeline response to distributed soil displacements were derived from the parametric study. The geometric characteristics of lateral spreads were shown to have a greater influence on the magnitude of tensile strains induced in the pipeline than did the soil strength and density. In changing the width from 10 to 50 m, the tensile strains induced in the pipeline decreased by a factor of 6.5. In contrast, only a 1.3-fold decrease in tensile strain was produced by decreasing the soil density from 20.4 to 18.8 kN/m^3, and the soil friction angle from 45 to 35 degrees. An almost bilinear relationship was shown between the pipeline strains and the maximum imposed soil displacements, which is a function of the maximum soil resistance. Once plastic flow of the soil was initiated, no additional force was imparted from the soil to the pipeline, which resulted in a steady but small increase in tensile strain, and a similar decrease in compressive strain. The increase in bending strains in the pipeline at this point had ceased, but axial elongation continued as long as soil displacements continued.

5.5 OVERVIEW

The findings of this study have several implications for lifeline earthquake engineering. Investigations of the exposed and buried morphological features of a region can be useful for identifying locations of potential permanent ground displacements, perhaps even to the point of predicting the pattern and approximate magnitude of movements. Since pipeline response to distributed ground movement is influenced more heavily by the geometric characteristics of the soil displacements, as shown in the parametric study, the ability to estimate soil displacement patterns is of paramount importance in predicting pipeline deformation. The geologic conditions of the site will indicate potentially liquefiable zones, while morphologic information will provide insight into the deformation patterns. The identification of zones of potentially large ground movements and the estimation of displacement patterns can be used to limit system failures through improved siting and design of future pipelines and the modification of existing ones.

SECTION 6

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APPENDIX A

Tables A.1 and A.2 are a compilation of the historical accounts of ground movements in San Francisco during the 1906 earthquake, as well as observations from photographs, which were used to prepare the maps of ground movements in Chapter 3. References for the two tables are given at the end of Table A.2, on page A-53.

TABLE A.1 DOCUMENTATION OF HISTORICAL ACCOUNTS AND OBSERVATIONS FROM PHOTOGRAPHS ON GROUND MOVEMENTS IN SAN FRANCISCO DURING 1906 EARTHQUAKE.

MISSION CREEK ZONE:

LOCATION	REF*	PHOTO	PAGE	DESCRIPTION
General:	3		238	"As stated briefly above, a similar district of high intensity occurs in an area of made land along the lower portion of the former course of Mission Creek. This district varies in width from 1 to 2 blocks, extending from Near the corner of Ninth and Brannan Streets westward for about 3 blocks, then southwestward for about 2 blocks more; and finally, westward some 4 blocks more to a point on Nineteenth Street just east of Dolores Street.
	3		239	...Enough evidence has been cited to demonstrate that high intensity prevailed thruout this district. Here, as in the other tract of made land which occupies the site of the old tidal marsh, the materials used for filling were shaken together, and caused a general depression of the surface over the whole district, accompanied by slumping or flow movements. The surface was deformed into waves, with accompanying fissures and sharp compressional arches. Here too, as in the tract previously described, the materials used for filling constitute a relatively thin rigid layer deposited upon the marshy fringes or in the shallow waters of the creek."
	4		26	"As in districts outside of San Francisco, the greatest damage was done to those structures having insufficient foundations built on soft alluvium or filled ground. The settling of the ground in the mud flats along San Francisco Bay and of the filled ground in old water courses was accompanied with great destruction. It was in such ground that the greatest number of breaks occurred in the cast-iron gas and water mains and the sewers. The breaks in the sewers were not so evident as those in the gas and water mains, for the reason that the latter were under pressure and breaks in them resulted in breaks from the settling of soft or filled ground occurred in Howard and Shotwell streets between Seventeenth and Eighteenth streets, Bryant street between Ninth and Tenth streets, Dore street between Bryant and Brannan streets (Pl. VI, A), and at the corner of Seventh and Mission streets. The settling was greatest in Howard, Dore and Bryant streets, being in Dore street at least 5 feet [1.5 m]."
	8b		739	"...In the Mission district, and in certain other parts of San Francisco, as, for instance, along the courses of former tidal streams, large areas of filled land exist.

* References listed at end of Tables 3.1 and 3.2.

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				In such cases, the effect produced by the earthquake was not generally in the form of waves, as already described, for the eastern portion of the city was basin-like, representing local settlement in the streets and adjacent areas, and in some cases, decided misalignment.... The most important local settlements and transverse movements occurred in the vicinity of the Valencia Hotel between 18th and 19th Sts., on Valencia St., on Howard St. between 17th and 18th; on 14th, between Mission and Howard; on Folsom, at the corner of 17th; on Mission, at the corner of 7th, and on Van Ness Ave., between Vallejo and Green Sts. Where the ground was hilly and solid, it was not decidedly affected by the earthquake shock. The distortion of the streets evidently has no relation to the character and nature of the artificial surface. For, of whatever construction this may have been, it is very evident that the street surface would follow the profile and alignment of the ground upon which it rested."
1) 19th near Guerrero	13		32	"Again, at its head between Valencia and Guerrero a side ravine came down from the south. The slip across Nineteenth street of about six and a half feet [2.0 m], as shown by bowing of the line of curbing, outlines the branch of the old depression, which was filled on steep gradient and quite naturally slid easily."
	3	pl. 94A		Photo: 1.8 m lateral spread over a distance of 38 m; 0.6 m sink on north side of 19th St. at western edge of disturbance. Lateral offset of roughly 0.3 m seen at western edge of lateral offset in foreground running across 19th. Photo caption: "View along Nineteenth Street, from Guerrero Street. Both ground and buildings moved north about 6 feet [1.8 m] toward center of old marsh, with component of movement down the channel."
	2	Fig. 50	55	Photo: Same as ref. 3 pl. 94A. Photo caption: "Lateral spread at the Youth's Directory on 19th and Guerrero Streets (loc. 217)."
	1	No. 69	97	Photo caption: "19th St. Looking West. Street & Sidewalk from A to B moved north about 5 feet [1.5m]. Brick building also moved somewhat."
	3		239	"On land made by filling in, "The Willows," a marshy tract formerly extending up the Eighteenth

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				Street from Mission near the corner of Nineteenth and Guerrero Streets, there was observed a considerable slumping or flow movement of the surface. The photograph (plate 94A) shows the Youth's Directory, a charitable institution for boys, where the street and building were moved northward and slightly eastward, toward the former channel and downstream, fully 6 feet [1.8 m]."
2) Valencia between 18th and 19th	13		32	"Mission creek estuary headed in a salt lagoon between the present lines of Howard, Harrison, Sixteenth and Nineteenth streets, extended north-west nearly to Eleventh and eastward to Mission bay at a point about on the line of King street, between Eighth and Ninth." "VALENCIA-STREET SUBSI-DENCE. A number of lesser footmarks of our earth-quake are to be seen in the streets between Seventh [17th?] and Eighteenth and east and south of Mission. These all coincide with the irregular limits of our indicated specially soft spots in the former salt marsh area, and we follow them around until we come to another pronounced case of subsidence and slip, which is noticeable in Valencia street between Eighteenth and Nineteenth, in Mission between Seventeenth and Eighteenth and in Howard from Seventeenth to Eigh-teenth. Valencia street shows subsidence of one to five and a half feet [0.3 to 1.7 m] and slip of one to six feet [0.3 to 1.8 m] for 450 feet [140 m] in length; Mission shows very slight subsidence and slip of only about one foot [0.3 m] for about 400 feet [120 m] in length, while Howard shows a subsidence of two to three feet [0.6 to 0.9 m] for over 500 feet [150 m] in length and a maximum slip of about four feet [1.2m]. The limits of this movement, platted on the map, exactly outline the ravine in which was formerly located, between Mission and Valencia, "The Willows," San Francisco's place of resort. The whole area was in those days moist land, with a little stream in it for a part of each year, and which has been filled upon between hard hillsides. Being soft beneath and on a down-grade in the line of greatest earthquake vibration, the temblor found in it an easy mark for a pronounced demon-stration."
	1		35	"One of the most serious breaks in the main pipe lines was caused by the earthquake shaking and settling down, by from one to five feet [0.3 to 1.5 m], the region between Eighteenth and Nineteenth streets, on Valencia

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				street. (See Photos 57, 58, 59, 63, 64, and 65.) Here an old swamp had been loosely filled in, many years ago, by any and all kinds of material and rubbish obtainable, the fill being twenty feet or more in depth. Our pipes, which had to be below the pavement of the street, had to cross this region. Being aware of this state of affairs regarding the character of the foundation and fill (I constructed the pipe in 1876), I remembered the location of this swamp, and made our 22-inch pipe of wrought-iron. I put in a number of cast-iron bell joints, with lead joints, which would give or yield somewhat in case of a slight settlement. This pipe is to-day in a perfect condition, except at the points where the swamp dropped down suddenly, during the earthquake, from four to five feet, [1.2 to 1.5 m] which naturally tore off the pipe both at the north and south boundary of the swamp. This serious break was quickly repaired by laying across the swamp and over the top of the pavement and well into terra firma on each side of the sink, a long stretch of 24-inch cast-iron pipe. This gave us a chance to drive a supply of water along Market to Sansome street, to Montgomery avenue, etc., to and into the Francisco Street Reservoir."
	1		30	"By 7 o'clock on the evening of April 20, or sixty-two hours after the earthquake, a second stream of water was pouring into its respective city reservoir, the College Hill Reservoir. This reservoir had been emptied of its contents of 11,400,000 gallons [43.1 x 10^6 liters] which it contained at 7 A.M. April 18, by its main arterial pipe, 22-inch diameter, and its companion pipe, 16-inch diameter, both on Valencia street, having both been torn off and destroyed between Eighteenth and Nineteenth streets by the sinking of Valencia street of from one to five feet [0.3 to 1.5 m]."
	1		43	"APPENDIX D. The following are extracts from the report of City Engineer Woodward on the breaks in the San Francisco sewer system caused by the earthquake as published in the "San Francisco Chronicle" of June 17, 1906: '...On Valencia street, between Eighteenth and Nineteenth streets, there was a lateral movement to the east, with a maximum of six feet [1.8 m] and a subsidence with a maximum of five feet [1.5 m]. This occurred in made ground over the old Willows marsh, one of the tributaries of Mission creek."
	1	No. 57	93	Photo caption: "Valencia St. between 18th & 19th

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				subsided about five feet [1.5 m]; destroying sewers, and besides gas & electric pipes & conduits, tore of one 16" and on 22" water main, which at this point had about 85 lbs pressure."
	1	No. 58	93	Photo: View south (taken from just north of 18th St.); hole in street seen in distance. Shows destruction of area by fire. Photo caption: "View south. Showing 24" [0.6m] and 16" [0.4 m] pipes quickly replaced on Val. St (see 57) on top pavement of Sunken Street."
	1	No. 59	93	Photo: View south; photo taken from south of 18th St. Hole in street in center foreground. Photo caption: "Valencia St. Sunken portion of Str. showing two above emergency pipes on top of pavement; also 5' [1.5 m] sink with car tracks and broken sewer at A."
	1	No. 63	95	Photo: Shows collapse of hotel, (opposite side of street from Valencia hotel?), street sunken, large hole in street. Photo caption: "Valencia Str. near 19th subsided (see No. 59) three Story Hotel collapsed on the left. Few minutes after picture was taken, the fire swept it clean."
	1	No. 64	95	Photo: View northward. Shows close up of hole in street with the destroyed Valencia Hotel just to the north of hole. Photo taken before fire. Note caption reads that this is same hotel as in No. 63 which is unlikely. Hotel of Photo No. 63 is most likely across the street. Photo caption: "Other view of same [as No. 63], showing large hole in sunken street; also water pipe at x."
	1	No. 65	95	Photo: Shows 18th as the northern edge of disturbance. Shows lateral movement of 2.1 m over at least 9 m (only see half of lateral spread in photo). Photo caption: "View taken on Valencia Str looking north (A-B-C crossing of 18th Str.) showing lateral movement of Str. to East. B-B1 original position of center of west car track. B-B11 being present position since earthquake. Lateral movement fully 7 feet [2.1 m], B1-B11."
	2	Fig. 48A	54	Photo: View northward, most likely between 18th and 19th, near 18th St. (Note discrepancy with photo caption).

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				Photo caption: "Valencia Street between 17th and 18th Streets (loc. 216). A. View northward shortly after earthquake showing collapsed Valencia Street Hotel in which tens of people were killed. Note lateral displacement of street in front of hotel.
	2	Fig. 48B	54	B. View southward after fire showing lateral and vertical displacements of 6 ft. (1.8 m) and two temporarily repaired arterial water pipelines that were ruptured by the ground movements, cutting off the water supply to a major part of the city."
				Photo: B. View southward from near 18th at the margin of the zone of deformation. Car tracks clearly show the form of the lateral spreading.
	3		239	"Sewers and water-mains were broken. At Eighteenth and Valencia Streets there was a serious break in the water-pipe. Here, on both sides of the street, the ground sank about 6 feet [1.8 m], causing the roadway to arch in a very noticeable way. Ten-inch [0.25 m] car rails were bowed up into arches from 24 to 30 inches [0.6 to 0.8 m] in height. The Valencia Street Hotel collapsed so that occupants of the fourth story could step out into the street. Casualties in

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				this district can never be known accurately, owing to the immediate onset of the fire, and the complete devastation it produced."
	3	pl. 93B		Photo: View southward. Shows hole in pavement, destruction by fire.
				Photo caption: "Valencia Street, near Eighteenth. Land in this neighborhood sank about 6 feet [1.8 m], flexing street surface."
	4		8	"Where the same earth flow crossed Valencia street the horizontal movement amounted to 6 feet [1.8 m]."
	5		214	"In the rocky portion of San Francisco the sewers were not affected. In portions where the rock was overlaid with sand, there were no permanent displacements except where the original ground supported a fill; in such areas settlements occurred, and the sewers were destroyed. In filled-in tidal areas, marsh-lands and swamps there was considerable movement in a number of places (the greatest near 16th St. [18th?], and Valencia St., where the settlement was 5 ft. [1.5 m] and the lateral movement 6 ft. [1.8 m]) and in all such disturbed areas the sewers were destroyed."

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
	5	PlXLIX Fig.2		Photo: Similar to that of ref. 1, No. 58. View southward. Intersection of 18th in near foreground (near 2nd rubble pile). Photo caption: "Displacement of Ground at Eighteenth and Valencia Streets, San Francisco."
	5		253	"Plate XLIX, gives some idea of the extent of this movement, both laterally and vertically, at Valencia and 18th Streets, where one of the most serious breaks occurred. It also shows the pipes laid temporarily on the surface to replace those parted underground."
	8b		739	"The most important and interesting case of settlement and throwing out of alignment of streets in San Francisco, due to the vibrations of the temblor, was that of Valencia St., between 18th and 19th Sts., in the Old Mission district. At this point there was formerly a tidal stream, known as Mission Creek, whose existence had long since ceased owing to the fact that its course had been filled in and the land so reclaimed had become thickly built up. The earthquake caused a settlement of from 6 to 8 ft. [1.8 to 2.4 m] for a distance of from 150 to 200 ft. [46 to 61 m] along this street, and at the same time shifted the entire street, with adjacent lands, eastward through a maximum distance of 9 to 10 ft [2.7 to 3.0 m]. This change in alignment and grade could, of course, mean nothing less than the entire destruction of all water and gas mains, electric lighting and telephone conduits, sewers, cable conduits, railroad tracks, etc. The breakage of these important lines, especially to the water mains, was of greatest significance. In this case the destruction of the water pipes, of which a 28-in. and a 16-in. were found in this street at this point, meant the cutting off of a large part of the water supply of the portion of the city which was soon to be in flames."
	8b		739	Photo: View similar to that of ref. 1, No. 59 from a little farther south. Hole in street in center distance, behind pedestrians. Clear view of settlement in street. Photo caption: "Temporary Main on Valencia Street... The second picture shows a 24-in. cast-iron pipe line hastily placed in Valencia St. to provide water for the higher districts, whose supply had been cut off by the destruction of the mains in this street between Eighteenth and Nineteenth Sts. A 16-in. pipe to supply the lower downtown districts was also placed here in an incredibly short time."

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
	8b		739	"The most important local settlements and transverse movements occurred in the vicinity of the Valencia Hotel between 18th and 19th Sts., on Valencia St., on Howard St. between 17th and 18th; on 14th, between Mission and Howard; on Folsom, at the corner of 17th; on Mission, at the corner of 7th, and on Van Ness Ave., between Vallejo and Green Sts.."
	8c	---	765	"Destruction of Sewer by Settlement of Street." "In the second picture, taken on Valencia St., between Eighteenth and Nineteenth Sts., the broken sewer is in a deep fill along the former course of Mission Creek."
	8c		767	"Probably at no point were more serious results produced than on Valencia St., between 18th and 19th Sts., already described in connection with street and sewer problems. Two very important distribution mains were located at this point and were, of course, ruptured. One of these pipes, a 22 in. in diameter, supplied the higher districts of the city; the other, a 16 in. pipe, was an important artery of the system furnishing water to the business section."
	9c	---	581	"STREET SUBSIDENCE IN SAN FRANCISCO. VIEW ON VALENCIA ST., NEAR 18TH ST., OPPOSITE SITE OF
				VALENCIA HOTEL. (It was at this place that street water mains were broken. The street dropped about 4 ft. [1.2 m] and moved eastward about 6 ft. [1.8 m] at the maximum point.."
	9b		551	"The Valencia Hotel, it will be remembered, was situated on Valencia St., near 18th, on filled ground, where once ran the old Mission or Islais Creek [the former Islais Creek was located farther south]. The Valencia Hotel and other cheap brick and frame buildings in that region from Valencia to Howard Sts. very generally collapsed. Many lives were lost in the Valencia Hotel."
	20		589	"The most remarkable instance of this kind is that of the Valencia Hotel, at the upper end of that street. It stood upon a deep filling of mud and sand. During the shake it dropped suddenly many feet into the earth, one side much deeper than the other, and doubled together like a jack-knife, the upper stories lying upon the lower."
3) Four block area between 17th and 18th from Valencia to Folsom	3		239	"Again, along the creek bed from Folsom Street, between Seventeenth and Eighteenth Streets, to the vicinity of Valencia Street at Eighteenth, great destruction was conspicuously prevalent. Less than a third of the

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				frame dwellings in this tract retained their vertical positions, and a few collapsed completely. Others remained standing only by leaning against each other. The south side of Howard Street, between Seventeenth and Eighteenth Streets, which escaped the fire, furnishes a good illustration of the damage produced here. (See Plate 93A.) As in other places, the streets were deprest, fissured, and thrown into waves. (Plate 90C.) Car rails were arched and bent laterally in a violent fashion. (Plate 92B.)"
(Capp St.)	1	No. 61	95	Photo: Looking south from near 17th toward 18th. Sidewalk on east side subsided 0.6 to 0.9 m and moved laterally eastward, about 0.6 to 1.2 m over 120 m, separating sidewalk slabs. General sink in southern portion of block from just west of the street eastward beyond photo. This depression has a scarp along the southern margin near 18th and another running diagonally about 30 m north of 18th at the northern margin of the sink. Photo caption: "Capp Str & Sidewalk sunken."
	2	Fig. 46	53	Photo: Same as that of ref. 1, No. 61. Photo caption: "Scarps, lateral and vertical displacements in Capp Street between 17th and 18th Streets (loc. 215)."
	2	Fig. 47	53	Photo: View eastward on 18th St. from the intersection of Capp St. Compression ridge running E-W through the intersection of Capp St. at the line of the northern edge of the north sidewalk on 18th St. Curbstone arched at this location (roughly 0.12 strain). Fire destroyed the entire block between Capp and Howard Sts. Howard St. seen in the distance, with houses leaning appreciably. Photo caption: "Buckled curbstone on Capp Street near 18th Street (loc. 215). Buckling was caused by sediments shifting toward old channel of Mission Creek."
(Howard St.)	1	No. 66	97	Photo: View looking southwestward across the intersection, from 17th down Howard. Shows general depression of 0.3 to 0.6 m. Photo caption: "Crossing of 17th & Howard. Street sunken, 12" & 20" pipes ruptured."
	1	No. 70	97	Photo caption: "Crossing 17th & Howard St 16" pipe badly fractured. (not yet repaired)"
	2	Fig. 43	55	Photo: View looking northward on Howard.

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				Compression ridge runs diagonally from SW-NE. Buckling of car tracks (0.12 m compression). Photo caption: "Buckling of rails by compression on Howard Street (south Van Ness Avenue) near 17th Street (loc. 215). "
	3	Pl. 92B		Photo: Same as ref. 2, Fig. 43. Note view can not be south, as said in caption, because houses on west side of Howard were destroyed by the fire. Photo caption: "Looking south on Howard Street from near Seventeenth Street. Compressional flexure of car rails."
	4	Pl. VIB		Photo: Same as ref. 2, Fig. 43. Photo caption: "BUCKLING CAUSED BY EARTH FLOW, HOWARD STREET, SAN FRANCISCO."
	4		8	"In taking the photograph reproduced in Pl. VI, B, the camera stood on ground made by the filling of Mission Lagoon, an expansion of Mission Creek, and was pointed northward, commanding a portion of Howard street. The made ground here flowed northeastward and the buckling of street-car tracks was caused by its motion."
	8b		740	"On Howard St., at the corner of 17th St., very complete destruction of the brick sewer, as well

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				as most other pipes and conduits in the ground, was caused by the extensive settlements in this region. On Valencia St., near 18th St, similar ruptures were produced."
	1	No. 67	97	Photo: Close up of homes which were most severely tilted. Photo caption: "East Side Howard, between 17th & 18th, showing effects of Earthquake on buildings."
	1	No. 62	95	Photo: View northward down Howard from 18th. Shows lateral offset of 0.3 to 0.6 m running diagonally NW-SE. Block west of Howard destroyed by fire. Houses on east side severely tilted. Car tracks show lateral spread; roughly 1.2 m over 150 m. Photo caption: " Howard St. north of 18th sunken & twisted (see car tracks) One 20" pipe, one 24" and two 6" pipes broken by earthquake"
	2	Fig. 44	51	Photo: View very similar to that of ref. 1, No. 62. Photo caption: "Looking north on Howard Street (South Van Ness Avenue) from near 18th Street toward 17th Street, San Francisco (loc. 215). Rails offset laterally by lateral-spreading ground failure."

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	4	Pl. LIIIB		Photo: View in the same direction as that of ref. 1, No. 62. Closer view of tilting houses. Shows general depression in front of houses. Photo caption: "GENERAL EARTHQUAKE EFFECT ON FRAME BUILDINGS SITUATED ON ALLUVIAL SOIL. HOWARD STREET, SAN FRANCISCO."
	5	Pl. 36 Fig. 1		Photo: View similar to that of ref. 1, No. 62. Photo caption: "Extreme Case of Earthquake Damage to Frame Buildings in San Francisco. (In Center.)"
	6		18	Photo: Top photo shows close up of tilted houses on east side of Howard St. The lower photo shows general depressions down Howard St. which give an impression of slight waves. Photo caption: "RESIDENCES. Howard Street near Seventeenth Street. The upper view shows two buildings that have been tilted badly out of plumb by the earthquake. The rear portions of the buildings settled about 10 ft [3.0 m]. The telegraph poles are approximately plumb. Note the building on the right, the corner of which has broken into the side of the tilted one. The lower view shows Howard Street, looking west. The front of the two buildings shown in the upper view are approximately in the centre. One building near the left-hand side has tilted away from the street line until it almost disappears."
	8a	---	703	Photo: Close up of tilted houses from directly in front. Depression - at least 0.3 to 0.6 m difference of ground surface in vicinity directly in front of buildings. Photo caption: "Houses on Soft Ground."
	9b	Fig. 11	550	Photo: Similar to that of ref. 8a, p. 703. Photo caption: "THREE WRECKED FRAME HOUSES ON FILLED GROUND, HOWARD ST., BETWEEN 17TH AND 18TH."
	9b		553	"Fig. 11 shows frame houses on Howard St., between 17th and 18th, on filled ground. The house at the extreme left has entirely collapsed."
	1	No. 68	97	Photo: View eastward on 18th from intersection of Howard. Subsidence scarp runs E-W along centerline of 18th. Settlement to the north, extends at least as far east as Shotwell St. Photo caption: "Along 18th from Howard. Street subsided; pipe broken."
	2	Fig. 45	52	Photo: Subsidence and settlement has pulled apart north sidewalk slabs. Southern side of

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				18th is the boundary of disturbed zone. Photo caption: "Scarps showing vertical movement and northward lateral movement (loc. 215). View eastward on 18th Street. Intersection of Howard Street (South Van Ness Avenue) is in the middle-ground."
	3	Pl. 93A		Photo: View northeast from 18th St. across towards east side of Howard St. Shows destruction by fire of block west of Howard; tilting of houses. Photo caption: "East side of Howard Street, between Seventeenth and Eighteenth Streets."
	3	Pl. 90C		Photo: View eastward along 18th from intersection of Shotwell. Shows subsidence to north along centerline of 18th, down 0.3 to 0.6 m. Length of disturbance appears to be at least 100 m, perhaps all way up to Folsom. Photo caption: "Eighteenth Street, just east of Shotwell. Fissuring and depression of pavement."
4) 14th St. near Howard	4		739	"The most important local settlements and transverse movements occurred ..., on 14th, between Mission and Howard...."
	1	No. 63	95	Photo caption: "14th St east of Valencia, Street torn open See water pipe below."
	1	No. 72	97	Photo caption: "14th St near Valencia. Sewer ruptured. 8" pipe broken. same as No. 63; picture taken from slightly changed point."
	8b	---	737	Photo: View westward on 14th? Appears to have some settlement to the south. Photo caption: "Effect of Broken Sewer on Fourteenth Street. The second picture shows the result of the destruction of a sewer on Fourteenth St., near Howard St. The crown of the sewer was broken in, the surface material washed in, as at A, and the sewer so completely filled with sand as to back up the sewage, as at B, above the elevation of the crown."
5) Block bounded by 9th, 10th, Bryant, and Brannan	3		238	Mission Creek was formerly a sinuous tidal stream, with narrow fringes of salt marsh about its banks. Near its mouth the stream wound around a rocky point where the serpentine hills of the Potrero rose abruptly from its southern bank. Here, along its margin, is found the most sudden transition from high to low intensity that is anywhere encoun-

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			239	tered in the city. Along Dore Street, a narrow alley running from Bryant Street to Brannan Street, between Ninth and Tenth Streets, the street pavement was broken into a series of waves. The photographs, plate 89D, looking along Dore Street from Bryant toward Brannan Street; plate 90A, looking from Brannan Street in the reverse direction; and 90B, showing in detail the trough of one of these waves, with the fissuring of the pavement near the farther crest, indicate more clearly that words the great intensity manifested here. Less than 2 blocks south on the hill slopes, more than 50 per cent of the chimneys were left standing, and no serious structural damage was noted. No comment seems needed to establish clearly the fact that the change in the character of the ground, this being the only variable factor, is in some way the cause of the change in the degree of intensity.
				On Ninth Street, east of Dore Street, between Bryant and Brannan Streets, the block pavement was badly damaged by fissuring, slumping, and the formation of surface waves. Frame dwellings were thrown from their underpinning, and a few collapsed. Plate 91A shows a wave trough near Bryant Street, with the resulting disturbance of the pavement. The

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				dwellings immediately in the trough have dropt from their foundation posts. In plate 91B, looking along Ninth Street from near Brannan Street, is shown the depression and fissuring of the street and its slumping or flow westward toward the former channel of a short branch of Mission Creek, which occupied the present location of Dore Street. Streets, curbing, car tracks, etc., are defected from 6 to 8 feet [1.8 to 2.4 m] from their former positions. The frame dwellings were not destroyed, but a careful examination of the picture will show that most of them are badly injured. Many were left in a dangerous condition by the shock.
				On Tenth Street, between Bryant and Brannan Streets, less violence was noted and the slumping of flow eastward (toward the channel of the little branch of Mission Creek) is scarcely noticeable."
(Dore St.)	2	Fig. 38A	45	Photo: View of whole length of Dore St. In center foreground is a manhole cover which has punched through the pavement. Shows lateral offset, of roughly 0.13 m, near north end of street, marking the northern margin of disturbed zone; four separate depressions, or troughs along length of street; two lateral offsets, of roughly 0.3 m,

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				in southern section of street; most of the houses are severely tilted. Photo caption: "View along Dore Street from Bryant Street toward Brannan Street, San Francisco (loc. 214). A. Photograph after the 1906 earthquake showing undulations as large as 6 ft (1.8 m) in street. As much as 6 ft (1.8 m) of lateral movement also occurred at this location. B. Dore Street today (September 1974) from approximately the same location as fig. 38A, showing ramps of the James Lick and Central Freeways and other structures constructed since 1906. C. Building at corner of Bryant and Dore Streets damaged by differential vertical and lateral movements."
	3	Pl. 89D		Photo: Similar to that of ref 2, Fig. 38A; taken from a little farther north, nearer Bryant. Northern offset shows more clearly, as well as lateral spread. North of manhole, street shows little deformation. Photo caption: "Looking along Dore Street, from Bryant toward Brannan. Undulating and fractured condition of pavement due to earthquake. Houses thrown off their underpinning and pitched out of the vertical."

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	3	Pl. 90A		Photo: View northward up Dore St. The southern 10 m or so of Dore (near Brannan) is not disturbed. Lateral movement is eastward. Photo caption: "Looking along Dore Street from Brannan toward Bryant. Larger undulations near Brannan. Dore Street is on site of an arm of Mission Creek."
	3	Pl. 90B		Photo: View southward on Dore St. Close up view of most southern trough. Shows lateral offset, of roughly 0.3 m, more clearly. Photo caption: "Dore Street, near Brannan. Vertical difference between crest and trough of undulations, 5 feet [1.5 m]."
	4	PL. VIA		Photo: View southward on Dore St. from middle of block. Close up of some of the most severely tilted houses on the east side of the block. Shows more clearly the break up and displacement of the western sidewalk pavement. Photo caption: "SETTLING (5 FEET) [1.5 m] ON DORE STREET, BETWEEN BRYANT AND BRANNAN STREETS, SAN FRANCISCO."
	4		9	" Another example of the effect on the filled-in land in this part of the city is shown in Pl. VI, A, a view of Dore street

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				between Bryant and Brannan streets. The settling of the soft ground caused the street to drop at least 5 feet [1.5 m] at this place.)"
	4		26	"The most noticeable destruction resulting from the settling of soft or filled ground occurred in Howard and Shotwell streets between Seventeenth and Eighteenth streets, Bryant street between Ninth and Tenth streets, Dore street between Bryant and Brannan streets (Pl. VI, A), and the corner of Seventh and Mission streets. The settling was greatest in Howard, Dore, and Bryant streets, being in Dore street at least 5 feet [1.5 m]."
(Bryant)	2	Fig. 39A	47	Photo: View almost directly east on Bryant. Shows scarp, roughly 0.15 m vertical offset, running diagonally across street in NW-SE direction; subsidence is to the south. Photo caption: "Scarps and right-lateral displacements caused by lateral spreading at two points on Bryant Street near the intersection of Ninth Street (loc. 214). A. Between Ninth and Tenth Streets (?). B. Between Eighth and Ninth Streets."
(Ninth)	2	Fig. 40A	48	Photo: Shows two levels of lateral spread, the greatest movement in the southern part of the street. In the foreground is close up of a depression in 9th St. at the location of the largest lateral spread. Scarp shows roughly 0.15 m settlement. Undulations show isolated depressions of roughly 0.6 to 0.9 m settlement. Photo caption: "Views along Ninth Street between Bryant and brannan Streets (loc. 214). A. View northwestward from near Brannan Street showing lateral displacement of street, rails, curb, walk, and buildings."
	3	Pl. 91B		Photo: Similar to that of ref. 2, Fig. 40A. Taken a little farther SE on Ninth. Photo caption: " Ninth Street, between Bryant and Brannan. Westward lurching of land toward former creek channel where Dore Street now is."
	4	Pl. V		Photo: Same as that of ref. 3, Pl. 91B. Photo caption: "RESULTS OF EARTH FLOW, NINTH STREET, SAN FRANCISCO."
	4		8	"Slips of this character grade into those of wet alluvium or "made ground" resting upon gentle slopes-ground which under ordinary conditions flows or creeps at an almost imperceptible rate, but which by shaking was made

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				to move several feet or yards in a few seconds. The filled districts of San Francisco afford several examples, and two of these are illustrated by Pls. V and VI, B. The view shown in Pl. V is northwestward on Ninth street, near Brannan. Before the earthquake the car tracks and curb line were straight and approximately level, and this condition was not disturbed on the relatively firm ground shown in the distance. In the nearer part of the view the street crosses a tract of made ground created by filling a valley tributary to a narrow tidal inlet called Mission Creek. The descent of this valley was southwestward, and the made ground flowed in that direction, carrying street and buildings with it."
	4		9	"A permanent disturbance of the ground also resulted in many instances from compacting. Just as sand or grain that has been poured into a measure can be made by shaking to settle down and occupy less space, so various loose formations, and especially artificial fillings, were shaken together by the earthquake and the ground surface lowered. In such compacting the particles making up the aggregate are readjusted so as to fit more closely together and the voids are reduced. In dry formations compacted by the earthquake the

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				reduction of voids was opposed only by the elasticity of the contained air. In wet formations it encountered the effectual resistance of the contained water, and could be accomplished only by the extravasation of some of the water. Ordinarily it was impossible to measure the settling due to compacting, or even to determine its occurrence as a phenomenon independent of ground flow, but it was clearly seen in various localities in San Francisco where those parts of graded streets which retained their simple shapes and straight lines served as reference planes for neighboring parts that were disturbed. (Compare the distance and foreground of Pl. V. [Note: the photograph does show lateral spreading, as mentioned by Gilbert, et al., in the above quotation]. Another example of the effect on the filled-in land in this part of the city is shown in Pl. VI, A, a view of Dore street between Bryant and Brannan streets. The settling of the soft ground caused the street to drop at least 5 feet [1.5 m] at this place.)"
	2	Fig. 40B	48	Photo: General wreckage, can not ascertain ground displacements. Photo caption: "B. View northwestward showing

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				building damage, some of which is due to southwestward lateral displacement of the ground."
	2	Fig. 40C	49	Photo: View southward on Ninth in mid section of the block. Leaning lamp post locates the photo in relation to Fig. 40A. Close up view of slumping southwestward. House severely tilted. Photo caption: "C. Close-up view of damage in midsection of block."
	2	Fig. 40D	49	Photo: View northward of western side of block towards Bryant. Shows trough, roughly 0.6 to 0.9 m settlement, slumping toward the southwest. Photo caption: "D. Close-up of damage at northwest end of block."
	3	Pl. 91A		Photo: Same as that of ref. 2, Fig. 40D. Photo caption: "Ninth Street, between Bryant and brannan. Undulation and fissuring of pavement and sidewalks. Houses over trough have been dropt from their underpinning."
	2	Fig. 41	50	Photo: Shows scarp across Ninth Street of roughly 0.15 m, slump towards the northwest. Photo caption: "View northeastward on Brannan Street, corner of Ninth Street (loc. 214). The

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				scarp and settlement in the foreground marks the southern boundary of the ground failure on Ninth Street."

SOUTH OF MARKET ZONE:

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General:	3		236	"High intensity was developed thruout a small elongate district having a width of about two blocks, which extends from near the corner of Eighth and Mission Streets to the vicinity of Fourth and Brannan Streets; from this point the boundaries are irregular and very sinuous, leading to the waterfront at about the crossings of Third Street with Berry and Channel Streets. A glance at the geological map, No. 17, shows that the regularly bounded portion of this district corresponds very closely with the area of a former tide-marsh, drained and flooded by one or two small tidal streams. The former shore line of Mission Bay was just north of Brannan Street, between Fourth and Fifth Streets, so that the irregular seaward portion of the district lies outside the old shore. This is one of two localities in the city, the other being a "made" land tract along the former course of Mission Creek, in which destructive effects of great magnitude were conspicuously developed. Only in very close proximity to the fault was greater violence manifested. For blocks the land surface, paved streets, and building plots alike, were thrown into wave forms, trending east and west about parallel to the length of the area. The amplitude and wave-length of these earth billows, and the distances to which they extend, are indefinite and irregular. The fissuring and slumping, and the buckling of block and asphalt pavements into little anticlines and synclines (arches and hollows), accompanied by small open cracks in the earth, characterize the land surface. This amount was greatest near the center, or channel, where the street lines were shifted eastward out of their former straight courses, by amounts varying from 3 to 6 feet [0.9 to 1.8 m]. A satisfactory photograph of this phenomenon was not obtainable, owing to the quick convergence of parallel lines in perspective, but to the observer in the field it was a very striking result of the shock. The greater part of the district was occupied by wooden dwellings and shops, with a small percentage of mediocre brick buildings and a few of substantial construction. The fire swept the area clear. Not even heaps of debris remained to cover the ground, most of the destructive effects being obliterated, along with the structures in which they were developed. Enough remained, however. Foundation walls and sidewalk pavements were broken and flexed; sharp little anticlines were produced

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				in the street by the arching of block paving, as on Russ Street between Folsom and Howard Streets (plate 88C); granite curbing was broken and thrust up into an inverted V, as on Moss Street, between Folsom and Howard Streets (plate 88D): there were fissuring and slumping in the block pavement, as along Columbia Street between Folsom and Harrison Streets (plate 89A), and sharp flexures of the paved streets and car tracks, as on Sixth Street just south of Howard Street. These effects point simply and clearly to the great magnitude of the intensity thruout the greater part of this old swampy district. Attention has already been directed to the slumping or flow movement to the east along the long axis of the area. The heavily ballasted car-tracks on Bryant Street, at the crossing with Fourth Street, were sharply flexed laterally, tho bounded by block paving. (Plate 89.) This was at the eastern end of the district where the marsh formerly bent to the south around the flanks of Rincon Hill, a mass of firm sandstone rising from the floor of Mission Valley. No similar sharp flexures were encountered along east-west streets in the western or central portion of the district, tho lateral displacement

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				and flat, sinuous curvings of the street lines were common enough; notably on Harrison Street between Fifth and Sixth Streets, and on Folsom Street between Fourth and Seventh Streets. Both these streets cut across the direction of the flow movement at a small angle. These phenomena are easy to understand if, as seems certain, Rincon Hill served as a solid buttress against which the flow to the east was arrested, causing sharp crumpling of the surface near the buttress, with less disturbance farther away. This was combined with a slight tendency to flow southward in the southeastern part of the district. The shaking caused the materials used in filling to settle together and occupy less space so that the surface over the whole district was lowered by amounts varying from a few inches to 3 feet [0.9 m] or more. This is clearly seen in the change of street levels along the margin of the solid ground, where the car rails are bent downward in little monoclines. Occasionally a structure with a relatively good foundation remains at its former level, with the whole neighborhood depress about it. Such a case is exemplified on Sixth Street, a little south of Howard Street, near the margin of the area. (Plate 89C). The flow movement is

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				thought to be due simply to the action of gravity, the loose, water-soaked material being compacted into less volume by the shaking. Besides this sinking of the district, and its flow movement, mention has been made of the deformation of its surface into irregular waves, trending approximately east and west parallel with the length of the district. Along the streets running approximately north and south, at right angles to the elongation of the area, car rails were bent abruptly to the side, or raised in arches, and sharp anticlines were formed in the block pavements. Large square concrete slabs, used for sidewalk paving, were thrust one over the other; and in one or two cases a slab entirely covered an adjoining one. These phenomena indicate shortening by compression in the north-south direction. On the other hand, however, a stretching of the surface is shown by fissures in the paving; by places where wedge-like blocks were deprest below the general level; and by the rails of car tracks which were pulled apart in amounts varying from 8 to 12 inches [0.2 to 0.3 m]. Owing to the relatively great and very variable structural strength of paved streets and heavily ballasted car tracks, these phenomena are not developed regularly nor frequently enough to

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				afford a satisfactory test of the hypothesis that they are directly associated with the wave forms into which the surface of this district was thrown. Besides, owing perhaps to the varying rigidity of the materials which make up the surface of the streets and building plots, the wave forms themselves, tho generally prevalent, are not persistent in their extension. The compression and distension effects, however, are believed to be due to the same cause as that which generated the wave forms; for there is no evidence of any true shortening, or lengthening, of the north-south dimension of this district, nor is there any probability of this having occurred. In addition, then to the flow movement and the settling together of the loose materials causing depression, there was some sort of rhythmic movement in this loose earth which produced wave forms in the surface, with places of compression and places of stretching. It is not believed that these surface waves were traveling waves "frozen" as the shock subsided. If they had been of that character, the ground surface should be more broken than it appeared to be; for in relatively rigid materials such waves must develop open fissures along the crests, which would close with crushing in the

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				troughs. It must be noted, without any attempt at explanation, that the destructive effects of great magnitude which have been described above, are practically confined to the "made" land which occupies the old marsh site. Southeast of Brannan Street, where formerly lay Mission Bay, such effects are of less magnitude, in general; are less regular in their occurrence and are, on the whole, less prevalent. The complete devastation caused by the fire in this neighborhood leaves little to indicate the actual damage to the buildings wrought by the earthquake. Certain hotels or apartment houses are known to have collapsed, and many fatalities must have occurred. Probably a few dwellings were thrown down. A fairly large percentage of the buildings, one must believe, were rendered dangerous for occupation, even tho not completely thrown down. ...The space formerly occupied by Mission Bay has been partly filled to provide building sites, and of course the materials used in filling were deposited in water. The district is occupied in part by structures of great strength, such as railway tracks; in part it is devoid of buildings. Thruout the district, evidence was insufficient and inconclusive. Except near the former outlet of Mission Creek, and in the area further north formerly occupied by the tidal marsh, the destruction produced does not denote intensity higher than Grade C. Apparently, therefore, land made by filling up spaces of open water is less dangerous, on the whole, than land made by depositing a thin rigid layer of filling upon a tract of marsh land. This, at least, is the lesson in San Francisco. The reasons for it are not very clear. Space forbids a discussion of theories which can not be adequately tested. It may be noted, however, that much of the material used in filling in areas of water has been broken rock derived from the grading down of neighboring rocky hills."
	3		229	"The blocks between the old tide-marsh area, extending east from near the Post-office, and the former course of Mission Creek, give evidence in the form of cracked foundation walls, broken concrete cellar floors, etc., of intensity values high in Grade C. The fire did much to destroy evidence here, as it was a district of wooden dwellings."
	8b		739	"In the Mission district, and in certain other parts of San Francisco, as, for

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				instance, along the courses of former tidal streams, large areas of filled land exist. In such cases, the effect produced by the earthquake was not generally in the form of waves, as already described, for the eastern portion of the city was basin-like, representing local settlement in the streets and adjacent areas, and in some cases, decided misalignment.... The most important local settlements and transverse movements occurred in the vicinity of...on Mission, at the corner of 7th...."
	13		32	"In the '50s and for some years later the high-water line of Mission bay, a tide-washed mud flat, came to Brannan street, between Fourth and Fifth, swept around to the line of Eighth, then Price, street at Channel and Hooper, then around to the point of high land near the present Central Basin, which point was then known as Point San Quentin. The opposing high-land point on the near side of Mission bay and about at the location of Third and King streets, was known as Steamboat point.
				...Adjacent to this mud-flat bay and the tidal estuary were areas of salt marsh land. Little streams of fresh water came to the heads or upper ends of these salt marsh tracts, forming small areas of fresh marsh,

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				extending back between hills and ridges of the higher lands. A notable area of soft marsh covered most of the space lying west of Brannan and along Fourth and Fifth street to Howard. Another considerable area bordered the bay edge from about the line of Seventh around to Point San Quentin. Another area lay west of the estuary to Folsom street, about on the lines of Fourteenth, Fifteenth and Sixteenth streets, and another area lay around the salt lagoon at the head of the estuary and to the south and west thereof.
				THE MISSION BAY SOFT SPOT It would be tedious in these articles which the writer is endeavoring to keep within readable limits to catalogue too many dry and hard facts made about hard and soft land, so he does not undertake to trace in detail the outline of the Mission bay and estuary, salt marsh and fresh swamp areas, as these have now again been made evident by the street subsidence and other movements caused by our king shake. The region is a large one. To go slowly step by step around it as we did in the case preceding, on foot, as it were, would take too long; so we move rapidly over most of it as in an auto, and commencing on Townsend street, near the Southern Pacific Railroad yards, we notice a disturbance near Cook street, another in

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				Brannan, near Ritch; another near Harrison and Fourth; another near Folsom and Fifth, and another near Howard and Sixth. We find that by these we may outline on the map the old salt marsh limit as far as the greater impress of the earthquake's heel, which is found in the neighborhood of the new Postoffice."
1) Post Office, 7th & Mission Sts.	13		32	MISSION AND SEVENTH STREET DISTURBANCE It looks pretty bad on Mission street at and near Seventh, to see the whole street disturbed for about 700 feet in length, to see that this disturbance extended for about 700 feet [210 m] in length, to see that this disturbance extended far down Seventh street, and that an area of the adjacent land had sunk. As an American one cannot but feel glad that the new Postoffice building escaped, though barely, being in this area of depression. As a San Franciscan who knew this spot fifty years age, who saw it a marsh with a little stream running through it, who saw hunters wearing gum boots tramping about shooting jacksnipe in that very area, who later saw it drained for market gardening, and still later saw it filled to a depth often of fifteen feet [3.0 to
				4.6 m] with sand dumped off a bank from side dump cars, and then saw it occupied by light wooden houses for a score and a half of years, it seems entirely natural that a real earnest earthquake should make it settle and move just as it has settled and moved. It never had an inducement to get down to a good bearing before. Now it has been shaken to where it will probably stay, and San Francisco will be the better for it. In the block south and west of the Postoffice this old John Sullivan marsh formerly headed. Its course was toward the east, joining an area of salt marsh which bordered Mission bay....The facts now are that under the earthquake influence the filling over this marsh area has settled at a number of places and to depths of from a few inches to three or three and one-half feet [0.9 to 1.1 m]. One of the most pronounced settlements is the one referred to on Mission and Seventh where the subsidence has reached a maximum of about three and one-half feet [1.1 m], as judged by the floor of the Postoffice building and the movement toward the bay, as judged by the street railway track and alignment of the trolley line support poles, has extended for 300 to 600 feet [90 to 180 m] in the length of Mission street,

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				reached a maximum of about five feet [1.5 m] at a point 175 feet [53 m] south and west of Seventh. Plotting the limits of this disturbance on the map, they are seen to coincide as near as can be measured with the outline of the old Sullivan marsh as shown by the Coast Survey map of 1857, and as the writer distinctly remembers it to have been."
	1	No. 76	99	Photo: Street slumps to the southeast in distant center of photo. Photo caption: "Mission St. near 7th in front of Post Office. Looking S.W. Street at X badly sunken & twisted (See picture No. 77)."
	1	No. 77	99	Photo: Shows monocline (0.6 to 0.9 m high) in street in front of east corner of Post Office. Car rails arched; pavement broken up. Photo caption: "Mission St. in front of Post Office, near 7th; street sunken, raised & twisted. 12" water pipe ruptured."
	1	No. 77A	99	Photo: Looking east on Mission St. in line with the sidewalk in front of the Post Office. Monocline in street in the distance, roughly 1.5 to 2.0 m from the sidewalk. Sidewalk slabs pulled apart. S corner of Post Office sunken, steps down roughly 0.3 to 0.6 m.

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				Photo caption: "New Post Office. Sidewalk & Street subsided."
	2	Fig. 35A	42	Photo: Front view of S corner of Post Office. Photo Caption: "Damage to San Francisco Post Office, Seventh and Mission Streets, caused by ground failure. A. View of southeast entrance showing differential, vertical movement of lower, nonstructural facing around building."
	2	Fig. 35B	42	Photo: Same as that of ref. 1, No. 77A. Photo caption: "B. View northeastward in front of building showing differential, vertical, and lateral movement of sidewalk."
	2	Fig. 35C	42	Photo: Close up of south corner of Post Office. Shows lateral displacement to the southeast, 0.6 m over 1.5 m. Photo caption: "C. View northeastward in front of Post Office showing settlement around building and lateral displacement of sidewalk to the southeast."
	3	Fig. 94B		Photo: View of the south corner of the building. Shows scarp of roughly 0.2 to 0.3 m across 7th St. in the intersection; subsidence to the southeast.

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				Photo caption: "San Francisco Post-office, Mission and Seventh Streets. Near corner of building is on edge of old marsh. Ground over marsh sank and lurched."
	3		238	"The new United States Post-office building (plate 94B), at the corner of Seventh and Mission Streets, was just on the margin of the district. It is a steel and granite structure, resting upon a foundation of piling driven to a considerable depth, but not as far as some had considered advisable. At its southwest corner, the streets are deformed into great waves, some with an amplitude of at least 3 feet [0.3 m], causing fissures and sharp compressional arches in the pavement and sidewalks. Some of the granite flanking structures, which did not rest upon the pile foundation of the building, shared this undulatory movement. In consequence, the building appears badly damaged to the casual observer. It is quite true that the structure was terribly shaken and greatly damaged -- such injuries as the destruction of mosaics in the arches of the corridor helped to increase the loss -- but the structure was not in peril of collapse, tho one of the low walls had to be supported by timbers. For the most part, the building survived the ordeal, and is in a safe condition for use."

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
	4		97	"To the south and west of Mission street was an elongated, narrow, curved area in which the earthquake damage was very severe. It was commonly reported that this area, which was not far from the south corner of the post-office building, was a stream bed or ravine that had been filled within the recollection of the older inhabitants of San Francisco...J.W. Roberts, the local representative of the Supervision Architect's Office of the Treasury Department... seemed of the opinion that the material under the building was a natural deposit, and not an artificial fill. But toward
			98	the south it was not of a nature to inspire confidence in its carrying power at the depth shown on the foundation plans. He accordingly obtained authority to lower the footings wherever the material at the depth shown on the plans seemed unreliable, so that the footings of the south half of the building were lowered - some of them, as I remember his statements, to a depth of 20 feet [6 m] or more below the basement floor level. At any rate, he carried them to a point where the material, in his judgement, was sufficiently hard and compact. All this underlying material is very sandy; but at considerable depths, I understand, gravel appears, and the combination is almost as hard as hardpan."

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
	4	Pl. XLIIB		Photo: View northward of an unfinished building directly across from the Post Office on the southwest side of 7th. Foundation settled unevenly roughly 0.3 m; lateral spreading as well. Photo caption: "B. EFFECT OF SETTLING OF GROUND SUBJECTED TO EARTHQUAKE VIBRATIONS, STEEL-FRAME BUILDING UNDER CONSTRUCTION. The concrete basement walls were not reenforced. Post-office in the background."
	4		99	"...there was a partially erected steel frame (Pl. XLII, B) on the southwest side of Seventh street, near the post-office. Before the earthquake all the columns were plumb and in true alignment. As a result of the shock there was a lateral shifting of the column bases--the relative movement being almost 2 feet [0.6 m] in some places--at the cellar floor level. The basement walls of the incomplete building were also shifted horizontally; at the east corner, where the walls had met at a right angle, they had been ruptured by a vertical crack and moved laterally in such a way that the angle between them was reduced to about 75 degrees, as nearly as I could estimate it without taking measurements."
	4	Pl. XLIIIB		Photo: Close up of SE corner of building, and monocline in street, 0.9 m high. Pavement badly broken, car rails arched. Photo caption: "B. EFFECT OF SLIP, MISSION STREET, SAN FRANCISCO. Corner of post-office building at the left."
	4		44	"The ground at the corner of Seventh and Mission streets settled about 5 feet [1.5 m] (Pl. XLIII, B). The floor of the building was slightly cracked at that point, and Mr. Roberts stated that there was a settling of about 1 3/4 inches [0.04 m]."
	4		99	"The south corner of the post-office building is shown in Pl. XLIV, A. Mr. Roberts states that accurate measurements show that the building proper settled a little at this point, but not more than one-eighth inch relative to other parts of the structure. The general appearance of the building bears out this statement. The result is remarkably gratifying when the great extent of the nearby surface disturbance on Mission street is considered. The street went down about 4 or 5 feet [1.2 or 1.5 m] at this point as a result of the earthquake (Pl. XLIII, B)."
	4	Pl. XLIVA		Photo: Similar to that of ref. 1, No. 77A. Photo caption: "A. CRACKS IN MASONRY AND SETTLING OF

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				OUTER TERRACE, POST-OFFICE BUILDING, SAN FRANCISCO."
	6	---	20	Photo: Same as that of ref. 1, No. 77. Photo caption: "EARTH-QUAKE EFFECT. Distortion of the surface of Mission Street near Seventh, showing the southeast corner of the United States Post-Office Building at the left hand side."
	6	---	193	Photo: Photo taken from opposite corner across Mission St. from Post Office. Shows extent of depression in front of southern corner of Post Office. Seventh St. affected to the northwest only half way along building. Photo caption: "UNITED STATES POST OFFICE. Northeast Corner of Seventh and Mission Streets. At the curb in front of the building, on the right-hand side, the ground settled 5 feet [1.5 m] and moved to the east away from the building about 6 feet [1.8 m]. At the building line, the ground settled about 2 feet [0.6 m], causing the displacement of the granite coping, steps, etc., at the sidewalk level, as shown. The sidewalk was originally a straight grade on the right-hand side where the sag is now shown."
	6		192	"UNITED STATES POST OFFICE. N. W. Cor. Seventh and Mission Streets.... At the southwest corner, the ground settled about 2 ft [0.6 m] at the building line and about 5 ft. [1.5 m] at the curb, the entire surface from the building line moving out about 5 ft. [1.5 m] to the south. This distorted the sidewalk and steps of the two entrances, there being cracks in the joints of the cement sidewalk slabs 8" [0.2 m] wide. It was necessary to place two temporary wooden steps of about 8" [0.2 m] rise from the sidewalk in its settled position to that portion of the steps which remain approximately at the original height."
	8a		701	"UNITED STATES POST OFFICE -- ...It is especially noteworthy since it is built upon filled land which everywhere in San Francisco was shown to be very unstable and capable of very great distortion and settlement as a result of the action of the oscillations and vibrations due to the earthquake... The effect of the earthquake throughout this vicinity has been most marked. Streets in this neighborhood have settled very considerably and the sidewalk has also seriously sunk. Mission St., at the corner of 7th, has been thrown bodily southward to the extent of at least two feet [0.6 m]."

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
	8b	---	737	Photo: View eastward on Mission St. from corner of 7th. Southwest corner of the building settled at least 0.1 to 0.2 m. Sidewalk and street displaced laterally, pulling apart sidewalk slabs. Photo caption: "Southwest Corner of U.S. Post Office. The first picture was taken at the corner of Mission and Seventh Sts. The ground is a loose fill and settled considerably, while Mission St. has apparently been thrown to the southeast at this point at least 2 ft. [0.6 m]."
	9c	---	580	Photo: Same as that of ref. 8b, p.737. Photo caption: "SOUTHWEST CORNER POST-OFFICE BUILDING, SAN FRANCISCO. PAVEMENT AND CURBING DISPLACED BY EARTHQUAKE."
	14	---	695	Photo: Two views of the southern corner of the Post Office. The first is a view directly facing the southeast entrance, showing the settlement of the steps and sidewalk relative to the building. The second view is south of Post Office from the intersection looking north. Shows just the edge of the scarp across 7th St.
	14		694	"...The soft upper strata of the surrounding street and sidewalk slipped away from the building [U.S. Post Office] at Seventh and Mission sts., from 1 ft. to 3-1/2 ft. [0.3 to 1.1 m], but the foundations are not defective."
	20		588	"In the case of those Class A buildings founded on made soil, such as filled in sand, mud and the like, the walls have been cracked and injured to a considerable extent, requiring in some instances large expense for repairs. A notable example is the U.S. Post Office at the corner of Seventh and Mission Sts. This is a handsome, well-designed and strongly built structure of the new type, three stories in height. Had it been raised upon an excellent foundation it would undoubtedly have passed through the "temblor" unharmed; but unfortunately, the site is a sand lot, across one
			589	corner of which formerly ran a branch of Mission Creek, underlaid by a deposit of mud and silt. The lot had long been filled into grade with sand; and the Government agents, after much adverse criticism of the sub-soil, endeavored to unify and strengthen the "bed of foundation" by the liberal use of steel-concrete piers. The granite walls on every side of the Post Office are badly cracked from the third story under the roof to the foundations; while the walls, the marble veneer, the partitions and the fireproofing on the interior

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				are badly shattered and thrown to the floor in great quantities. The sidewalks around the building are separated from it and each other, and upon Mission St. side have, in places, sunk 3 ft. [0.9 m] below grade, indicating very plainly a defective support."
2) Howard & 6th	8b		738	Photo: View looking northwest on 6th at intersection of Howard. Gradual settlement, shows no scarp; decreases to the north. Photo caption: "Settlement on Sixth Street. The first picture was taken on Sixth st., just south of Howard St. The amount of drop at the lamp post where the men are standing was fully 2 ft [0.6 m]. The sidewalk north of this point and the street at the junction of Howard appears to have settled but very little."
	9b	Fig. 13	551	Photo: Same as that of ref. 8b, p. 738. Photo caption: "STREET SUNKEN; SIDEWALK IN PLACE BECAUSE ON FOUNDATIONS."
	9b		553	"Fig. 13 shows a drop in Howard St. Where the men are standing a sidewalk on foundations has remained in place."
	3	Pl. 89C		Photo: Shows general settlement of street.

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				Photo caption: "C. Sixth Street, near Howard. Once occupied by marsh. Street dropt nearly 3 feet [0.9 m]. Sidewalk held up by piling foundation of a building."
3) Mission & 5th	4	Pl. XXXVIIIA		Photo: View looking north. Shows scarp across Mission street of roughly 0.3 m. Photo caption: "GOOD EARTHQUAKE ENDURANCE OF A BUILDING OF THE MONUMENTAL TYPE: UNITED STATES MIN, SAN FRANCISCO."
4) Near Folsom between 6th and 7th	3		237	"Foundation walls and sidewalk pavements were broken and flexed; sharp little anticlines were produced in the street by the arching of block paving, as on Russ Street between Folsom and Howard Streets (pl. 88C); granite curbing was broken and thrust up in to an inverted V, as on Moss Street, between Folsom and Howard Streets (pl. 88D): there were fissuring and slumping in the block pavement, as along Columbia Street between Folsom and Harrison Streets (pl. 89A), and sharp flexures of the paved streets and car tracks, as on Sixth Street just south of Howard Street. These effects point simply and clearly to the great magnitude of the intensity thruout the greater part of this old swampy district."

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
	3	Pl. 88C		Photo: Compression ridges trending north-south along Russ St. Roughly 0.3 m compression. Photo caption: "C. Russ Street, between Folsom and Howard Streets, San Francisco. Paving blocks forced up into sharp arches and dislodged by compression."
	3	Pl. 88D		Photo: View facing north-east. Compression ridges trending north south (roughly 0.3 m compression in east-west direction). On west side of street is a curb block arched sharply into an inverted V indicating compression in the north-south direction. Photo caption: "D. Moss Street, between Folsom and Howard Streets, San Francisco. Paving blocks archt by compression along sinuous crest, curbing thrown into an inverted V."
	3	Pl. 89A		Photo: View south down Columbia Street. Shows lateral spread; scarp irregular; slumping 0.6 m roughly to east. Photo caption: "Columbia Street, just south of Folsom Street, San Francisco. Slumping, depression, and furrowing of block pavement."
	2	Fig. 33	40	Photo: Same as ref. 3, Pl. 89A.
				Photo caption: "Disruption of block pavement and sidewalk on Columbia Street just south of Folsom Street."
5) Fifth near Harrison	2	Fig. 34	41	Photo: Shows extension of car rails of roughly 9 cm. Pavement blocks pulled apart at this location. Photo caption: "Rails on Fifth Street near Harrison Street, San Francisco, pulled apart by extensional movements associated with lateral spreading of underlying sediments."
	8b		740	"On Fifth St., between Folsom and Harrison Sts., the brick sewer settled with the adjacent land, its crown was destroyed and the channel had become filled with sand which blocked up the sewage to a dangerous extent.
6) (4th, near Bryant)	10	---	554	Photo: Shows compression in car tracks, both by arching and by bending. Photo caption: "STREET CAR TRACK ON 4TH ST., NEAR BRYANT, SAN FRANCISCO, DISTORTED BY EARTHQUAKE."
	10		554	"The view was taken looking north on Fourth street, near Bryant street, shortly after the fire had burned itself out in the vicinity. As this piece of track was observed hours before the fire

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				devastated both sides of the street of the buildings, it is impossible to attribute the distortion to heat...The common type of track distortion caused by the earthquake waves is also shown in the view on the left hand track. Here both rails are bent upward about 5 ins. in 3 ft [0.13 m in 0.9 m]. The rails bend up in cases of this kind, because of the resistance offered in every other direction by the ties, pavement and track itself."
	3	Pl. 89B		Photo: View looking northeast up Bryant from 4th? Compression of car rails by roughly 0.06 m. Photo caption: "B. Bryant Street, near Fourth Street, San Francisco. Flexure of heavily ballasted car tracks in block pavement; an effect of sharp compression."
	5		265	"...the writer twice passed by a number of brick warehouses on the north side of the Southern Pacific Railroad, between Fourth and Sixth Streets, and took a little time to examine them...They also had the disadvantage of being built on extremely poor foundations, their settlement during the earthquake, with respect to the railroad right of way, just south of them, being about 2 ft [0.6 m]."

FOOT OF MARKET ZONE:

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
General:	3		233	"About the Ferry Building, at the foot of Market Street, is a district of "made" land, shown on map 17, in which high intensity was manifested....In spots the streets sank bodily, certainly as much as 2 feet [0.6 m], probably more. Accompanying this depression, concrete basement floors were broken and arched, as if to compensate for it. The surface of the ground was deformed into waves and small open fissures were formed, especially close to the wharves. Buildings on the water side, along East Street, generally slumped seaward, in some cases as much as 2 feet [0.6 m]. The damage was greatest close to the water's edge, growing less as the solid land was approached, gradually at first, then more rapidly. These phenomena seem to suggest that the materials used in filling were shaken together so as to occupy less space with the accompanying development of waves, fissures, and structural damage. The more recent the filling, the more it would be compacted; hence the greater prevalence and magnitude of destructive effects near the water's edge. As well as could be made out from the inadequate evidence left by the fire, the district which suffered intensity of Grade B

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				is limited on the landward side by a line drawn from Filbert Street to Market Street, between Battery and Front Streets; thence between First and Fremont Streets to a little south of Folsom Street, where the line turns and runs eastward to the wharves. Flanking this district on the landward side is a narrow, sinuous area limited by a line drawn from Filbert Street to Green Street, just east of Sansome Street; thence between Sansome and Montgomery Streets to Market Street; thence to the corner of Mission and First Streets; thence between First and Fremont Streets to a point south of Folsom Street; thence easterly nearly to the wharves. Between Washington and Sacramento Streets, this boundary is barely east of Montgomery Street. Immediately west of these districts, low intensity prevailed."
	8b		739	"THE STREET PROBLEM.--The effect of earthquake was very marked in San Francisco wherever filled ground existed. The original shore line was far different from that of today and passed around the easterly base of Telegraph Hill to the southerly end of Montgomery Avenue, thence southeasterly to Market Street at Battery Street, thence to Russian Point, thence around the old Mission Bay. All of the lane now lying east of

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				this broken line has been recovered from the sea by filling on the marshes and tidal lands. EARTHQUAKE EFFECTS.--Within this filled district, the vibrations of the earthquake caused a general but irregular settlement. The streets naturally followed the changes in elevation and a wave-like effect was produced. Observations on Market, Mission, East and other streets frequently indicate an amplitude of wave height of two feet [0.6 m], while occasional places are found with greater differences in elevation. ...The most important wave-like distortions were observed on Lower Market and Mission Sts., and on East St. along the present water front."
	9a		503	"The earthquake destruction was most marked on soft and sandy soil and upon made ground. The Ferry building, at the foot of Market St. is decidedly damaged...This building rests upon excellent foundations, but it is supported upon material which seems to have acted like a viscous fluid...All of the made ground between the Market St. water front and the region of Montgomery St. has been decidedly moved and deformed. Wave-like effects are common along

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				lower Market St. and the water front. Wave-like depressions and crests amounting to four and five feet [1.2 and 1.5 m] are found throughout this region. The same observations can be made in many other localities of the city, where soft ground is met."
	13		32	"THE PRINCIPAL SOFT SPOT. Here are, in general terms, the leading facts of earthquake effect within the area which we may designate as the city's principal soft spot, even though it is not the largest area of marsh and mud flat which has been filled over and built upon. Beginning on the east and west streets at the north limit of the area of disturbance, we find the uppermost evidence on Pacific street at about the corner of Front; then on Jackson street at about the intersection of Battery; next on Washington street at the intersection of Battery; next on the Clay near the intersection Sansome; then on Sacramento, also at the intersection of Sansome; on California it is doubtful whether the disturbance at Sansome is due to general subsidence or only to subsidence toward foundation excavation; on Pine street the upper limit of pronounced street disturbance is about 100 feet [30 m] above Battery; on Market the line is very distinct at about the foot of Bush street; on Mission a sharp line of subsidence occurs across the street about 100 feet [30 m] below First; on Howard and in the line of Fremont to the east thereof the line is again plainly marked by a crack and subsidence below it; and finally, on the line of Folsom, about seventy feet [21 m] below Beale, a line of subsidence is very distinctly marked diagonally across the street exactly at the location and in the direction of the foot of the old hard ground and edge of the former mud flat. Platting these points on the map it is found that they either lie within or outline very nearly the limit of the former soft spot in the lower portion of the principal business part of the city. Below this bounding line the streets are nearly all waved, there being depressions of from six inches [0.15 m] to as much as four feet [1.2 m] in one or two places and two or three feet [0.6 or 0.9 m] at quite a number of points. While it cannot be said that the whole street area within this zone has sunk, a considerable portion of it has, and near the water front most of it has sunk from six to twelve inches [0.15 to 0.3 m], with several areas of greater depression.

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				Davis street, from Vallejo to California street, presents perhaps the extreme case, there being distinct depressions of from one to three feet [0.3 to 0.9 m] at every street crossing or within every block; but it is apparent that some of this is due to slip of the street filling into building foundation areas, consequent upon failure of retaining walls and poor foundation os the buildings themselves. The area of about eighty feet [24 m] frontage which has sunk to a maximum of about eighteen inches [0.46 m] in front of the Market-street Ferry building, and the depression of somewhat greater area and to a maximum depth of about three feet [0.9 m], at the northwest corner of the same building, are the extreme cases of subsidence along the main waterfront. Contrasted with the very heavy masonry Ferry building, founded on piles and concrete, which has not sunk at all, these subsidences will illustrate the point that it is only the soft mud and loose filling thereon which has been disturbed by the earthquake. There are places on the north and south streets where the whole street appears to have been thrown a few inches toward the bay and at East street, which is the waterfront street, there is much evidence of similar movement to about six to eighteen inches

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				[0.15 to 0.46 m] where the pavement has been shoved against wharves, piers and other water-front structures and caused to buckle up. Street and water-front railway rails are in a number of places buckled up six inches to two feet [0.15 to 0.6 m] or are thrown as much as six inches [0.15 m] out of line. Throughout the filled area above street-corner silt basins have been tripped out of plumb and bulged into sidewalk area, and sewer manholes in street intersections are in several places canted up, showing sewer disturbance beneath; while granite curbs for 100 feet [30 m] or more in length were tripped up by unequal movement of street pavements and the underlying ground and thrown out bodily on their sides upon the pavement or sidewalk. The Market-street Railway track, carried on a prism of concrete founded on piles for its length within this area, did not sink with the street on each side of it and is yet nearly on its original grade except at one point, where it has sunk apparently about four inches [0.1 m] for several hundred feet. It is noticeable that streets have sunk least or not at all in front of the newer deep-piled foundations for adjacent build

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				ings -- Market street in front of the Hotel Terminus and in front of the Buckley building, for instance -- and this indicates that a part of the street movements is due to settlements into cellar and foundation excavations on failure of their retaining walls. A curious revelation is noticed on the west side of Davis street, between the Broadway and Pacific street. Here in 1857 was the water-front wharf. When the street was filled in it is evident that all the piles were not removed. The street pavement, which is basalt blocks, has sunk six inches (0.15 m) or more for the full length of the frontage, and the position of the pile heads for about half the length is marked by their punching the pavement up in little pyramids, and for the other half length the position of the pile bents with caps on is shown by the pavement sinking on either side of the caps, leaving ridges of paving blocks over them. Evidences of old structures beneath the surface and filling are brought out in a similar way at a number of points."
	20		589	"The water front and sea wall of San Francisco lie far outside the old shore line of the harbor; and the intervening mud-flats and silt bottom between

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				were at an early day in the history of the city filled in with rubbish and sand. Upon this material, sometimes using long piling, and at other times platforms and grillages of timber, a large number of the commercial and wholesale houses were erected. This "made ground" was greatly disturbed and rolled into waves by the earthquake; but wherever the piling under the brick or stone walls had been heavy, deep and well done there resulted little injury to the masonry."
1) East St.	8b	---	739	Photo: View on East St. looking west up Pacific St. Scarp running NW-SE across East St., slight subsidence to north. Car rails bent into arch-compression of roughly 0.1 m. Photo caption: "Tracks at East and Pacific Streets. The first picture shows the distortion of car tracks and the line of the local fault in East St. at its junction with Pacific St., along the water front. This territory is within the area of made land on the marshes of the old harbor front."
	9b	Fig. 14	552	Photo: Same as that of ref. 8b, p. 739. Photo caption: "DEFORMATION STREET RAILWAY TRACKS, EAST AND PACIFIC

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				STS., NEAR FERRY BUILDING."
			553	"Fig. 14 shows the destruction of car rails and street surface, corner East and Pacific Sts., near the Ferry Building."
	11	Fig. 12	125	Photo: Same as that of ref. 8b, p. 739. Photo caption: "Rupture of Car Tracks and Pavement on East Street, Corner of Pacific Street."
	9b	Fig. 15	552	Photo: View looking west on Market. Shows roughly 0.1 m scarp and crack running across width of East St. Crack crosses tracks which are not disturbed. Street slumps to the south. Photo caption: "CRACK IN PAVEMENT IN FRONT OF FERRY BUILDING."
			553	"Fig. 15 is a similar view in front of that building [Ferry Building]."
	11	Fig. 11	123	Photo: Similar to that of ref. 9b, Fig. 15. Photo caption: "Street Surface in Front of the Ferry Tower, Showing Undulations and Cracks in the Asphalt Pavement."
	1	No. 71	97	Photo: View northward [?] on East St. near Ferry building. No exact location given. Shows cracks in pavement resembling a graben formed by lateral spread eastward as much as 1.0 m.

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				Photo caption: "Street on Waterfront badly broken up."
	2	Fig. 31	37	Photo: Similar to that of ref. 1, No. 71. Photo caption: "Cracks and separations in roadway pavement near the San Francisco waterfront caused by lateral spreading in the foot of Market zone."
	6	---	19	Photo: Similar to that of ref. 1, No. 71. Photo caption: "EARTHQUAKE EFFECT. A fissure on East Street near the water front. Note the settlement of the street surface as shown by the exposed curb at the right had side. In this locality the ground was "made," or artificially filled in."
2) Lower Market St.	3		236	"..The cable-car system on lower Market Street...were constructed upon piling to secure permanence of grade. On both sides of them the street sank in places as much as 2 feet [0.6 m], and the pavement was broken, fissured, and thrown into waves."
	11		98	"Within the confines of the city of San Francisco one finds evidence of great variation in shock closely related to and to be explained by the nature of the surface topography. It is a general observa-

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				tion that the earthquake waves transmitted by the softer and less coherent materials and formations appeared to be much more destructive than waves which traversed the hard and more elastic rocks and other sound deposits. The billow-like effects that appeared in the streets of San Francisco near the Ferry house are most excellent examples of deformations in soft, incoherent materials. The sliding and rolling effects observed on some of the sand dunes and especially along the hillside at the northern end of Van Ness Avenue may be cited as allied phenomena."
	1	No. 63	95	Photo: View eastward toward Ferry building from foot of Market. Shows pavement broken up; street thrown into waves. Photo caption: "Market St. near Ferry. Street subsided. 16" water pipe ruptured."
	4	Pl XLVIA		Photo: View of Ferry building. Shows undulations in street just in front, the crest trending N-S. Shows subsidence along NW corner of building. Photo caption: "A. BRICKWORK THROWN DOWN BY EARTHQUAKE VIBRATION, TOWER OF UNION FERRY BUILDING, SAN FRANCISCO."

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
	4		135	"The Union Ferry Building (Pl. XLVI,A), with the exception of its high tower, was little injured, and the level of its floors was no perceptibly changed. At the same time, the streets at its front, which rested simply on the made soil, were rolled into waves 3 or 4 feet [0.9 to 1.2 m] in height."
	5		319	"On Market Street, near the Ferry House, the cable tracks resting upon piles moved very little in comparison with the collapse of the street surface on both sides of the car tracks."
	5	Pl. XXXV Fig.2		Photo: View east on Market St. towards Ferry building. Shows settlement of street relative to sidewalk roughly 0.9 m. Photo caption: "Destruction of Buildings of Type 2 Construction at Foot of Market Street, San Francisco."
	6		226	"UNION FERRY BUILDING.... The street surface at the N.W. corner settled 2' [0.6 m], cracking the asphalt pavement and carrying down a large section of the sidewalk."
	1	No. 60	95	Photo: View looking west up Market St. Aetna building on left in distance. Shows settlement around car rails and waves.

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				Photo caption: "Market St. west of Steward sunken & 16" pipe broken."
3) Spear St.	5		288	"The Foljer Building, on Howard Street...but otherwise the building was not damaged; and yet, at this very site, the street in front of the building had settled about 2 ft [0.6 m]."
	6		28	"FOLGER BUILDING. S.W. Cor. Howard and Spear Streets....The street level settled about 2 ft. [0.6 m] at the northeast corner of the building, but there are no earthquake cracks visible in the walls at this point.
	20		590	"...Folger Building [SW corner of Howard and Spear Sts.] ... it rested on pile foundation and the streets on both sides of it sunk 5 ft. [1.5 m] bodily."
	1	No. 78	99	Photo: View southwest up Market St. from Spear St. Aetna (or Young) building in left foreground of photo. Shows subsidence of roughly 0.6 m of street relative to sidewalk.
				Photo caption: " Market St. Cor. of Spear St. Street sunken several feet. 16" water pipe ruptured."

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
	4	Pl. XXVA		Photo: View looking SE down Spear St. from Market St. Shows subsidence of street of roughly 0.6 m relative to sidewalk.
				Photo caption: "A. SUBSIDENCE OF STREET IN FRONT OF AETNA BUILDING, SAN FRANCISCO."
	4		76	"The steel-frame structure at the corner of Spear and Market streets...Pl. XXV, A, shows the corner of the building and the subsidence of the street at this point. The inlet at the corner indicates the original level of the street. There was a vault under the Market street sidewalk, immediately behind the wall at the curb line. The basement floor in this vault was of concrete and had a total thickness of 7 or 8 inches [0.2 m]. The earthquake caused the earth to bulge up in the portion of the basement under the sidewalk, rupturing the concrete floor and turning it up on its edge, so that where there had previously been a clear headroom of 7-1/2 feet [2.3 m] the highest point of the bulge was within 3-1/2 feet [1.1 m] of the beams carrying the sidewalk."
			32	"The basement floor, which was of concrete 7 or 8 inches [0.2 m] thick, was pushed up under the sidewalk, reducing the headroom at this point from 8 feet to 3-1/2 feet [2.4 to 1.1 m], approximately. This bulging was probably

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				caused by settling (Pl. XXV, A), as the foundation piling did not extend under the sidewalk."
	6		80	Photo: Shows subsidence of 0.6 m of street relative to sidewalk. Photo caption: "YOUNG (OR SELLER) BUILDING...The foundations on the east side has settled, and the east and west walls are out of plumb about 5"..."
	6	---	81 84	"YOUNG OR SELLER BUILDING. S.W. Cor. Spear and Market Streets...Levels on the water table show that the N.E. and S.W. corners are 3" and 6" lower respectively than the N.W. corner. These facts would indicate that the foundations had moved sufficiently to tilt the entire building to the east. From marks on the curb of the sidewalk, it is also apparent that the surface of the ground settled considerably around the N.E. corner."
	7	---	643	Photo: Young Building and sidewalk on good foundations; street in front subsided 0.6 to 0.9 m. View shows corner of Spear and Market Sts., looking southeast. Settlement less up Spear St. to the southeast. Note other references place this building at Spear not Stewart. Photo caption: "The Young Building."
	7		643	"The Young Building is at the corner of Stewart and Market Sts. This building was within one block of the water-front...The building is on pile foundation, as it is situated in the portion of the city that is constructed on filled ground. The portion of the street on the corner, it will be seen, has settled on this account about 2 ft [0.6 m]."

OTHER LOCALITIES:

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
1) Duboce Park	3	Pl. 90D		Photo caption: "Southwest corner Portola and Waller Streets. Buildings have shifted down hill slightly, upper parts more than lower."
				Photo: Difficult to ascertain ground movements.
	3		240	"Near the corner of Waller and Portola Streets, not far north of the head of Market Street, is a locality, less than a block in extent, where houses were shifted slightly on their foundations; their upper stories were moved farther eastward (downhill) than the foundations, as a result of shearing in the framework of the basement or of the first story of the buildings. (Plate 90D.) There also occurred minor bucklings and breaking of the thin asphalt pavement. The intensity, which belongs low in the range of Grade B, diminishes rapidly in all directions, and the district is surrounded by a band where the intensity is Grade C. Here a thin layer of sand reposes upon the slopes of a little upland valley between the low serpentine hills to the east and the high chert hills to the west. The effects are such as would be produced by a shaking downhill of this thin sand layer, with the structures which rest upon it."
2) Steiner and Sutter Sts.	3	Pl. 87B		Photo caption: "Geary and Street, between Filmore and Steiner Streets, San Francisco. Buildings of mediocre construction on sand and alluvium of no great depth."
				Photo: Ground movements difficult to ascertain.
	3	Pl. 92A		Photo caption: "St. Dominic's Church, Bush and Steiner Streets. Brick and masonry structure upon sand and alluvium of no great depth."
				Photo: Ground movements difficult to ascertain.
	3		231	"In the neighborhood of the crossing of Steiner and Sutter Streets, there is an irregularly bounded district a little larger than a city block in which several buildings not conspicuously weak were totally destroyed. St. Dominic's Church, at the corner of Steiner and Bush streets, was a complete ruin, as the illustration (plate 92A) shows. Its steeple towers were ruined, its roof fell in, and all its walls were so badly cracked that it became a menace to the neighborhood....Near by small frame dwellings were pitched from their underpinning.
				On Geary Street, just above Fillmore Street, two wooden-framed brick buildings standing side by side -- the Albert Pike Memorial Temple (Masonic)

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				and a Jewish Synagogue--were utterly wrecked, as the illustration shows. (Plate 87B.) The Girls' High School near by on O'Farrell Street, at Scott Street, poorly and flimsily built, was badly damaged. Its walls were much cracked and portions of the gable walls were thrown down. This district of Grade B intensity is on the floor of Upper Hayes Valley and is surrounded by a relatively broad area in which Grade C effects prevail. It lies near the base of the hills which hem in the valley on the east. The surface strata are sand and alluvium extending to no great depth, unless the slopes of the bedrock hills change suddenly where they pass under the mantle of loose materials. No explanation can be offered for the occurrence of this limited area of high intensity (Grade B) unless it be that the district has been converted into "made" ground by extensive grading in the preparation of the surface for building sites and streets."
3) Vallejo St. and Van Ness Ave.	1	No. 73	99	Photo caption: "Van Ness Avenue between Vallejo St. & Broadway." Photo: Shows abrupt subsidence of street, with 0.1 to 0.2 m scarp across street.
	1	No. 74	99	Photo caption: "van Ness Ave. near Vallejo. 8" [0.2 m] water pipe ruptured." Photo: Shows 0.1 to 0.2 m scarp across street, lateral spreading.
	1	No. 75	99	Photo caption: "vaNess Ave. between Vallejo & Broadway." Photo: Shows subsidence of street of 0.1 to 0.3 m, and lateral spreading of roughly 0.3 m.
	1		43	"The vicinity of Van Ness avenue and Vallejo street is one of the prominent points of interest. It was found that Van Ness avenue had been more or less affected from a point 150 feet [46 m] south of Vallejo street, the greatest subsidence being two feet [0.6 m] at the crossing of Vallejo street. There was also subsidence of Vallejo street from 150 feet [46 m] on each side of Van Ness avenue. There was a lateral movement to the north on Van Ness avenue of about three feet [0.9 m] on Vallejo street, decreasing to about one foot [0.3 m] on Green street, the ground and buildings upon it having been moved bodily so that now the buildings encroach upon the neighboring lots or upon the street. As a result of the subsidence and lateral movement the sewers extending east, south and west of the crossing of Van Ness avenue and Vallejo street

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				were broken for about 150 feet [46 m]. The scene of the disturbance was an old fill of about forty feet [12 m] which had been made years ago in the ravine leading to the northwest to the lagoon formerly called Washerwoman's bay."
	2	Fig. 52	58	Photo caption: "Disruption of Van Ness Avenue over a filled in ravine. Lateral movements as great as 3 ft (0.9 m) and vertical movements as great as 2 ft (0.6 m) occurred at this location." Photo: Shows jagged scarp across street.
	3		231	"At the corner of Vallejo Street and Van Ness Avenue, fissures were formed in the asphalt paving, sidewalk pavements were thrust over the curbing, and water-mains and sewers were broken. Buildings were thrown out of the vertical, and foundations and lower story walls were shifted and crusht. The walls about the foundation of one brick building were actually deformed into undulations with much consequent cracking. This building was so badly damaged that it had to be taken down. Surrounding this corner is a small ovoid district, about 2 blocks in extent, in which the intensity was clearly of Grade B. This was once a sharp ravine and had

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				been filled to a depth of 40 feet [12 m] in order to provide a suitable grade for streets and buildings. The filling was shaken together and moved slightly downhill."
	4		118	"At one point on Van Ness avenue (see B, Pl. LVI) [shows location on map], where I happened to see the mains uncovered, a heavy water pipe, apparently about 20 inches [0.51 m] in diameter, had been broken into pieces not more than 2 feet [0.6 m] long. The total length of the break, however, was not more than 40 or 50 feet [12 to 15 m], so far as I could judge from what I saw uncovered."
	20	---	592	Photo caption: "Houses on Van Ness Ave., San Francisco, Where the Street Sank Five Feet [1.5 m]." Photo: Shows depression of sidewalk in front of houses. Difficult to say from photo that the street sank 1.5 m at this point.
	20		590	"The view of the houses on Van Ness Ave. is interesting, as showing the effect of the earthquake at a point where the street surface dropped 5 ft [1.5 m]."
4) Lombard and Octavia Sts.	3		232	"On Lombard, between Gough and Octavia Streets, is a little area, less than a

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				block in extent, in which the destructive effects were of Grade B. No particularly notable effects were produced. It is a district of made land, formerly the site of a little lagoon in the sands, known as Washerwoman's Lagoon."
5) Union and Steiner Sts.	3	Pl. 88B		Photo caption: "Slip of a fill on Union Street, just west of Steiner Street, San Francisco." Photo: Shows lateral movement and subsidence of street and sidewalk. Resembles lateral spreading.
	3		232	"A portion of Union Street, between Pierce and Steiner Streets, not more than a quarter of a block in length, where a filling had been made to equalize the street grade, was shaken down into the adjacent building lot on the north. The north sidewalk was shifted about 10 feet [0.3 m] to the north, and depress about 10 feet [0.3 m] below its original level. The south sidewalk was deprest a few inches and shifted to the north from 2 to 3 feet [0.6 to 0.9 m]. The paving and the cable conduit suffered more severe damage than at any other point in the city. The photograph (plate 88B) conveys a graphic conception of the very great violence which occurred here. The phenomena have no general significance, however, despite their striking character, being merely a sliding of unconsolidated material not supported on the sides. But that such places are dangerous building sites, especially in regions subject to seismic disturbances, is unequivocally demonstrated."
	2	Fig. 53	58	Photo caption: "Slump in Union Street between Pierce and Steiner Streets." Photo: Same as that of ref. 3, Pl. 88B.
	1	No. 79	99	Photo caption: "Union St. between Steiner & Pierce St. Street, cable road, sidewalks, water & other pipes & sewer moved laterally several feet [0.3 to 0.6 m]. Note: A-A_1-B shows former position of center slot of North Track. (looking west.)" Photo: Shows lateral movement and subsidence to the north.
	1	No. 80	99	Photo caption: "Same as No. 79, only taken from different point. Note destruction of formerly straight sidewalk." Photo: View similar to that of ref. 3, Pl. 88B. View west.
	1	No. 81	99	Photo caption: "Union St. with Cable Road. Same as No. 79, only taken in opposite direction, looking from west to east."

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
6) Marina District	3		232	"Along the north shore water-front, between Fillmore and Steiner Streets, from bay Street to the water's edge, was a plot of made ground occupied by a gas-producing plant. Here brick walls were cracked and partly thrown down; part of the wooden framework was wrenched out of position, and the chimney stack was broken. One of the large gas-containers was badly wrecked, but whether its destruction was caused directly or in some secondary way, as by rapid leakage, is not known. The intensity was clearly Grade B."
	4		27	"The group of building comprising the plant of San Francisco Gas and Electric Light Company, built on the soft ground along San Francisco Bay just west of Fort Mason, was badly shaken, and none of the buildings escaped damage...The ground settled very considerably under the vibrations of the earthquake, and further destruction was caused by the unequal settling of the building ...The end wall of the retort house was pushed out 1 foot [0.3 m] at the center, but was saved from collapse by the tie-rods which held it to the roof truss."
	3		232	"Along Lyon, Baker, and Broderick Streets, north of North Point Street, is a small locality 2 blocks wide and 4 blocks long, where the Baker Street sewer was broken and frail frame buildings were thrown out of the vertical. This district was partly made land, but the greater part was on the point of a sand-pit. Unquestionably extensive grading had been done to prepare the ground for building."

TABLE A.2 DOCUMENTATION OF PIPELINE BREAKS RESULTING FROM GROUND MOVEMENTS IN SAN FRANCISCO DURING 1906 EARTHQUAKE.

PIPELINES BREAKS:

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
	1		43	"APPENDIX D. The following are extracts from the report of City Engineer Woodward on the breaks in the San Francisco sewer system caused by the earthquake as published in the "San Francisco Chronical" of June 17th, 1906: ...He further summarizes the following breaks in the sewers: 'On Fourteenth street, between Valencia and Harrison streets; on Harrison street, between Twelfth and Thirteenth streets; on Eleventh street, between Harrison and Bryant streets; on Ninth street, between Bryant and Brannan streets; on Dore street, between Bryant and Brannan streets; on Laguna street, between Greenwich and Lombard streets; on Seventeenth street, between Folsom and Harrison streets; on Howard street, between Seventeenth and Eighteenth streets."
	4		19	"The failure to control the fire by reason of the crippling of the water supply was not due to the failure of the system outside of the city, but to the breaks in the distributing mains within the city, which rendered unavailable about 80,000,000 gallons of water store within the city limits. The breaks occurred (see the map, PL. LVI) wherever the pipes passed through soft or made ground. No breaks occurred where the cast-iron pipe was laid in solid ground or rock."
	4		26	"The settling of the ground in the mud flats along San Francisco Bay and of the filled ground in old water courses was accompanied with great destruction. It was in such ground that the greatest number of breaks occurred in the cast-iron gas and water mains and the sewers. The breaks in the sewers were not so evident as those in the gas and water mains, for the reason that the latter were under pressure and breaks in them resulted in breaks in the streets themselves. The most noticeable destruction resulting from the settling of soft or filled ground occurred in Howard and Shotwell streets between Seventeenth and Eighteenth streets, Bryant street between Ninth and Tenth streets, Dore street between Bryant and brannan streets (Pl. VI, A), and at the corner of Seventh and Mission streets."
	5		253	"In San Francisco, all serious fractures of water mains, as a result of the earthquake, were due to lateral displacements, or subsidences of filled or soft ground across which, unfortunately, the main

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				supply pipes from the lower and middle service distributing reservoirs passed. The map, Plate XLVIII, shows these areas bordered with a heavy line where clearly defined and with a broken line where not so distinctly and connectedly traceable. The displacement laterally amounted in places to as much as 6 or 7 ft. [1.8 or 2.1 m]; vertically, it amounted to several feet."
	8b		740	"Attention might well be called to the destruction of gas mains during the fire, owing to the fact that these were badly shattered by the earthquake in certain places, probably for the most part where they passed from soft to firm ground and vice versa. It is possible to believe that some explosions of gas occurred due to the heat of the fire itself. Many of the explosions produced serious destruction of the streets in their vicinity, and probably wrecked large water pipes, sewers, etc., which would otherwise have been uninjured."
	8b		740	"THE SEWAGE PROBLEM.--The construction and maintenance of sewers in certain parts of San Francisco where filled land exists has always presented a difficult problem. As an example, it may be said that in 1903, when the Sixth St. sewer, between Howard and Folsom Sts. was reconstructed, it was found that the former sewer had settled from 6 to 9 ft. [1.8 to 2.7 m] during the twenty years of its existence. The distortion was ultimately such as to cause failure of the crown in certain places. As the result of the earthquake, however, many of the effects which had been produced very slowly before, were caused at once with very serious consequences. Brick sewers, the usual type for the larger conduits, were in many places distorted, their crowns were broken and their earth covering was carried into them, causing almost complete stoppage in some cases.
MISSION CREEK ZONE:				
1) Valencia St. between 17th and 18th	1	No. 57	93	Photo: Shows destruction of sewer; does not show water mains. Photo caption: "Valencia Str. between 18th & 19th subsided about five feet [1.5 m]; destroying sewer, and besides gas & electric pipes & conduits, tore of one 16" [0.41 m] and one 22" [0.56 m] water main, which at this point had about 85 lbs pressure."
	1	No. 59	93	Photo: Another view of photo, ref 1, No. 58. Does not show sewer, but does show street subsidence and lateral spread.

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				Photo caption: "Valencia Str. Sunken portion of Str. showing two above emergency pipes on top of pavement; also 5' [1.5 m] sink with car tracks & broken sewer at A."
	1	No. 64	95	Photo: Shows street subsidence; does not show pipe. Photo caption: "Other view of same [ref. 1, No. 63 showing Valencia Hotel], showing large hole in sunken street; also water pipe at x."
	3		239	"Sewers and water-mains were broken. At Eighteenth and Valencia Streets there was a serious break in the water-pipe. Here, on both sides of the street, the ground sank about 6 feet [0.2 m], causing the roadway to arch in a very noticeable way. (Plate 93B.)"
	8b	---	738	Photo: On Valencia. Shows general destruction by fire, but ground displacements are unclear. Shows explosion crater in middle of street. Photo caption: "Disruption of Valencia Street... The disruption of the street in the second picture was caused by the explosion of a large gas main."
	8b	---	739	Photo caption: "Temporary Main on Valencia Street... The second picture shows a 24-in. [0.6 m] cast-iron pipe line hastily placed in Valencia St. to provide water for the higher districts, whose supply had been cut off by the destruction of the mains in this street between Eighteenth and Nineteenth Sts. A 16-in. [0.4 m] pipe to supply the lower downtown districts was also placed here in an incredibly short time."
	8b		739	"The earthquake caused a settlement of from 6 to 8 ft. [2 to 2.4 m] for a distance of from 150 to 200 ft. [46 to 61 m] along this street, and at the same time shifted the entire street, with adjacent lands, eastward through a maximum distance of 9 to 10 ft. [2.7 to 3.0 m]. This change in alignment and grade could, of course, meant nothing less than the entire destruction of all water and gas mains, electric lighting and telephone conduits, sewers, cable conduits, railroad tracks, etc. The breakage of these important lines, especially to the water mains, was of greatest significance. In this case the destruction of the water pipes, of which a 28-in. and a 16-in. [0.71 and 0.41 m] were found in this street at this point, meant the cutting off of a large part of the water supply of the portion of the city which was soon to be in flames."

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
	8c	---	765	Photo caption: "Destruction of Sewer by Settlement of Street....In the second picture, taken on Valencia St., between Eighteenth and Nineteenth Sts., the broken sewer is in a deep fill along the former course of Mission Creek."
	8c		767	"PIPES DESTROYED BY UNEQUAL SETTLEMENT.--Wherever filled ground existed, settlement in greater or less degree took place as the result of the tremblor. Pipes passing from comparatively firm and incompressible earth into such softer materials or vice versa, suffered considerably. This effect was particularly noticeable in San Francisco on account of the large number of places where such conditions were to be found. Probably at no point were more serious results produced than on Valencia St., between 18th and 19th Sts., already described in connection with street and sewer problems. Two very important distribution mains were located at this point and were, or course, ruptured. One of these pipes, 22 in. [0.56 m] in diameter, supplied the higher districts of the city; the other, a 16-in. [0.41 m] pipe, was an important artery of the system furnishing water to the business section."
	9b	Fig. 10	550	Photo: Similar to that of ref. 8b, p. 738, but closer view. Photo caption: "Exploded Gas Main, on Valencia St., near Market."
	9c	---	581	Photo: Shows street subsidence and lateral spread; does not show broken pipes. Photo caption: "STREET SUBSIDENCE IN SAN FRANCISCO. VIEW ON VALENCIA ST., NEAR 18TH ST., OPPOSITE SITE OF VALENCIA HOTEL. (It was at this place that street water mains were broken. The street dropped about 4 ft. and moved eastward about 6 ft. at the maximum point. The 24-in. [0.6 m] main on the left was laid after the fire. All buildings in this vicinity were burned."
2) Howard and 17th	8b		740	"On Howard St., at the corner of 17th St., very complete destruction of the brick sewer, as well as most other pipes and conduits in the ground, was caused by the extensive settlements in the region. On valencia St., near 18th St., similar ruptures were produced."
	9	Fig. 20	554	Photo caption: "GENERAL WRECKAGE OF CAR TRACKS, SEWER, WATER AND GAS PIPES, HOWARD AND 17TH STS., SAN FRANCISCO."

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				Photo: Shows car track pulled apart, destruction of pipes. Two pipes [0.3 and 0.51 m ?] pulled out at the joints.
	9		554	"Fig. 20 shows a street view, taken April 25, at the corner of Howard and Seventeenth Sts., where the ground was much distorted. The car tracks are twisted out of shape, the brick sewer is broken and the water and gas pipes are wrenched and snapped. A redwood plug has been driven into the broken water main, but the pipe is badly leaking. Views of this kind were common in the first week after the fire."
	1	No. 66	97	Photo: Shows subsidence of street. Does not show pipes. Photo caption: "Crossing of 17th & Howard. Street sunken, 12" & 20" [0.3 & 0.51 m] pipes ruptured."
3) Howard St. 18th	1	No. 62	95	Photo: Shows lateral and spread and street subsidence; does not show pipes. Photo caption: "Howard St. north of 18th sunken & twisted (see car tracks). One 20" [0.51 m] pipe, one 24" [0.6 m] and two 6" [0.15 m] pipes broken by earthquake."
	1	No. 68	97	Photo: Shows street subsidence and lateral spread; does not show pipe. Photo caption: "Along 18th from Howard. Street subsided, pipe broken."
4) 14th near Howard	8b	---	737	Photo: Shows street subsidence near sewer. Complete destruction by fire to left of photo. Photo caption: "Effect of Broken Sewer on Fourteenth Street....The second picture shows the result of the destruction of a sewer on Fourteenth St., near Howard St. The crown of the sewer was broken in, the surface material washed in, as at A, and the sewer so completely filled with sand as to back up the sewage, as at B, above the elevation of the crown."
	8b		740	"On 14th St., between Mission and Howard Sts., practically the same effects were produced. The crown was broken in many places and sand from the street filled the channel to such an extent that sewage was backed up above the original crown line. These breaks were indicated on the surface by great holes where all the covering, including the stone block paving, had settled down into the sewer."
5) 14th and Valencia Sts.	1	No. 63	95	Photo caption: "14th St. east of Valencia. Street torn open. See water pipe below."

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
SOUTH OF MARKET:				
1) Howard and 7th	1	No. 70	97	Photo: Shows broken pipe; pipe sheared abruptly near the joints from soil displacements. Photo caption: "Crossing 7th & Howard St. 16" pipe badly fractured (not yet repaired)."
2) 5th St.	8b		740	The more important breaks in main sewers observed by the writer may be described as follows: On Fifth St., between Folsom and Harrison Sts., the brick sewer settled with the adjacent land, its crown was destroyed and the channel had become filled with sand which blocked up the sewage to a dangerous extent.
	1		43	"APPENDIX D....The Fourth and Sixth street sewers were also greatly damaged, some of them showing a vertical and horizontal movement of as much as five or six feet."
FOOT OF MARKET ZONE:				
	13		32	"Throughout the filled area above street-corner silt basins have been tripped out of plumb and bulged into sidewalk areas, and sewer manholes in street intersections are in several places canted up, showing sewer disturbance beneath..."

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
OTHER LOCATIONS:				
1) Vallejo St. Van Ness Ave.	1	No. 74	99	Photo: Shows street and subsidence and lateral movement; does not show pipe. Photo caption: "van Ness Ave. near Vallejo. 8" [0.2 m] water pipe ruptured."
	4		118	"At one point on Van Ness avenue (see B, Pl. LVI) [shows location on map], where I happened to see the mains uncovered, a heavy water pipe, apparently about 20 inches [0.51 m] in diameter, had been broken into pieces not more than 2 feet [0.6 m] long. The total length of the break, however, was not more than 40 or 50 feet [12 to 15 m], so far as I could judge from what I saw uncovered."
	1		43	"The vicinity of Van Ness avenue and Vallejo street is one of the prominent points of interest. It was found that Van Ness avenue had been more or less affected from a point 150 feet [46 m] south of Vallejo street, the greatest subsidence being two feet [0.6 m] at the crossing of Vallejo street. There was also subsidence of Vallejo street from 150 feet [46 m] on each side of Van Ness avenue. There was a lateral movement to the north on Van Ness avenue of about three feet [0.9 m] on Vallejo street, decreasing to about one

LOCATION	REF	PHOTO	PAGE	DESCRIPTION
				foot [0.3 m] on Green street, the ground and buildings upon it having been moved bodily so that now the buildings encroach upon the neighboring lots or upon the street. As a result of the subsidence and lateral movement the sewers extending east, south and west of the crossing of Van Ness avenue and Vallejo street were broken for about 150 feet [46 m]."
2) Union and Steiner Sts.	1	No. 79	99	Photo caption: "Union St. between Steiner & Pierce St. Street, cable road, sidewalks, water & other pipes & sewer moved laterally several feet [0.3 to 0.6 m]. Note: A-A_1-B shows former position of center slot of North Track. (looking west.)"

REFERENCES - TABLES A.1 AND A.2

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